INTRODUCTION

This notebook is intended to be a working tool that provides a readily available compilation of current FHWA policy and guidance on pavements. Users are encouraged to add material as they see fit.

The notebook is composed of:

(1) Reference to appropriate Federal-aid Highway Program Manual directives;

(2) Other issuances, such as Technical Advisories and Notices which present short-term instructions or interim policy;

(3) FHWA memorandums clarifying policy or providing technical guidance;

(4) Discussions reflecting current state-of-the-art or philosophy;

(5) Material on developmental and research areas related to pavements.

The material is arranged by subject into chapters and sections. The Table of Contents shows current date for each document.

Any comments, suggested additions, or revisions to the notebook should be directed to the Federal Highway Administration, Attn: Mr. Peter J. Serrano, Pavement Division, HNG-46, 400 Seventh St., S.W., Washington, D.C.; Telephone number 202.366.1341 or email at *Peter.J.Serrano@fhwa.dot.gov.*

Enclosed is the second revision to the *Pavement Notebook For FHWA Engineers*. Please make the changes contained in the attachment. Submit the attached form on the following page so that we can include your name and address on our mailing list. For further information or additional copies of the notebook contact Mr. Peter J. Serrano at 202.366.1341 or *Peter.J.Serrano@fhwa.dot.gov.*

Refer to: HNG-40

Chief, Pavement Division
Federal Highway Administration
400 Seventh Street, SW., Room 3118
Washington, D.C. 20590-0001

Attn: Mr. Peter J. Serrano, P.E.

Dear Sir:

I have received a copy of the *Pavement Notebook for FHWA Engineers* and would like to be on your distribution list for future updates and/or additions to the notebook.

Request for additional copies should be addressed to:

Federal Highway Administration
Pavement Division - Attn: Mr. Peter J. Serrano, P.E.
Pavement Design and Rehabilitation Branch (HNG-42)
400 Seventh Street, S.W.
Washington, D.C 20590

Please mail or fax the form below.

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Name: ______________________________

Title: ______________________________

Agency: ______________________________

Address: ______________________________

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Telephone Number: ______________________________

Federal Highway Administration - Pavement Division
Attn: Mr. Peter J. Serrano, P.E. (HNG-42);
Fax number: 202.366.3713

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Chapter 1

Pavement Policy

AP-21 Geotechnical Microcomputer Programs

DESCRIPTION : This project has involved the development of several geotechnical programs under contract with geotechnical microcomputer programming firms. These programs have been made available to the States by the OTA.

BACKGROUND : The microcomputer industry has undergone rapid changes in recent years. New developments in hardware and software make the use of the microcomputer in civil engineering applications more feasible, practical, and almost indispensable.

The microcomputer can be used to solve many geotechnical problems that need repetitive and yet complicated calculations, such as analyzing embankment and foundation deformations, estimating pile behavior under static and dynamic forces, and calculating foundation settlements. Five of the microcomputer programs developed or under development are:

COM624P: Analyzes the behavior of piles or drilled shafts, subjected to lateral loads using the p-y method.

EMBANK: Determines one-dimensional compression settlement because of embankment loads.

SPILE: Calculates the ultimate static pile capacity in cohesive and cohesionless soils.

RSS: Analyzes stability of slopes that contain soil reinforcement. The analysis is performed using a two-dimensional limiting equilibrium method.

MSEW: Designs and/or analyzes required reinforcement for mechanically stabilized retaining walls, which does not consider specific facing configurations.

DRIVEN: This program is the updated version of the SPILE Program.

PILE FOUNDATION : This program will be developed based on the University of Florida program - LPGSTAN which is capable of analyzing bridge foundations subject to extreme events (hurricanes, ship and ice imports). The program will extend its capabilities to include the analysis and design of sound walls, retaining walls, signs and high mast lighting structures.

PROJECT MANAGER : Chien-Tan Chang, HTA-22, (202) 366-6749

STATUS : The SPILE Program has been upgraded, the new program is called Driven. This program is estimated to be completed by the end of 1995. RSS Program has been completed. It will be tested for about 2 months and will be distributed early December 1995. Contracts are being negotiated to develop a new version of MSEW program and a multiple faceted program called Pile Foundations.

CHAPTER 1

PAVEMENT POLICY

1.1 Pavement Design and Management Requirements

- Pavement Management System, 23 CFR 500, Subpart B, April 22, 1994.
  - 500.201, Purpose
  - 500.203, PMS definition
  - 500.205, PMS general requirements
  - 500.207, PMS components
  - 500.209, PMS compliance scheduling
- Non-Regulatory Supplement, October 05, 1995.
  - 500.205, General Pavement Design Considerations

1.2 ISTEA Pavement Management Systems

- Action: ISTEA Pavement Management Systems, November 4, 1994
  - Technical Guidance

1.3 Cost Comparison of Asphalt versus Concrete Pavement, OIG Final Report, July 26, 1994.

1.4 Proposed Final Interstate Maintance Fund Transfer Policy, September 21, 1994).

- Transfer of Interstate Maintance Program Funds, Proposed Final Policy Statement, Federal Register, September 02, 1994).
- Transfer of Interstate Maintance Program Funds, Interim Policy Statement, Federal Register, March 03, 1993).

SUBCHAPTER F - TRANSPORTATION INFRASTRUCTURE MANAGEMENT

PART 500 - MANAGEMENT AND MONITORING SYSTEMS

Subpart B - Pavement Management System

Authority: 23 U.S.C. 134, 135, 303 and 315; 49 U.S.C. app. 1607;
23 CFR 1.32; and 49 CFR 1.48 and 1.51.

Source: 58 FR 63475, Dec. 1, 1993 [Effective Jan. 3, 1994]

Sec. 500.201 Purpose.

The purpose of this subpart is to set forth requirements for development, establishment, implementation, and continued operation of a pavement management system (PMS) for Federal-aid highways in each State in accordance with the provisions of 23 U.S.C. 303 and subpart A of this part.

Sec. 500.203 PMS definitions.

Unless otherwise specified in this part, the definitions in 23 U.S.C. 101(a) and Sec. 500.103 are applicable to this subpart. As used in this part:

Pavement design means a project level activity where detailed engineering and economic considerations are given to alternative combinations of subbase, base, and surface materials which will provide adequate load carrying capacity. Factors which are considered include: materials, traffic, climate, maintenance, drainage, and life-cycle costs.

Pavement management system (PMS) means a systematic process that provides, analyzes, and summarizes pavement information for use in selecting and implementing cost-effective pavement construction, rehabilitation, and maintenance programs.

Sec. 500.205 PMS general requirements.

(a) Each State shall have a PMS for Federal-aid highways that meets the requirements of Sec. 500.207 of this subpart.

(b) The State is responsible for assuring that all Federal-aid highways in the State, except those that are federally owned, are covered by a PMS. Coverage of federally owned public roads shall be determined cooperatively by the State, the FHWA, and the agencies that own the roads.

(c) PMSs should be based on the concepts described in the ``AASHTO Guidelines for Pavement Management Systems.'' [AASHTO Guidelines for Pavement Management Systems, July 1990, can be purchased from the American Association of State Highway and Transportation Officials, 444 N. Capitol Street, NW., suite 225, Washington, DC 20001. Available for inspection as prescribed in 49 CFR part 7, appendix D.]

(d) Pavements shall be designed to accommodate current and predicted traffic needs in a safe, durable, and cost-effective manner.

Sec. 500.207 PMS components.

(a) The PMS for the National Highway System (NHS) shall, as a minimum, consist of the following components:

(1) Data collection and management.

(i) An inventory of physical pavement features including the number of lanes, length, width, surface type, functional classification, and shoulder information.

(ii) A history of project dates and types of construction, reconstruction, rehabilitation, and preventive maintenance.

(iii) Condition surveys that include ride, distress, rutting, and surface friction.

(iv) Traffic information including volumes, classification, and load data.

(v) A data base that links all data files related to the PMS. The data base shall be the source of pavement related information reported to the FHWA for the HPMS in accordance with the HPMS Field Manual. [Highway Performance Monitoring System (HPMS) Field Manual for the Continuing Analytical and Statistical Data Base, DOT/FHWA, August 30, 1993, (FHWA Order M5600.1B). Available for inspection and copying as prescribed in 49 CFR part 7, appendix D.]

(2) Analyses, at a frequency established by the State consistent with its PMS objectives.

(i) A pavement condition analysis that includes ride, distress, rutting, and surface friction.

(ii) A pavement performance analysis that includes an estimate of present and predicted performance of specific pavement types and an estimate of the remaining service life of all pavements on the network.

(iii) An investment analysis that includes:

(A) A network-level analysis that estimates total costs for present and projected conditions across the network.

(B) A project level analysis that determines investment strategies including a prioritized list of recommended candidate projects with recommended preservation treatments that span single-year and multi-year periods using life-cycle cost analysis.

(C) Appropriate horizons, as determined by the State, for these investment analyses.

(iv) For appropriate sections, an engineering analysis that includes evaluation of design, construction, rehabilitation, materials, mix designs, and preventive maintenance as they relate to the performance of pavements.

(3) Update. The PMS shall be evaluated annually, based on the agency's current policies, engineering criteria, practices, and experience, and updated as necessary.

(b) The PMS for Federal-aid highways that are not on the NHS shall be modeled on the components described in paragraph (a) of this section, but may be tailored to meet State and local needs. These components shall incorporate the use of the international roughness index or the pavement serviceability rating data as specified in Chapter IV of the HPMS Field Manual.

Sec. 500.209 PMS compliance schedule.

(a) By October 1, 1994, the State shall develop a work plan that identifies major activities and responsibilities and includes a schedule that demonstrates full operation and use of the PMS on the NHS by October 1, 1995, and on non-NHS Federal-aid highways by October 1, 1997.

(b) By October 1, 1995:

(1) The PMS for the NHS shall be fully operational and shall provide projects and programs for consideration in developing metropolitan and statewide transportation plans and improvement programs; and

(2) PMS design for non-NHS Federal-aid highways shall be completed or underway in accordance with the State's work plan.

(c) By October 1, 1997, the PMS for non-NHS Federal-aid highways shall be fully operational and shall provide projects and programs for consideration in developing metropolitan and statewide transportation plans and improvement programs.

U. S. Department of Transportation

Federal Highway Administration

Federal-Aid Policy Guide

Subject

FEDERAL-AID POLICY GUIDE - CHANGE

Date

Transmittal Number

14

1. PURPOSE. To transmit new and revised pages to the Federal-Aid Policy Guide (FAPG).

2. COMMENTS. The FAPG is being updated to include the following items.

   a. Federal-aid regulations previously published in the Federal Register.

      (1) Revised sections: (a) 23 CFR Part 630, Preconstruction Procedures, (b) 23 CFR Part 637, Construction Inspection and Approval, (c) 23 CFR Part 645, Utilities, and (d) 49 CFR Part 18, Grants and Cooperative Agreements to State and Local Governments.

      (2) Removed section: 23 CFR Part 1204, Uniform Guidelines for State Highway Safety Programs.

   b. Supplemental sections NS 23 CFR 140G, NS 23 CFR Part 500, NS 23 CFR Part 635D, NS 23 CFR Part 645A and NS 23 CFR Part 660A have been revised.

   c. Revised pages to the Table of Contents are also included with this transmittal.

3. REGULATORY MATERIAL. The regulatory material contained in this directive has been published in the Federal Register and will be codified in Title 23, Code of Federal Regulations.

4. ACTION. Each recipient office is responsible for filing the attached FAPG pages into the binders provided.

Moore

George S. Moore, Jr.
Associate Administrator
for Administration

DISTRIBUTION: OPI:

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INTENTIONALLY

NON-REGULATORY SUPPLEMENT

OPI: HNG-42

1. GENERAL PAVEMENT DESIGN CONSIDERATIONS (23 CFR 500.205(d))

Title 23 CFR 500.205(d) establishes the following requirement: "*Pavements shall be designed to accommodate current and predicted traffic needs in a safe, durable, and cost-effective manner.*" The regulations do not specify the procedures to be followed to meet this requirement. Rather each State Highway Agency (SHA) is expected to use a design procedure which is appropriate for their conditions. The SHA may use the design procedures outlined in the AASHTO Guide for Design of Pavement Structures or they may use other pavement design procedures that, based on past performance or research, are expected to produce satisfactory pavement designs.

a. FHWA Evaluation of Pavement Design Procedures

(1) Consistent with FHWA's Operational Philosophy on process review/product evaluation (PR/PE) attached to Executive Director Carlson's November 12, 1991 memorandum, the FHWA field offices will conduct periodic reviews of the SHA's pavement design process. As part of the review, FHWA field offices will sample a sufficient number of projects to determine that the pavement design process is being followed and the process provides reasonable engineering results. If the reviews show that the SHAs have and are following an acceptable pavement design process, routine pavement design reviews of individual projects will not be required.

(2) The FHWA encourages the development of mechanistic pavement design procedures. To promote consistency in application of mechanistic related design procedures,

the Pavement Division will participate with the Region and Division offices in reviewing and discussing these procedures with the State during their development.

b. Factors to Consider in Pavement Design.

Highway agencies should pay particular attention to the following items in designing pavements.

(1) Traffic. Pavement designers should work closely with the SHA component responsible for the development of the Traffic Monitoring System for Highways (TMS/H) required under 23 CFR 500.801. The TMS/H should reflect the accuracy of traffic volume, classification, and truck weight data required for pavement design.

(a) Accurate cumulative load (normally expressed as 18 kip equivalent single axle loads or ESALs) estimates are extremely important to structural pavement design. Load estimates should be based on representative current vehicle classification and truck weight data and anticipated growth in heavy truck volumes and weights. Representative current traffic data should be obtained using statistically valid procedures for obtaining count, classification, and weight data based on the concepts described in the FHWA "Traffic Monitoring Guide" and the "AASHTO Guidelines for Traffic Data Programs."

(b) Accurate vehicle classification data on the number and types of trucks is essential to estimating cumulative loads during the design period and should be given special emphasis. Weight information should be obtained using weigh-in-motion (WIM) equipment since this data is more representative than data obtained using static enforcement scales which are plagued with avoidance problems. States should continue to automate their monitoring program through installation of strategically placed automatic vehicle classification and WIM systems as soon as possible to improve the current base traffic data used to forecast future truck volumes and loads.

(c) The SHA's forecasts of future loadings should, as a minimum, be based on two truck classes: trucks up to 4-axle combination and trucks with 5-axles or more. Changes in load factors should also be monitored and forecasted. The forecasting procedures should consider past trends and future economic activity in the area. A traffic data collection and forecasting program that identifies the most important truck types and the changes in numbers and weights of these truck types during the design period should provide realistic load estimates.

(2) Roadbed Soils. Both the 1986 and 1993 versions of the "AASHTO Guide For Design of Pavement Structures" require the use of the Resilient Modulus (M_R) (a measure of the elastic property of soils) in lieu of soil support value as the basic materials value to characterize roadbed soils for flexible pavements. The AASHTO Guide strongly recommends that SHAs acquire the necessary equipment to measure M_R. SHAs who use M_R values converted from CBR and R-value should conduct correlation studies using a range of soil types, saturation levels, and densities to determine realistic input values. For rigid pavements, the

use of a k-value is required. NCHRP Report 372, Support Under Portland Cement Concrete Pavements, provides improved guidance on selecting appropriate values for this factor. Proper roadbed soil support is needed for longer pavement service lives and more cost-effective pavement design.

(3) Drainage

(a) Drainage is one of the more important factors in pavement design, yet inadequate subsurface drainage continues to be a significant cause of pavement distress, particularly in portland cement concrete pavements. During the last 10 years significant strides have been made in the development of positive drainage systems for new and reconstructed pavements. There have also been major developments in products and materials which can be used for retrofit longitudinal edgedrains.

(b) The developments in permeable base technology and longitudinal edgedrains make positive pavement drainage possible and affordable. Accordingly, pavement design procedures need to consider the effects of moisture on the performance of the pavement. Where the drainage analysis or past performance indicates the potential for reduced service life due to saturated structural layers or pumping, the design needs to include positive measures to minimize that potential.

(4) Shoulder Structure

(a) Recent studies demonstrate that full structural shoulders improve both mainline pavement and shoulder performance. Research results have shown that widening the right pavement lane and placing the edge stripe 0.5 m from the outside pavement edge significantly improves pavement performance.

(b) The SHAs are encouraged to use paved shoulders where conditions warrant. Shoulders should be structurally capable of withstanding wheel loadings from encroaching truck traffic. On urban freeways or expressways, strong consideration should be given to constructing the shoulder to the same structural section as the mainline pavement. This will allow the shoulder to be used as a temporary detour lane during future rehabilitation or reconstruction.

(c) On new and reconstructed pavement projects, the SHAs are encouraged to investigate the advantage of specifying that the shoulder be constructed of the same materials as the mainline, particularly on high-volume roadways. Constructing shoulders of the same materials as the mainline facilitates construction, reduces maintenance costs, improves mainline pavement performance, and provides additional flexibility for future rehabilitation.

(5) Engineering and Economic Analysis.

The design of both new and rehabilitated pavements should include an engineering and economic evaluation of alternative strategies and materials. The project specific analysis should be evaluated in light of the needs of the entire system. Appendix B of the 1993 "AASHTO Guide for Design of Pavement Structures," and the "FHWA Pavement Rehabilitation Manual," provide guidance on engineering considerations. The Engineering evaluation should include consideration of the use of recycled materials or pavement recycling techniques where feasible. Economic considerations include an economic analysis based on Life Cycle Costs (LCC). The FHWA interim policy statement on LCC analysis published in the July 11, 1994 Federal Register provides guidance on LCC Analysis.

(a) Pavements are long term public investments and all the costs (both agency and user) that occur throughout their lives should be considered. LCCA identifies the long term economic efficiency of competing pavement designs. However, the resulting numbers themselves are less important than the logical analysis framework fostered by LCCA in which the consequences of competing alternatives are evaluated. When performing LCCA for pavement design, the variability of input parameters needs to be considered. The results of LCCA should be evaluated to determine whether differences in costs between competing alternatives are statistically significant. This evaluation is particularly important when the LCC analysis reflects relatively small economic differences between alternatives.

(b) The FHWA's policy on alternate bids, which would include bids for alternate pavement types, is addressed in 23 CFR 635.411(b). This section requires the use of alternate bid items "When ... more than one... product... will fulfill the requirements... and these ... products are judged... equally acceptable on the basis of engineering analysis and the anticipated prices... are estimated to be approximately the same.

(1) The FHWA does not encourage the use of alternate bids to determine the mainline pavement type, primarily due to the difficulties in developing truly equivalent pavement designs.

(2) In those rare instances where the use of alternate bids is considered, the SHA's engineering and economic analysis of the pavement type selection process should clearly demonstrate that there is no clear cut choice between two or more alternatives having equivalent designs. Equivalent design implies that each alternative will be designed to perform equally over the same performance period and have similar life-cycle costs.

c. <u>Rehabilitation Pavement Design</u>. It is essential that rehabilitation projects be properly engineered to achieve the best return possible for the money expended. When an existing pavement structure is sound and the cost to restore serviceability is minor when compared to the cost of a new pavement structure or major rehabilitation, an engineering and economic analysis of alternative actions may not be necessary. In general, for all major rehabilitation projects, each of the following steps should be followed to properly analyze and design the project.

(1) <u>Project Evaluation</u>

(a) Obtain the necessary information to evaluate the performance and establish the condition of the in-place pavement with regard to traffic loading, environmental conditions, material strength, and quality. Historical pavement condition data, obtained from the Pavement Management System (PMS), can provide good initial information.

(b) Identify the types of pavement distresses and the factors causing the distresses before developing appropriate rehabilitation alternatives. The tools necessary to analyze pavement failures, such as coring, boring, trenching, and deflection measurements, are well known, and need to be employed more often.

(c) Evaluate the array of feasible alternatives in terms of how well they address the causes of the deterioration, repair the existing distress, and prevent the premature reoccurrence of the distress.

(2) Project Analysis

(a) Perform an engineering and economic analysis of candidate strategies. The engineering analysis should consider the traffic loads, climate, materials, construction practices, and expected performance. The economic analysis should be based on life cycle cost and consider service life, initial cost, maintenance costs, user costs, and future rehabilitation requirements, including maintenance of traffic.

(b) Select the rehabilitation alternative which best satisfies the needs of a particular project considering economics, budget constraints, traffic service, climate, and engineering judgment.

(3) Project Design

(a) Conduct sufficient testing, both destructive and non-destructive, to verify the assumptions made during the alternative evaluation phase. The SHAs should consider a new distress survey if the original condition survey was sample based or if the survey is not current in terms of the time the project is scheduled to go to contract.

(b) Consider and address all factors causing the distress in addition to the surface indicators in the final design. Such factors as structural capacity, subgrade support, surface and subsurface drainage characteristics need to be considered and provided for in the final design.

(c) Once a rehabilitation alternative is selected, design the project using appropriate engineering techniques. A number of publications are available to guide the selection of these engineering techniques. The FHWA's "Pavement Rehabilitation Manual," and training course "Techniques for Pavement Rehabilitation" provide excellent guidelines. There are also a number of excellent guides available from the asphalt and concrete industries.

(4) <u>Project Implementation</u>

(a) Document the intent of the design in the project plans and specifications to provide both the contractor and the construction engineering personnel a clear and concise project proposal. In addition, maintain adequate communication between the design and construction engineers. This will reinforce the intent of the design and provide feedback on project constructability and performance to aid timely evaluation of the selected rehabilitation alternative.

(b) The performance information should also be included as a part of the SHA's PMS. The lack of good performance data on pavement rehabilitation techniques is one of the weaker points in the rehabilitation process. Increased emphasis should be placed on developing basic performance and maintenance cost data on rehabilitation techniques where performance data is not presently available.

2. SAFETY (23 CFR 500.205d)

a. The SHAs should provide skid resistant surfaces on all projects, regardless of funding source. New pavement surfaces constructed with Federal funds must have skid resistant properties suitable for the needs of the traffic. New pavement surfaces on projects where a skid resistant surface was previously constructed with Federal funds must have skid resistant properties suitable for the needs of the traffic even if not now financed with Federal-aid funds.

b. The SHAs should analyze pavement performance histories and existing skid data to ensure that the materials, mix designs, and construction techniques used are capable of providing a satisfactory skid resistant surface over the expected performance period of the pavement. Each SHA's skid accident reduction program should include a systematic process to identify, analyze, and correct hazardous skid locations. The SHA's should use the same construction procedures and quality standards used in constructing new pavements in pavement maintenance operations.

c. Plans and specifications for proposed pavement rehabilitation and reconstruction projects should include items to minimize disruption and ensure adequate protection of the motorists and workers within the construction work zone in accordance with the provisions of 23 CFR 630, subpart J and 23 CFR 635, subpart A.

NOV 04 1994

ACTION: ISTEA Pavement Management Systems

Director, Office of Engineering HNG-41

Regional Administrators

We are approaching the first bench mark in implementing the Pavement Management System (PMS) provisions in ISTEA. By January 1, 1995, each State is required to submit to the division office the certification statement, work plan, and status for implementing its PMS. The division office should review the submission and forward its comments and a copy of the documents to the region. The regional office has the responsibility to review and accept the submission and notify the division office accordingly.

The purpose of this memorandum is twofold. First, we want to provide technical guidance and criteria in order to implement the PMS provisions in ISTEA in a complete and consistent manner. Secondly, we request your cooperation and assistance in providing us with PMS information, so we can continue to monitor the States' progress in developing and implementing their PMS's.

1. During the past months, we have assisted several field offices in reviewing draft work plans and noted some deficiencies and inconsistencies that warrant attention. Presently, we need to focus on four technical items: (1) multi-year prioritization, (2) life-cycle cost analysis, (3) condition survey distresses, and (4) condition survey samples. Attached is technical guidance on these four items for your use. We have reiterated some of the fundamentals of PMS for the benefit of the States and divisions who are experiencing a high turnover and influx of engineers and managers who are new to PMS.

2. For the past 8 years the Pavement Management Branch has maintained a national database on the status of the States' PMS's that is used to assess and guide the national PMS program. With the advent of the ISTEA certification process, the information in the database will continue to play an important role in managing the national program. As you know, the information has always been collected and reported by the FHWA staff. We are requesting your cooperation and assistance to have the division office PMS specialists update this information when they concurrently review the States' PMS certifications and work plans. Please send the completed PMS Survey form (copy attached) to the Pavement Management Branch, HNG-41 by January 17, 1995.

Implementing the PMS provisions in ISTEA is of vital importance to FHWA. The key to success is a strong joint effort between Headquarters and the field offices. We will continue to provide technical guidance and direction as needed to help achieve a comprehensive and consistent PMS program. If you have any questions, or need technical assistance, please contact Mr. Frank Botelho at 202-366-1336.

William A. Weseman

William A. Weseman

TECHNICAL GUIDANCE

1. Multi-Year Prioritization. Multi-year prioritization is the heart of a PMS. It provides a prioritized listing of projects for which rehabilitation/preservation actions are recommended for each year of the planning horizon. The multi-year prioritized list of candidate projects and treatments is a "first cut" list that is normally produced by the Pavement Management Engineer(s) and submitted to the appropriate offices in the Agency to be used as input in developing the statewide pavement preservation program. The prioritization is based on priority factors, predicted performance, and economic analysis relative to the goals set by the State for its network. The candidate projects should have a high benefit cost ratio based on life-cycle cost analysis. The prioritization process must be objective, analytical, formalized, and automated (computerized for State and large local networks) in order to be stable and repeatable with time and changing of personnel. Its established engineering criteria and analytical methodology are the basis and means of producing and documenting an accountable and justifiable pavement preservation program.

   Many States have not yet established or utilized the above criteria for multi-year prioritization. Rather, they are prioritizing projects solely on a subjective, manual, and "worst first" basis. The field offices need to promote and support major efforts by the State highway agencies (SHA's) to satisfy the intent of our regulation on multi-year prioritization.

2. Life-Cycle Cost Analysis. The need and purpose for life-cycle cost analysis is strongly emphasized in ISTEA. The FHWA issued an interim policy statement on life-cycle cost on July 11, 1994. This policy statement should be used by the field when evaluating the States' life-cycle cost analysis procedures. Prioritization and life-cycle cost analysis are the analytical basis for demonstrating that the expenditure of Federal-aid funds are justifiable and cost effective.

   A State PMS must include a life-cycle cost analysis (that is commensurate with the level of investment and types of preservation treatments) for candidate projects in order to compare alternative treatments and strategies to produce a cost effective preservation program that satisfies the goals of the Agency. The life-cycle cost analysis should be based on the performance prediction and economic models used in multi-year prioritization. Life-cycle cost analysis of specific project treatments should consider future treatments required to maintain the pavement until reconstruction. Life-cycle cost analysis of network-level strategies requires an analysis period of at least one complete cycle in the life of the network, which should be at least 35 years.

3. <u>Condition Survey Distresses</u>. Pavement condition data are the foundation for measuring and monitoring: the "health" of the network; the current and predicted performance of pavements; and the remaining service life of the network. A PMS condition survey bridges the "information gap" between general planning data and detailed design data. Condition data are combined with performance data, life-cycle cost analysis, and priority factors to develop the multi-year list of prioritized projects. The type, extent, and severity of the individual distresses are also used to determine viable preservation treatments.

   The types of distresses that are measured in a pavement condition survey should be chosen on the basis that they support the decisions on where, when, and how to preserve the network. A "sufficiency rating" (commonly used for planning purposes) or a single distress survey do not constitute a PMS condition survey. The premise of using either one as a "common denominator" does not provide the engineering detail needed in PMS's.

4. <u>Condition Survey Samples</u>. The reliability of condition data is crucial to the credibility of a PMS. The least amount of error will occur if 100 percent of the pavement is sampled. The viability of sampling 100 percent is only possible when using automated survey equipment, such as the equipment that is currently used to measure roughness, rutting, and faulting. In the absence of automated equipment, SHA's customarily measure distress data using an approximate 10 percent representative sample. That is, a 10 percent sample on each and every mile of the network. This may somewhat increase or decrease depending on the variability in pavement condition.

   Because of the expanded network coverage of ISTEA (i.e., a total of 936,000 centerline miles of Federal-aid highway), some SHA's are exploring cost cutting measures to reduce the added burden of collecting pavement condition data. Generally, reducing the number of distresses or reducing the sample size does not result in real cost savings because of the increased risk of errors in PMS. However, SHA's can achieve real cost savings by reducing the <u>frequency</u> of the condition surveys. Condition surveys can be conducted every 2 years instead of every year. Biennial surveys should be supplemented with annual updates for newly improved sections and when unexpected changes occur caused by either the environment, loading, premature failures, or accelerated deterioration.

While these fundamental criteria apply to all Federal-aid highways, we want t prevent unnecessary data collection and analysis burdens, so please remind PM practitioners that the level of effort needed to do items 1, 2, and 3 is far less for lower order roads than for the proposed National Highway System.

Date____

NHS PMS SURVEY

(Question II(A) applies to both the NHS and Non-NHS)

I. ORGANIZATION

A. State ____________________

B. FHWA Region ____

C. State Staffing Resources

The following staffing information pertains only to the staff at the central office. It does not apply to district staff or field data collection crews.

1. Does the SHA have a person who is designated as the State's PMS Engineer[1]? Yes____ No____ (If no, still provide a name, address, etc. for the point of contact).

Name ____________________
Address ____________________
____
____
City ____________ ST ___ Zipcode______ PlusFour________
Phone ____________ FAX ____________

2. Does the PMS Engineer work full time on PMS? Yes____ No____ If part-time, what percentage is spent on PMS? Part-Time Percentage______.

3. Does the PMS Engineer have the full responsibility and authority to lead the development, implementation, and operation of PMS? Yes____ No____.

4. If NO, how is PMS managed?

____
____

5. If the PMS engineer has an assistant(s), staff, or in-house support; indicate each position(person's name), percent time spent on PMS, and a brief description of their primary function(s). This pertains only to the central office and excludes condition survey crews.(Add additional names on separate sheet.)

	Name	Percent Time	Primary Function(s)
a.			
b.			
c.			

[1]PMS Engineer is the person who is in charge of leading and working on developing, implementing, and operating the PMS on a day-to-day basis.

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D. Does the State have an active PMS committee(s) or group(s) that guide and update the PMS? Yes ____ No ____. Provide the positions(i.e. pavement design. materials. etc.) of PMS committee(s) members on an attached sheet.

II. PMS DATABASE

A. PMS Coverage

	Federal-aid Highway Mileage (Centerline)				
	Covered		Not Covered		
	NHS	Non NHS	NHS	Non NHS	Total
State					
Local					
Toll Roads					

B. Inventory Data	Yes	Under Development	Considering In Future	No
1. Pavement type	___	___	___	___
2. Pavement width	___	___	___	___
3. Shoulder type	___	___	___	___
4. Shoulder width	___	___	___	___
5. Number of lanes	___	___	___	___
6. Layer thicknesses	___	___	___	___
7. Joint spacing	___	___	___	___
8. Load transfer	___	___	___	___
9. Subgrade classification	___	___	___	___
10. Material properties	___	___	___	___
11. Resilient modulus	___	___	___	___
12. Drainage	___	___	___	___
13. Other (specify) ________________				

C. Project History	Yes	Under Development	No
1. Construction	___	___	___
2. Rehabilitation	___	___	___
3. Maintenance[2]	___	___	___

[2]"Maintenance" refers to preventive maintenance not corrective maintenance. Corrective maintenance refers to pot hole repair. etc.

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D. Condition Survey

	Yes	Under Development	Considering In Future	No	Equipment
1. Ride	___	___	___	___	________
2. Rutting	___	___	___	___	________
3. Faulting	___	___	___	___	________
4. Cracking	___	___	___	___	
5. Surface Friction	___	___	___	___	________
6. Network-level Deflection	___	___	___	___	________

E. Distress

	Yes	Under Development	Considering In Future	No
1. High speed windshield survey at 30 to 55 mph.	___	___	___	___
2. Low speed survey at 0 to 10 mph.	___	___	___	___
3. Combination of high and low speed.	___	___	___	___
4. 35mm film viewed at a workstation.	___	___	___	___
5. Video tape viewed at a workstation.	___	___	___	___
6. Distress Identification Manual with pictorial references used to calibrate extent and severity.	___	___	___	___
7. Fully automated.	___	___	___	___

Specify equipment: ________________________________

F. What is the frequency of condition data collection on the NHS? ______
____

G. How does the State collect their condition data?
In House ______ Contractor(specify) ________________

H. Traffic/Load Data

1. Does the PMS database contain

	Yes	Under Development	Considering In Future	No
a. Annual ESAL's	___	___	___	___
b. Forecast ESAL's	___	___	___	___
c. Cumulative ESAL's	___	___	___	___

2. Does the PMS have an ESAL flow map that is route specific?

Yes ____ Under Development ____ Considering in Future ____ No ____

I. Does the PMS provide IRI or PSR(circle one) to FHWA HQ for the HPMS sample sites?

Yes ____ Under Development ____ No ____

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J. Does the PMS have a relational database?

Yes ____ Under Development ____ No ____

K. How much work has been completed in developing the PMS database? Development work would include: establishing data files. collecting data. loading data. writing application programs for analysis. etc..

0-25% ____ 25-50% ____ 50-75% ____ 75-100% ____

III. INVESTMENT ANALYSES

A. Prioritization

1. Does the PMS office/unit produce a multi-year prioritized list of recommended candidate projects(this is considered a "first cut" list)?

Yes ____ Under Development ____ No ___

2. What method does the PMS use to produce the multi-year prioritized list of projects?

	Yes	Under Development	Considering In Future	No
a. Subjective[3]	____	____	____	____
b. Objective[4]				
1. Priority Model	____	____	____	____
2. Incremental Benefit Cost	____	____	____	____
3. Marginal Cost Effectiveness	____	____	____	____
4. Optimization				

	Yes	Under Development	Considering In Future	No
a. Linear Programming	____	____	____	____
b. Non-Linear Programming	____	____	____	____
c. Integer Programming	____	____	____	____
d. Dynamic Programming	____	____	____	____
e. Other (Specify) ________				

[3]"Subjective" indicates that the projects were prioritized by individuals using only personal knowledge of the roads.

[4]"Objective" means that the projects were prioritized using a repeatable analytical process.

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3. If the answer to question 2(b) is Yes or Under Development, who developed the software? In House ______ Contractor(specify) ____________.

4. Check the factors used to prioritize projects:

	Yes	Under Development	Considering In Future	No
a. Distress	___	___	___	___
b. Ride	___	___	___	___
c. Traffic	___	___	___	___
d. Functional class	___	___	___	___
e. Skid	___	___	___	___
f. Structural adequacy	___	___	___	___
g. Other (Specify)				

B. Preservation Treatment

1. Does the PMS assign a preservation treatment to a candidate project?

   Yes ____ Under Development ____ No ___

2. If the answer to question 1 is Yes or Under Development, which groups of treatments does the PMS cover?

	Yes	Under Development	No
a. Reconstruction	___	___	___
b. Rehabilitation	___	___	___
c. Maintenance[5]	___	___	___

3. What method is used to assign a preservation treatment to a candidate project.

	Yes	Under Development	Considering In Future	No
a. Subjective[6]	___	___	___	___
b. Objective[7]				
1. Matrix	___	___	___	___
2. Decision tree	___	___	___	___
3. Cost Benefit	___	___	___	___
4. Optimization Method listed previously.	___	___	___	___
5. Other (Specify)				

[5]"Maintenance" refers to preventive maintenance not corrective maintenance. Corrective maintenance refers to pothole repair, etc.

[6]"Subjective" indicates that the projects were prioritized by individuals using only personal knowledge of the roads.

[7]"Objective" means that the projects were prioritized using a repeatable analytical process.

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4. If the answer to question 3(b) is Yes or Under Development, who developed t. software? In House ____ Contractor(specify) ______________________

5. Does the PMS do a life-cycle cost analysis for the recommended preservation treatments?

   Yes ____ Under Development ____ No ___

6. If the answer to question 5 is Yes or Under Development, who developed the software? In House ____ Contractor(specify) ______________________

C. Pavement Performance Monitoring and Projection

1. Does the PMS monitor pavement performance?

   Yes ____ Under Development ____ No ___

2. Check all the pavement indices used to monitor pavement performance:

	Yes	Under Development	Considering In Future	No
a. Ride	___	___	___	___
b. Distress	___	___	___	___
c. Combined Index	___	___	___	___
e. Other (Specify)	______________________			

3. Is load data (cumulative ESAL's) used to monitor pavement performance?

   Yes ____ Under Development ____ Considering in Future ____ No ___

4. Does the PMS generate pavement performance curves?

   Yes ____ Under Development ____ Considering in Future ____ No ___

5. Are the curves developed for?

	Yes	Under Development	Considering In Future	No
Family of pavements	___	___	___	___
Each pavement	___	___	___	___

6. Does the PMS monitor and predict performance using?

	Yes	Under Development	Considering In Future	No
Markov Transition	___	___	___	___
Semi-Markov Transition	___	___	___	___

7. Does the PMS monitor pavement performance using another method? (specify). ______________________

____

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8. Does the PMS compute the Remaining Service Life of the network?

Yes ____ Under Development ____ No ___

9. If the answer to question 8 is Yes or Under Development, who developed the software? In House ____ Contractor(specify)______________________

IV. ENGINEERING ANALYSIS

A. Is the performance data in the PMS database used to evaluate either the accuracy, quality, or the cost effectiveness for:

	Yes	Under Development	Considering In Future	No
1. New pavement design procedures	____	____	____	____
2. Overlay design procedures	____	____	____	____
3. Rehabilitation techniques	____	____	____	____
4. Materials	____	____	____	____
5. Construction	____	____	____	____
6. Preventive maintenance	____	____	____	____
7. Mix designs	____	____	____	____
8. Other (Specify) ______________________				

V. PRODUCTS

A. Is the PMS's multi-year prioritized list of recommended projects used as input in the development of the State's:

	Yes	Under Development	No
1. Pavement Preservation Program	____	____	____
2. Statewide Transportation Improvement Program(STIP)	____	____	____
3. Transportation Improvement Program(TIP)	____	____	____

B. Is the PMS's multi-year prioritized list(first cut) compared to the final approved list of pavement preservation projects for reasonableness?

Yes ____ Under Development ____ Considering in Future ____ No ___

VI. UPDATE

Does the SHA annually evaluate and update the PMS relative to the agency's policies, engineering criteria, practices, experience, and current information?

Yes ____ Under Development ____ No ___

Revised 10/20/94

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: **INFORMATION**: OIG Final Report on the Audit of Cost Comparison of Asphalt Versus Concrete Pavement

Date: July 26, 1994

From: Rodney E. Slater
Administrator

Reply to Attn. of: HMS-11

To: The Honorable A. Mary Schiavo
Inspector General (JA-1)

We have completed our review of the final report on the Audit of Cost Comparison of Asphalt Versus Concrete Pavement in Region 4. Your transmittal memorandum requested that we reconsider our nonconcurrences with your recommendations and provide specific target dates and further clarification where we have agreed to corrective actions.

Our specific comments relative to each recommendation are contained in the attachment to this memorandum. For clarification, we have included our responses to the draft report, as well as a summary of the OIG comments on those responses in the attachment.

Our further review of the report reveals a fundamental philosophical difference in our approach to administering the Federal-aid highway program. This difference is specifically stated in the report's synopsis, alluded to in the report itself, and incorporated into many of the report's recommendations.

The philosophical difference is clearly articulated in the statement on page iv which reads as follows: "*. . .the continuing problem with FHWA's traditional strategy of facilitating, rather than mandating . . . .*" The report suggests that the FHWA needs to alter its operational relationship with State highway agencies (SHA) and adopt, as we interpret it, a strategy that is inconsistent with this Administration's approach toward customer service and minimizing mandates. We find this to be totally unacceptable and continue to nonconcur with that premise and in all recommendations in the report that would lead the FHWA in that direction.

The FHWA's basic philosophy of "*facilitating, rather than mandating*" is based upon the fact that the Federal-aid highway program is a federally assisted State program. The FHWA must administer it in that light. The Federal-aid highway program is fundamentally a formula allocated program. With finite allocations, SHAs are independently under intense fiscal pressure to assure the most efficient use of all highway dollars, whether they are Federal, State, or local dollars.

The FHWA's fostering of a cooperative partnership approach has served FHWA, the States, and the Nation well since its inception. This partnership approach was strengthened by the passage of the Intermodal Surface Transportation Efficiency Act of 1991. The FHWA continues to look toward bettering, not dismantling, this relationship in the future.

In response to the specific recommendations contained in the report, among other things, we have attached specific clarification and timetables for life-cycle cost analysis (LCCA) and pavement design activities as you requested. The FHWA believes that it is important to note that we have made significant progress over the last few years in both of these areas.

In the area of LCCA, we have reviewed the recent 1993 American Association of State Highway and Transportation Officials (AASHTO) survey of SHA applications of LCCA, conducted an FHWA/AASHTO symposium on LCCA in December 1993, and plan to publish an interim policy statement on LCCA. This policy statement will include recommendations on minimum analysis periods to be used and references Office of Management and Budget Circular A-94 for guidance on the selection of appropriate discount rates. The goal of this policy statement is to clearly define the FHWA's position on some of the more important components of LCCA, including analysis period, discount rate, and user costs. We intend to publish this policy statement in early summer.

It is important to note that we are making significant progress in this area and will be in a better position to further determine our course as current efforts evolve.

The same is true in the area of assuring high quality, cost-effective highway pavement design, construction, maintenance, and preservation. The new December 1993 Pavement Management System (PMS) regulation requires SHAs to develop comprehensive coordinated systems to effectively manage pavement to address current and evolving long-term pavement needs. It also broadens the pavement design requirements to include an analysis of the entire pavement structure (subgrade, subbase, base, and pavement). The regulation specifically requires that pavement design analysis consider life-cycle costs.

The FHWA intends to rewrite its Federal-Aid Policy Guide (FAPG) on pavement design to better track with the recently revised PMS regulation by the end of this calendar year. The revised FAPG, in conjunction with the new PMS regulation, will provide significantly more definitive guidance on pavement design. As noted in our earlier response, the FHWA agreed to direct its regional pavement engineers to participate with the divisions in pavement design and management reviews in each State during the next 2 years. Headquarters pavement engineers will participate in at least one of these reviews per region.

Further, we continue to stand by our original position, as stated in our September 2 memorandum, that the audit report does not support a finding of a material internal control weakness.

We appreciate the opportunity to comment on this draft report concerning the Audit of Cost Comparison of Asphalt Versus Concrete Pavement in Region 4.

2 Attachments

New Jersey reported the performance of their experimental permeable base pavement sections constructed in 1979-1980 at the 1988 Transportation Research Board Meeting. Their initial observations/findings on the AC sections were that the thinner sections were performing as well as the thicker sections with rutting being about the same. On PCC pavement sections, there was less deflection, no faulting or pumping, and substantially reduced frost penetration.

Pennsylvania rated the performance of their experimental permeable base sections constructed in 1980 much better than dense-graded aggregate base sections. Based on the positive interim results of these sections, a permeable base layer between the PCC pavement and dense-graded aggregate subbase became the State standard in 1983.(3)

Rideability

All of the States indicated that the rideability of permeable base pavements was no different than that on dense-graded bases. This was substantiated in California and North Carolina (asphalt cement treated) and Michigan (untreated). The rideability of some recently constructed PCC pavements in these States had been measured using the California and Rainhart profilographs at 0-5 inches per mile. In general, those States using a stringline for both horizontal and vertical control had a substantially better ride quality than those that did not. Also, those States that had incentives/disincentives for rideability had projects with very good ride quality.

Cost

Bids for permeable base materials were generally found to have slightly higher costs per unit weight than the impermeable dense-graded materials they replaced. Five of the seven States that used an untreated permeable base found that they were slightly more costly per unit measure than conventional dense-graded aggregate bases while two States, Iowa and Michigan, indicated that the unit costs for their permeable base material were the same or sometimes less.

As expected, the treated permeable base materials were two to three times more costly per unit measure than conventional dense-graded aggregate bases. However, all three States that predominantly used treated permeable base material found that the unit costs for it were about the same as those for dense-graded AC base. In addition, all three noted that because of the higher void content of the permeable material, the yield was 15-30 percent higher than dense-graded AC. California found that asphalt cement treated permeable base was generally less costly per unit measure than cement treated base (CTB) and lean concrete base (LCB). The material unit costs were the same or slightly more than asphalt concrete base but because of the large void content the yield was 20 percent higher. Kentucky, which had used some asphalt treated permeable base within the past year, also found that its

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: INFORMATION: Proposed Final Interstate Maintenance Fund Transfer Policy

Date: SEP 21 1994

From: Director, Office of Engineering

Reply to Attn of: HNG-42

To: Regional Administrators

Attached is a copy of the FHWA's proposed final policy statement on Interstate Maintenance Fund Transfers, which was published in the Federal Register on Friday, September 2. It addresses criteria relating to the decisions on adequate maintenance of the Interstate System for purposes of the Interstate Maintenance Program Transfer provisions of Title 23, United States Code, Section 119(f)(1). It is a proposed replacement for the Interim Maintenance Fund Transfer Policy, published at 58 Federal Register 12229, on March 3, 1993.

The proposed final policy statement would add safety and geometric criteria not originally proposed in the interim policy, and modify the existing criteria for pavements. Modifications to the pavement criteria would change the IRI criteria from 240 cm/km (150 inches/mile) to 200 cm/km (127 inches/mile), modify the faulting criteria to reflect a faulting rate of 525 mm/km (33 inches/mile) for both plain and reinforced jointed concrete pavements, and add a surface friction related criteria.

We have reopened the docket and will be accepting written public comments until November 1, 1994. We would appreciate it if FHWA field offices would adhere to that date in submitting any comments. Please note, that until we publish a final policy statement, the interim Interstate Fund Transfer Policy, published in the Federal Register on March 3, 1993, is still in effect and governs Interstate Maintenance Fund Transfer requests.

The Pavement Division continues to coordinate this effort for the Office of Engineering. Please direct any questions relating to this policy and/or its implementation to Mr. John Hallin. He can be reached at (202) 366-1323.

for William A. Weseman

Attachment

NOTE : The proposed final policy statement proposes changes to agency policy and has been published to gather public comment. Until the statement becomes final the interim policy statement will prevail for transfer of interstate maintenance program funds.

NOTE: The proposed final policy statement proposes changes to agency policy and has been published to gather public comment. Until the statement becomes final the interim policy statement will prevail for transfer of interstate maintenance program funds.

Federal Highway Administration

[FHWA Docket No. 93-10]

Transfer of Interstate Maintenance Program Funds

AGENCY: Federal Highway Administration (FHWA), DOT.

ACTION: Proposed final policy statement; requests for comments.

SUMMARY: This proposed final policy statement sets forth the FHWA's policy for addressing the interstate maintenance program funds transfer provisions of 23 U.S.C. 119(f)(1). The criteria for determining what constitutes adequate maintenance, which are included in this policy, are associated with only the transfer of Interstate Maintenance (IM) funds and are not related to the State's responsibility to properly maintain projects constructed with Federal-aid funds outlined in 23 U.S.C. 116, Maintenance.

DATES: Comments must be received on or before November 1, 1994.

ADDRESSES: Submit written, signed comments concerning this policy statement to FHWA Docket No. 93-10, Federal Highway Administration, Room 4232, HCC-10, Office of the Chief Counsel, 400 Seventh Street, SW., Washington, DC 20590. All comments received will be available for examination at the above address between 8:30 a.m. and 3:30 p.m., e.t., Monday through Friday, except Federal holidays.

FOR FURTHER INFORMATION CONTACT: Mr. John Hallin, Chief, Pavement Design and Rehabilitation Branch, (202) 366-1323, or Ms. Vivian Philbin, Attorney-Advisor, Office of Chief Counsel, General Law Branch, (202) 366-0780, Federal Highway Administration, 400 Seventh Street SW., Washington, DC 20590.

SUPPLEMENTARY INFORMATION:

Background

On March 3, 1993, the FHWA published an interim policy statement on the transfer of Interstate maintenance program funds at 58 FR 12299, and provided a 60-day public comment period which closed on May 3, 1993. During the intervening period, FHWA has evaluated the comments and reconsidered its initial position. As a result, the FHWA is proposing to modify the pavement roughness and faulting criteria and to add additional criteria that were not proposed in the interim policy.

A total of 18 State highway agencies (SHAs) and the Highway User Federation for Safety and Mobility (HUFSAM), a public interest group, provided written comments to the docket established for the interim policy statement.

The SHA comments ranged from administrative type questions, such as requests for clarification of measurement procedures and use of existing pavement management system data, to fundamental positions on the individual indicators and the specific established criteria. Some SHAs endorsed various portions of the criteria established, while others took exception to part or all of the criteria.

The HUFSAM strongly endorsed the interim policy. It stressed the need to assure that the Interstate System be maintained at a very high level and noted that, from its studies, nationwide, the Interstate maintenance funding levels are inadequate.

After evaluating the comments received, the FHWA continues to believe that transfers of apportioned IM funds specifically earmarked for Interstate maintenance to other designated programs should be permitted only when the Interstate System routes are in a physical, operational, and safe condition and perform at or near the level for which they were designed, and constructed. Because pavement and bridge activities constitute the major cost items of IM eligible activities, the interim policy focused on pavement and bridge condition indicators as the determining factors for eligibility to transfer IM funds. Other essential elements, necessary to maintain the physical and operational integrity of the Interstate, must also be considered in transportation decisions. Responses to the interim policy, however, indicate a concern that other essential elements need not be considered in transfer decisions. This was not the intent of the interim policy statement.

Section 101(a) of Title 23 U.S.C. defines "maintenance" to mean the preservation of the entire highway, including surface, shoulders, roadside, structures, and such traffic control devices as are necessary for its safe and efficient utilization. As the IM program now provides the major resources for rehabilitation, resurfacing, and restoration (3R) work on the Interstate System, extending the service life of all major components and enhancing highway safety on the system should receive first priority for IM fund use. For example, over 25 percent of the projects and approximately 10 percent of funds from the IM program are currently being expended on traffic and safety improvement projects. The FHWA supports a continued strong emphasis on safety.

In a sampling of SHA pavement management systems conducted during the past year, the FHWA found that the pavement condition indicators established in the interim policy are generally collected and used by the States in evaluating the condition of the Interstate for their own management purposes. While the data collection and reporting procedures differ somewhat, the fundamental indicators are consistently used by the SHA's to manage their Interstate pavements.

The proposed final policy includes the original pavement and bridge condition indicators established in the interim policy and adds pavement surface friction as a fourth pavement condition indicator. However, the roughness criteria has been modified and the separate faulting criteria for jointed plain and joint reinforced concrete pavement (JPCP and JRCP) has been replaced with a single criterion of 525 mm/km (33 inches/mile) for both jointed pavement types.

In addition to these interim factors, this proposed final policy statement adds criteria for the additional traffic and safety related indicators of (1) safety appurtenances, (2) traffic control devices, and (3) geometric elements. These indicators are equally critical to the Interstate System which relies heavily on the availability of IM funds for continued adequacy. Maintenance of the Interstate System's operational as well as physical characteristics in a satisfactory manner remains the first priority for the use of these funds.

Comments Received

This section addresses specific SHA comments organized around the criteria established for each of the individual condition indicators.

Pavement Roughness

Three SHAs suggested that the International Roughness Index (IRI), developed at the International Road Roughness Experiment, is not the appropriate measure of rideability. The FHWA recognizes that IRI does have some limitations. It does, however, provide a common quantitative basis with which to reference the different measures of roughness. Further, it is currently collected by SHAs and provided to FHWA under the Highway Performance Monitoring System (HPMS) submission requirements. Although the FHWA is open to use of improved pavement surface rideability measures, until such time that improved measures and equipment to measure them are accepted and readily available to SHA's, the FHWA will continue to rely on IRI as the ride indicator.

Four SHAs commented that the specific IRI criteria of 240 cm/km (150 inches/mile) was too severe. The FHWA disagrees. The selection of the 240 cm/km upper limit criteria on pavement roughness was directly tied to the FHWA's desire to require Interstate pavement to be in fair or better condition. The interim policy noted that initial IRI to pavement serviceability rating [1] (PSR) conversion studies [2] indicated a 240 cm/km IRI is equivalent to a PSR range of 3.0 to 3.5. Pavements within this range are classified as fair in the FHWA's "1992 Highway Statistics" [3] report. Subsequent additional analysis of the IRI/PSR correlation indicates that a 240 cm/km IRI more accurately reflects a much lower PSR range of 2.5 to 2.8 (pavements in this range are classified as being in poor to mediocre condition [4]). Based on this further analysis, the FHWA has established an upper limit of allowable IRI of 200 cm/km (127"/mile). This converts to a PSR of between 2.8 and 3.2 which is more consistent with the FHWA's original objective that pavements be in fair or better condition [5].

Rutting

Rutting comments were limited to data collection difficulties and reflected a degree of uncertainty about what data collection equipment and procedure would be considered acceptable. No comments were received concerning the appropriateness of the rutting indicator or the established criteria. Therefore the FHWA has retained 15 mm (5/8 inch) as the upper allowable limit of rutting. Concerns related to data collection equipment and procedures are addressed under "Pavement Data Collection," later in the preamble.

Faulting

The SHA comments on the faulting criteria were split evenly: five SHAs thought that the faulting criteria were too restrictive, while five SHAs commented that the criteria were acceptable. In addition, the HUFSAM found the criteria acceptable.

One SHA recommended simplifying the policy by replacing the separate faulting criteria for jointed plain and jointed reinforced concrete pavement (JPCP and JRCP) with a single faulting criterion in mm/km (inches/mile) for both pavement types. A mm/km based criteria would eliminate the need to take joint frequency into account, as the average allowable faulting per joint would be directly related to the number of joints/mile. The FHWA recognizes the merit in this recommendation and has replaced the separate faulting criteria of 3 mm on JPCP and 6 mm on JRCP with an equivalent maximum faulting rate of 525 mm/km (33 inches/mile) for both. This faulting rate is equivalent to 3 mm per joint on typical JPCP with 6 meter (20 foot) joint spacing and 6 mm per joint on JRCP with 12 meter (40 foot) joint spacing. Because joint spacing varies between States, the allowable faulting per joint will differ from State to State, even though the faulting rate per km remains constant.

Administrative—Procedural Tolerance Limits

The most common comment, received from seven SHAs, was that the scope of the application of the criteria was too stringent. The crux of the argument was that some tolerance limit should be established to allow a SHA in substantial compliance to transfer funds. A common suggestion was that the FHWA only require that 90 to 95 percent of the Interstate System meet the criteria before allowing transfer.

The FHWA recognizes that there are continually evolving pavement and bridge needs and, at any one point in time, even SHAs with exceptionally good pavements might not meet the criteria on 100 percent of their Interstate system. The FHWA has already provided relief for this situation. The interim policy specifically allows transfer when all criteria are not met on the Interstate if the work necessary to correct any deficient segments is included in the approved State Transportation Improvement Program, required by 23 U.S.C. 135(f). This relief is included in the final policy. The FHWA believes that allowing a 5 to 10 percent exemption or tolerance would be unwise, as it would allow transfer of money necessary to maintain the Interstate highway system.

[1] The PSR concept was developed at the 1958 American Association of State Highway Officials (AASHO) road test to relate the pavement serviceability index (PSI), computed from objectively measured pavement distress, with subjective serviceability ratings by panels of road users.

[2] Bashar Al-Omari and Michael I. Darter, "Relationships between IRI and PSR: A Report of the Findings of Pavement Model Enhancements for the Highway Performance Monitoring System (HPMS)," Transportation Engineering Series No. 69, University of Illinois at Urbana Champaign, Report No. UILU-ENG-92-2013, September 1992. This document is available for inspection in FHWA Docket No. 93-10.

[3] FHWA, "Highway Statistics 1992," FHWA-PL-93-023. A copy of this document is available for inspection in FHWA Docket No. 93-10.

[4] Ibid.

[5] Ibid.

Pavement Data Collection

Several SHAs posed comments and questions on data collection and reporting procedures. The primary concern appeared to be whether FHWA would require a specific data collection effort using some standardized equipment and procedures that would be different from what is currently used by the individual SHAs. Further, the comments included request for flexibility in summarizing the data. Several suggested that FHWA should use whatever SHA PMS data was available to determine the acceptability of a certification accompanying a transfer request.

The FHWA intends to rely primarily on current surface roughness, rutting, and faulting information contained in SHAs PMS database(s) and from information reported in HPMS in evaluating the pavement component of State certifications accompanying Interstate maintenance fund transfer requests.

The FHWA recognizes the uniqueness of each SHA's PMS and the diversity of equipment and procedures used by the SHAs to meet their particular pavement management needs. The FHWA is not prescribing new specific uniform data collection equipment, procedures, sampling, or data reduction techniques to determine compliance with the pavement Interstate maintenance transfer criteria.

Bridges

Only two SHA's commented on the bridge section of the policy. Both endorsed the use of the current National Bridge Inventory (NBI) bridge deck condition rating (Item 58) as an indicator and supported the criteria requirement that bridge decks have a condition rating of 5 or better. This is consistent with the long standing use of a deck rating of less than 5 to determine a structurally deficient bridge.

Both States also recommended that FHWA include the NBI ratings for superstructure and substructure in the policy and delete the load posting requirement contained in the interim policy.

The FHWA originally considered using superstructure and substructure ratings as specific criteria when it initially developed the interim policy. Upon further consideration, FHWA still supports "load posting" criterion which reflects superstructure and substructure condition ratings and is also a measure of potential safety concern.

The need for load posting is an end result of applying superstructure and substructure conditions, along with other factors, in making load carrying capacity calculations. Changes in condition ratings, and therefore, the load posting, are affected by a reduced maintenance effort which eventually leads to continual and long-term deterioration of bridge elements.

One of the SHAs further recommended that the FHWA incorporate failure susceptibility as an indicator. Failure susceptibility is not required nor normally assessed by States in the course of inspecting bridges to meet national bridge inspection standards. As a result, the FHWA believes it would be inappropriate to use failure susceptibility as a nationwide criterion in the IM fund transfer policy, and has not included it.

Finally, one SHA recommended that bridge railing adequacy should be included in the decision factors. The FHWA considered including bridge railing adequacy as indicated by NBI Item 36 in the early development of policy criteria. The NBI Item 36 is a four segment item that rates bridge railings for adequate impact strength, and approach guardrail for adequate vehicle safety and protection.

The adequacy of bridge railings and approach guardrail is a serious safety concern and should be considered in the States' maintenance program as well as in developing highway safety projects.

Bridge Data Collection

The NBI ratings are determined in accordance with the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges" (Coding Guide) U.S. DOT/FHWA, December 1988.

Policy

For the purpose of 23 U.S.C. 119(f)(1), which provides for transfer of State apportioned IM funds that are in excess of a State's need to the State's NHS and STP apportionment, the FHWA will accept a State's certification if the State's Interstate routes meet the following criteria:

Pavement:

(1) An IRI of 200 cm per km (127 inches per mile) or less;

(2) Rutting of 15 mm (5/8 inch) or less on flexible pavements;

(3) Cumulative faulting of 525 mm per km (33 inches/mile) or less on jointed rigid pavements; and

(4) Surfaces have adequate surface friction and drainage, based on the State accidents record system not identifying any locations with a high incidence of wet weather accidents.

Bridges:

(1) Bridge decks in "fair condition" or better (Coding Guide item 58 rated 5 or better); and

(2) No load posting required (Coding Guide item 70 rated 5).

Safety Appurtenances:

Guardrail, bridge rails, safety barriers, and other safety features including the upstream ends of all traffic barriers meet (a) the performance criteria of 23 CFR 625, (b) acceptable use warrants, and (c) installation requirements per State standard plans.

Traffic Control Devices:

All major guide, regulatory, and warning signs meet the minimum size, shape, color, format, and message requirements as well as the day and night legibility and visibility requirements of the MUTCD and amendments.

Geometric Elements:

(1) The horizontal and vertical alignment, and widths of median, traveled way, and shoulders meet the AASHTO Interstate Standards, as incorporated in 23 CFR 625, in effect either at the time of original construction, major reconstruction, or inclusion into the Interstate system, whichever was the latest; and

(2) Hazardous features (fixed objects, steep sideslopes, etc.) within the clear zone are either eliminated, corrected, or adequately shielded.

In the event that the condition, as reflected by current databases, does not meet the required criteria, for any segment of Interstate, the State's request for funding transfer may not be approved unless the State certifies that the deficient segments have either been subsequently upgraded to meet the required criteria or that the work necessary to correct any such deficient segments is included in the approved State Transportation Improvement Program, required by 23 U.S.C. 135(f).

Section 119(f)(2) of Title 23, U.S.C., allows the States to transfer up to 20 percent of the apportioned IM funds to the NHS and STP apportionment based solely on the request of the States.

(23 U.S.C. 119 and 315; 49 CFR 1.48(b))

Issued on: August 29, 1994.

Rodney E. Slater,

Federal Highway Administrator.

[FR Doc. 94-21737 Filed 9-1-94; 8:45 am]

BILLING CODE 4910-22-P

NOTE: The proposed final policy statement proposes changes to agency policy and has been published to gather public comment. Until the statement becomes final the interim policy statement will prevail for transfer of interstate maintenance program funds.

Federal Highway Administration

[FHWA Docket No. 93-10]

Transfer of Interstate Maintenance Program Funds

AGENCY: Federal Highway Administration (FHEA), DOT.

ACTION: Interim policy statement.

SUMMARY: This interim policy statement establishes the FHWA's policy for addressing the interstate maintenance program funds transfer provisions of section 119(f)(1) of title 23, United States Code (U.S.C.), which was amended by Section 1009 of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991. By publishing this interim policy statement the FHWA seeks to advise States of the criteria the agency will use in evaluating a State's request to transfer interstate maintenance funds, while providing the opportunity for public comment prior to issuing a final policy statement.

DATES: Comments must be received on or before May 3, 1993.

ADDRESSES: Submit written, signed comments concerning this policy statement to FHWA Docket No. 93-10, Federal Highway Administration, room 4232, HCC-10, Office of the Chief Counsel, 400 Seventh Street, SW., Washington, DC 20590. All comments received will be available for examination at the above address between 8:30 a.m. and 3:30 p.m. e.t., Monday through Friday, except legal Federal holidays.

FOR FURTHER INFORMATION CONTACT: Mr. Louis Papet, Chief, Pavement Division, (202) 366-1324, or Mrs. Vivian Philbin, Attorney Advisor, Office of Chief Counsel, General Law Branch, (202) 366-0780, Federal Highway Administration, 400 Seventh Street SW., Washington DC 20590.

SUPPLEMENTARY INFORMATION:

Background

Section 1009 of the ISTEA amended 23 U.S.C. 119 by replacing "Interstate System resurfacing" with the "Interstate maintenance program" (IM) Public Law 102-240, section 1009, 105 Stat. 1914, 1933. Section 1009 also established additional constraints affecting the States' options for transferring a portion of these funds to the States' apportionments for other Federal-aid programs.

Section 119(f)(1), as amended, allows the transfer of IM funds to other Federal-aid highway programs provided the State certifies to the Secretary that: (1) Any part of the IM funds are in excess of the needs of the State for resurfacing, restoring, or rehabilitating Interstate System routes and (2) that it is adequately maintaining the Interstate System, and the Secretary accepts such certification. Notwithstanding section 119(f)(1), section 119((f)(2), as amended, allows the States to "unconditionally" transfer up to 20 percent of unobligated IM apportioned funds based solely on the request of the States.

Further, section 1009(c)(2) of the ISTEA requires the Secretary to develop and make available to the States criteria for determining what constitutes adequate maintenance of the Interstate System for the purposes of section 119(f)(1) of title 23, United States Code. The criteria for determining what constitutes adequate maintenance, which are included in this policy, are associated with only the transfer of IM funds and are not related to the State's responsibility to properly maintain projects constructed with Federal-aid funds outlined in 23 U.S.C. 116, Maintenance.

In developing the specific criteria, the FHWA believes that transfers of apportioned IM funds specifically earmarked for Interstate maintenance to other designated programs should only be allowed when the Interstate System routes are in a physical condition to perform at or near the level for which they were designed and intended.

Pavement and bridge activities constitute the majority of IM eligible activities. The FHWA has focused on pavement and bridge condition indicators as determining factors for eligibility to transfer IM funds.

The FHWA has selected Interstate pavement condition indicators (surface roughness, rutting, and faulting) and bridge condition indicators (bridge deck condition and the need for load posting) for evaluating State's requests to transfer IM funds under the provisions of 23 U.S.C. 119(f)(1). These indicators are collected and used by the States in evaluating the condition of the Interstate for their own management purposes. They are generally incorporated into State pavement and bridge management systems and the national bridge inventory and highway performance monitoring system.

Pavement Condition Indicators

Roughness

The FHWA will use the International Roughness Index (IRI) to evaluate roadway roughness, and has set an upper IRI limit of 240 cm per km (150 inches per mile) for surface roughness.

The IRI was developed at the International Road Roughness Experiment sponsored by the World Bank and several countries, including the United States, in Brazil in 1982. It is designed to provide a common quantitative basis with which to reference the different measures of roughness. It summarizes the longitudinal surface profile in the wheel track and simulates the response of one wheel of a typical passenger car traveling 80 km per hour (50 miles per hour) to road roughness.

The IRI upper limit of 240 cm per km, selected by the FHWA, is based on consideration of research efforts that relate actual roadways with a known IRI with the public's perception of ride quality. A recent study [1] conducted for the FHWA indicated that objectively developed IRI numbers could be mathematically correlated with subjectively developed pavement serviceability ratings [2] (PSR) generated by panels of road users. This work included mathematical formulas that allow conversions between IRI readings and anticipated road user evaluation of pavement performance (*i.e.*, PSR).

Conversion formulas [3] indicate that an IRI of 240 cm per km correlates to a PSR range of between 3.0 and 3.5, which is slightly greater than the 2.5 to 3.0 PSR range associated with terminal serviceability for Interstate highway pavements.[4]

[1] Bashar Al-Omari and Michael I. Darter, "Relationships between IRI and PSR: A Report of the Findings of Pavement Model Enhancements for the Highway Performance Monitoring System (HPMS)," Transportation Engineering Series No. 89, University of Illinois at Urbana Champaign, Report No. UILU-ENG-92-2013, September 1992. This document is available for inspection in FHWA Docket No. 93-10.

[2] The PSR concept was developed at the 1956 American Association of State highway Officials (AASHO) road test to relate the pavement serviceability index (PSI), computed from objectively measured pavement distress, with subjective serviceability ratings by panels of road users.

[3] Includes conversion formulas developed inhouse by the State of Maine, for the South Carolina pavement management system by PMS Inc. and the previously mentioned Al-Omari and Darter research cited in footnote No. 1.

[4] The "AASHTO Guide for Design of Pavement Structures", AASHTO, 1986 (page I-8) defines terminal serviceability index as the lowest acceptable level before resurfacing or reconstruction becomes necessary for the particular class of highway. The AASHTO Guide goes on to note that

Continued

Rutting

The FHWA has established 15 mm (5/8 inch) as the upper allowable limit of rutting.

The American Association of State Highway and Transportation Officials (AASHTO) Highway Subcommittee on Construction surveyed State highway agencies in 1988 on rutting. The survey revealed that for State maintained roads, 1/2 inch rutting would initiate rehabilitation in about 35 percent of the States. An additional 35 percent of the States indicated that 3/4 inch of rutting would initiate rehabilitation. The "Highway Pavement Distress Identification Manual" (HPDIM)[3] classifies 1/2 to 1 inch of rutting as moderate severity.

The FHWA 15 mm (5/8 inch) criterion is consistent with the performance levels expected on the Interstate System.

Faulting

The FHWA has established two levels of faulting criteria that are related to pavement type. The FHWA has established an upper limit on faulting of 3 mm (1/8 inch) on jointed plain concrete pavements (JPCP), and an upper limit on faulting of 6 mm (1/4 inch) on jointed reinforced concrete pavements (JRCP).

Generally, State highway agencies consider faulting to be objectionable in the 1/8 to 1/2 inch range. The HPDIM classifies faulting between 1/16 and 1/2 inch as moderate severity. The "Pavement and Shoulder Maintenance Performance Guides," August 1984, FHWA publication number TS–84–208, indicates faulting should be repaired at 1/4 inch. A copy of TS–84–208 is available for inspection in FHWA Docket No. 93–10.

The FHWA selected a lower level of faulting for JPCP than for JRCP because JPCP joints occur more frequently. The levels selected are consistent with the higher expectation the traveling public associates with Interstate highways.

Pavement Data

Procedures for developing IRI are currently well defined in the guidance provided in the "Highway Performance Monitoring System (HPMS) Field Manual," Appendix J "Roughness Equipment, Calibration and Data Collection." This document is widely available in planning sections of State highway agencies and the FHWA division offices and a copy of this publication is available for inspection in FHWA Docket No. 93–10. IRI data are collected annually and reported to the FHWA under the HPMS program.

The FHWA pavement policy, (23 CFR part 626) requires each State to have an operational pavement management system (PMS) for principal arterials (which includes the Interstate system) in place by January 13, 1993.

The FHWA envisions that the States will assemble necessary pavement surface roughness, rutting, and faulting information from data currently available in the States' PMS database(s) and from information reported in HPMS.

The FHWA division offices will work with the States in identifying acceptable procedures for measuring and compiling the data available from the States' PMS. Data supporting each State's IM transfer request will be made available for inspection by the FHWA.

Bridge Condition Indicators

The FHWA will use the current national bridge inventory (NBI) bridge deck condition rating (item 58) and the rating indicating whether the bridge requires load posting (item 70) as indicators of Interstate bridge condition for purposes of evaluating States' requests for IM transfer. The NBI ratings are determined in accordance with the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges" (Coding Guide) US DOT/FHWA, December 1988. A copy of this publication is available for inspection in FHWA Docket No. 93–10.

Bridge Decks

The FHWA will require that bridge decks have a condition rating (item 58) of 5 or better.

Bridge decks are rated in item 58 on a scale of 0 to 9 with a rating of 9 representing a bridge deck in excellent condition. A Coding Guide deck rating of less than 5 indicates a poor condition with the deck showing deterioration and spalling. In relation to pavement roughness, a deck with a rating less than 5 is considered a rough deck that would not provide a reasonably smooth ride. A deck rating of less than 5 is a long-standing condition rating used to determine a structurally deficient bridge.

Posting

The FHWA will require that NBI item 70, for load posting, must be a rating of 5.

The National Bridge Inspection Standards (23 CFR part 650, subpart C) require the posting of load limits only if the maximum legal load in a State produces stresses in excess of the operating stress levels. The operating stress level will result from the absolute maximum permissible load to which a bridge may be subjected. Coding Guide item 70 of the NBI is the item for bridge posting, and a State's rating of 5 indicates that no posting is required at the operating level.

Load posting of a bridge reduces the level of service of the system of which the bridge is an integral part and can potentially disrupt interstate and intrastate commerce. Heavy vehicles may be required to take long detour routes thereby indirectly adding to the costs the public must bear for goods and services. Load posting of a bridge may also be an indicator of a bridge's superstructure or substructure capacity that may have been affected by continual and long term deterioration of the bridge's elements and which could have been prevented or abated by adequate preventive maintenance.

Policy

For the purpose of 23 U.S.C. 119(f)(1), which provides for transfer of IM funds apportioned to the States, the FHWA will accept a State's certification if the State's Interstate routes meet the following criteria:

Pavement

(1) An IRI of 240 cm per km (150 inches per mile) or less;

(2) Rutting of 15 mm (5/8 inch) or less; and

(3) Faulting of 3 mm [illegible] inch) or less on JPCP and 6 mm (1/4 [illegible]) or less on JRCP.

Bridges

(1) Bridge decks in [illegible] dition" or better (Coding Guide it[illegible] ated 5 or better); and

(2) No load posting re[illegible] Coding Guide item 70 rated 5).

In the event that the co[illegible]tion, as reflected by current condition data bases, for any segment of Interstate pavement or bridge does not meet the required criteria, the State's request for funding transfer may later be approved only if the State certifies that the deficient segments have been subsequently upgraded to meet the required criteria or that the work necessary to correct any such deficient segments is included in the approved State Transportation Improvement Program, required by 23 U.S.C. 135(f).

Section 119(f)(2) of title 23 U.S.C. allows the States to "unconditionally" transfer up to 20 percent of unobligated IM apportioned funds based solely on the request of the States.

Authority: 23 U.S.C. 119 and 315; 49 CFR 1.48(b).

Issued on: February 24, 1993.

E. Dean Carlson,

Executive Director, Federal Highway Administration.

[FR Doc. 93–4809 Filed 3–2–93; 8:45 am]

BILLING CODE 4910–22–M

a terminal serviceability index of 2.5 to 3.0 is often suggested for use in the design of major highways. A copy of this publication is available for inspection in FHWA Docket No. 93–10.

[3] The "Highway Pavement Distress Identification Manual", US DOT/FHWA, DOT–FH–11–9175/NCHRP 1–19, March, 1979 reprinted February 1986. This Publication is available for inspection in FHWA Docket No. 93–10.

CHAPTER 2

PAVEMENT ISSUES

2.1 Reserved.

2.2 Reserved.

2.3 Tire Pressure, Technical Paper 89-001, February 15, 1989.

2.4 Reserved.

2.5 A Discussion of Discount Rates for Economic Analysis of Pavements, February 1990.

2.6 Resilient Modulus Testing Equipment, February 24, 1988.

2.7 Longitudinal Joint Construction and Edge Drop-Offs, March 1989.

2.8 Reserved.

2.9 Reserved.

2.10 Life Cycle Cost Analysis, September 15, 1992.
- Interim Policy Statement - FR, July 11, 1994.

2.11 Reserved

2.12 ISTEA Implementation Interstate Maintenance Program, Memorandum, May 21, 1992.

2.13 Preventive Maintenance, July 27, 1992.
- Information on Interstate Maintenance Program, June 14, 1993.

2.14 Computer Software
- McTran's Software, July 1995.

U.S. Department
of Transportation

Federal Highway Administration

Memorandum

Subject: Technical Paper 89-001 - Tire Pressure

Date: FEB 15 1989

From: Chief, Pavement Division
Washington, D.C. 20590

Reply to Attn of: HHO-11

To: Regional Federal Highway Administrators
Direct Federal Program Administrator

The effects of increased tire pressure on pavement performance has been a topic of considerable discussion during the past several years. The attached "Technical Paper" is issued to provide FHWA field engineers with the latest information on trends in tire pressures, and tire pressure effects on pavement performance. The paper's format consists of a series of questions and answers to the most commonly asked questions on this subject.

Two copies of the paper are attached for your use and handling. Please make distribution to the division offices following your normal distribution process.

Additional information on the effects of tire pressure on pavement performance may be obtained by contacting the Pavement Division, Pavement Management Branch at FTS 366-1337.

for Louis M. Papet

Technical Paper 89-001 - Tire Pressure

The purpose of this technical paper is to provide pavement specialists with the latest information regarding trends in tire pressures and the effects of increased tire pressure on pavement performance. The format consists of questions and answers to four of the most commonly asked questions by pavement engineers.

1. Has there been an increase in tire inflation pressure since the AASHO Road Test?

   Cold inflation pressures for tires used at the AASHO Road Test conducted between 1958 and 1960, were 75 psi for the 7.5 to 11-inch tires and 80 psi for the 12-inch bias ply tires. The tire pressures are recommended cold inflation pressures for specified wheel loads, ranging from about 3000 pounds for the 7.5-inch tires to about 6800 pounds for a 12.00x24-inch tire. Hot tire pressures were typically 9 to 20 psi higher than the cold pressures and averaged 11 psi higher. Hot tire pressures for the heavier wheel loads would therefore have averaged about 85 to 90 psi although pressures from 23 to 130 psi were reported. It was noted at the Road Test that tire pressures increased gradually with truck operation but stabilized after 90 minutes.

   A 1987 nationwide tire pressure survey cooperatively done by the FHWA and State motor carrier safety organizations, showed that 81 percent of the 5040 hot tire pressures measured fell in the range of 85 to 115 psi. While the size and type of tire was not recorded in this survey, it is estimated that 70 percent of the tires were radials. Contact with tire manufacturers indicate that 75 percent of all truck tires now sold are steel-belted radials with recommended cold inflation pressures of 95 to 100 psi.

   Tire pressure surveys done in Canada, Florida, Illinois, Kentucky, New Mexico, and Oregon, in 1985 and 1986, involving more than 4000 trucks showed average hot radial tire pressures of between 96 and 107 psi.

   It should be noted that radial tire pressures are approximately 5 psi higher than bias ply tire pressures for the same wheel load. While radial tires flex more during operation, less heat is generated due to their radial ply construction. To compensate for the lower operating temperature of radial tires, manufacturers recommend higher inflation pressure which reduces tire deflection, and equalizes the footprint between radials and bias ply tires.

   The Wisconsin Department of Transportation did a detailed survey of 6780 truck tires in 1987 that showed 93 percent of hot tire pressures fell between 85 and 115 psi. The average hot tire pressure was about 100 psi. Wisconsin found that 12 percent of tractor semitrailer tires were overinflated and 14 percent were underinflated. In the national survey sponsored by FHWA, 11 percent were overinflated and 19 percent were underinflated. Over- and underinflation were defined as plus or minus

10 psi with respect to the tire manufacturer's recommended inflation pressure. From this information, it does not appear that truck drivers are intentionally overinflating their tires.

The most common tire size reported in the Wisconsin survey was an 11-inch tire on a 24.5-inch rim followed by an 11-inch tire on a 22.5-inch rim.

The Tire and Rim Association publishes recommended tire sizes and maximum cold inflation pressures for various tire loads. A comparison of recommended tire pressures for the years 1930, 1969, and 1985 show only modest increases in inflation pressure for given loads. For example, the recommended tire pressure for a 10-inch tire to carry 4000 pounds is 65 psi for all 3 years. The recommended tire pressure for a 11-inch tire to carry 5000 pounds was 70 psi in 1930, and 80 psi in 1969 and 1985. For a 6000-pound load on a 12-inch tire, the recommended inflation pressure was 80 psi in 1930 and 90 psi in 1969 and 1985.

The 1974 Highway Act raised the single axle maximum weight limit from 18,000 pounds to 20,000 pounds, the tandem axle weight limit from 32,000 pounds to 34,000 pounds, and established a maximum gross weight limit of 80,000 pounds. These increases in allowable weight limits have resulted in gradually increasing truck weights and 18,000-pound equivalent single axle loads. For example, on the rural Interstate System, the average number of equivalent single axle loads has been increasing about 7 percent per year, between 1970 and 1985. A recent study by Texas A&M University entitled "Improved Prediction of EAL" suggests that this increase is due to use of larger trucks rather than an increase in truck loads.

In conclusion, tire pressures for given load rated tires have not changed much over the last 50 years. Due to the increase in load being carried and the use of radial tires, fleet tire pressures have increased about 10 to 20 psi when compared to tire pressures at the AASHO Road Test.

2. Does an increase in tire pressure accelerate pavement deterioration?

There are six recent studies that suggest flexible pavement deterioration is accelerated by increased tire pressure. This is especially true for thin pavements, i.e., AC surface course 1 to 3 inches in thickness. The six studies were done at the Universities of Kentucky, Munich, Texas, and Waterloo (Canada), and at the Massachusetts Institute of Technology and Texas A&M University. These studies are summarized in NCHRP 1-25, "Effects of Heavy Vehicle Characteristics on Pavement Response and Performance." Brief extracts from the six studies are as follows:

Kentucky: A distress model was developed to predict loads to fatigue failure. It was determined that the load equivalency factor increased rapidly with increasing tire contact pressure and decreasing pavement thickness. Damage at 120 psi was 5.5 times greater than at 75 psi for thin AC pavements.

Munich: Rutting rate versus tire pressure for single and dual tires was studied. Rut depth doubled when tire pressure was increased from 100 to 130 psi.

Texas: Strain increased significantly in 1-2" AC pavements as tire pressure was increased from 75 to 110 psi. The increase in tire pressure resulted in a 25 percent decrease in pavement life.

Waterloo: An increase in tire pressure from 60 to 120 psi increased strain in the AC surface course and top of the subgrade causing fatigue cracking. A seven fold increase in rut formation was predicted for the same increase in tire pressure.

MIT: Damage at 125 psi is more than two times greater than the damage at 75 psi for 1-3" AC pavements. The time to rutting failure was reduced by 30 percent, and surface rutting increased 300 percent for a tire pressure increase from 75 to 125 psi.

Texas A&M: There was a 50 percent decrease in fatigue life of a 2-inch AC pavement over 8 inches of aggregate base when the tire pressure was increased from 75 to 125 psi. The higher tire pressure substantially increased the rate of fatigue cracking in the thin AC surface.

It should be emphasized that the above results are for thin AC pavements (1-3 inches of AC) and are based on computer models and not on field surveys.

At the FHWA's accelerated loading facility (ALF) in Virginia, the effects of tire pressure were evaluated on a flexible pavement consisting of 2 inches of asphalt concrete wearing course, 5 inches of asphalt concrete binder course and 12 inches of crushed aggregate base course constructed on an AASHTO A-4 subgrade. Tire pressures of 76 and 140 psi and both radial and bias ply tires were used in the study. A second variable in the study was dual wheel load set at 9400 and 19,000 pounds. Surface deflection and strain and tensile strain at the bottom of the AC layer were measured.

Results show that doubling the load from 9400 pounds to 19,000 pounds on this thick pavement section increased predicted pavement damage by 1000 percent, while doubling the tire pressure increased predicted damage by only 20 percent. It was quite obvious that for this pavement section increasing wheel load affected the pavement considerably more than increasing tire pressure. Predicted pavement damage was in terms of fatigue equivalency factor developed using an exponential relationship between the number of cycles to failure and the magnitude of the tensile strain at the bottom of the asphalt layer.

While it is safe to say that wheel loads affect the pavement considerably more than tire pressures, care should be exercised in making judgments about the effect of load and tire pressures on real trafficked pavement sections. The ALF does not duplicate actual truck wheel loads in that ALF does not have a suspension system equivalent to a truck suspension system. Loads are also not applied to the pavement in the same manner as under actual highway conditions.

One other finding of interest related to tire pressure is from an Australian study as summarized in NCHRP 1-25. It was found that dynamic load induced by the drive wheels of a tractor semitrailer truck decreased with an increase in tire pressure. Tire pressure did not, however, affect dynamic load induced by the trailer wheels. This finding is contrary to a study done by the Massachusetts Institute of Technology which found that an increase in tire pressure from 75 to 120 psi increased the dynamic load coefficient from 0.12 to 0.14. Dynamic load coefficient is defined as the standard deviation of the dynamic load divided by the mean dynamic load.

3. What efforts have been made to assess tire pressure trends and quantify the impacts on flexible pavements?

The most significant effort to assess tire pressure trends and to define the extent of the tire pressure problem, was a 1-day symposium held in the spring of 1987 in Austin, Texas. The symposium which was sponsored by AASHTO and the FHWA was attended by 70 individuals representing the highway, tire, and trucking industries. Questions used to guide the discussion included: Has there been an increase in tire pressure since the AASHO Road Test? Has increased tire pressure accelerated pavement damage? Is legislation needed to regulate new tire and truck designs? Is it time to accelerate our efforts to improve mix design? Should load equivalency factors be increased?

Findings from the symposium are reflected in the answers to our first two questions. It was generally agreed that the introduction of legislation was not an appropriate course of action at this time to regulate new tire and truck designs. Such legislation would also be very difficult and costly to monitor and enforce. A legislated solution to the effects of tire pressure should be a last resort approach.

Those in attendance thought that efforts to improve mix design should be accelerated and that load equivalency factors should be increased. It was felt that better communication between segments of the transportation industry and more research are needed to define the relationship between vehicle characteristics and pavement deterioration.

The Second North American Conference on Managing Pavements was held in Toronto, Canada November 2-6, 1987. At this conference papers were presented on all aspects of pavement management including impacts of trucks. Mr. Jack Friedenrich, Chairman of the AASHTO Task Force on High Pressure Truck Tires, presented a paper summarizing the Task Force's work to date. In addition to discussing trends in truck tire design and inflation pressure, Mr. Friedenrich outlined the pavement problems resulting from higher tire pressures and discussed an approach and needed research to solve these problems.

Tire pressure and related pavement problems are not solely those of increased rate of pavement deterioration. The AASHTO pavement design equations are based on a set of assumptions including the assumption that today's tire pressures are the same as those at the Road Test. Because we have shown that today's tire pressures are higher and because other assumptions about environment, roadbed soils, vehicle characteristics, etc. may have not been met, the design and analysis of pavement structures using AASHTO procedures are being questioned. The basic AASHTO design equations are also being questioned due to changes in allowable axle loads, suspension systems, wheel configurations, and axle spacing since the Road Test.

Finding a solution to the problem of increased rate of pavement deterioration is not easy because of the number of factors that contribute to the problem. For this reason, the solution must involve all aspects of the complex relationship between vehicle and pavement. Models that incorporate all vehicle characteristics need to be developed and verified so that accurate dynamic loads applied to the pavement can be determined. Pavement models that predict the number of load applications to a particular type of pavement failure also need to be developed and verified. When we can confidently predict the effects of changes in vehicle characteristics and pavement design, we can begin making those decisions which will give us the longest pavement life for the least cost.

The solution to the problem also involves a continuing dialogue among those building the pavements, those using our highways, and tire and truck manufacturers. Toward that end, Mr. Friedenrich suggested future symposiums like the one held in Austin, Texas.

There were two tire pressure-related papers presented at the 1988 Transportation Research Board annual meeting in Washington, D.C. The first paper is entitled: "Effect of Load, Tire Pressure, and Type on Flexible Pavement Response" by Messrs. Ray Bonaquist, Charles Churilla, and Ms. Deborah Freund of the Federal Highway Administration. The paper presents the findings of work with the FHWA's accelerated loading facility for the first two pavement sections tested. These findings are included in the answer to the second question.

The second paper is entitled: "Evaluation of Increased Pavement Loading and Tire Pressure" by Stuart Hudson and Stephen Seeds of Austin Research Engineers, Inc. (ARE). The authors summarize work done for the Arizona Department of Transportation to develop computer programs to calculate 18,000-pound equivalent single axle loads (ESAL's) from both loadometer and weigh-in-motion data. The programs have the capability of using either AASHTO load equivalency factors or factors developed by ARE. The ARE load equivalency factors take into consideration tire pressure, pavement structure, truck classification, wheel configuration, and axle configuration. Based on the results of a survey of 350 trucks in Arizona, the authors believe that tire pressure should be included in flexible pavement design. The survey showed that the average hot tire pressure for the tires on the truck steering axle to be 106 psi, and 102 psi for the tires on the drive and trailer axles. The values are about 20 percent higher than the 85 to 90 psi hot tire pressures measured at the AASHO Road Test. Using a fatigue damage model developed by Mr. Fred Finn and a pavement section composed of 3 inches of AC over 6 inches of aggregate base and 8 inches of aggregate subbase, the authors concluded that an increase in tire pressure from 90 psi to 121 psi would reduce pavement life by 38 percent. Ninety percent of the tire pressures measured in the Arizona survey fell in the range of 90 to 121 psi. The shortened pavement life due to increased tire pressure is the reason the authors believe that load equivalency factors used to design pavement structures should consider tire pressure. The tire pressure adjusted load equivalency factor would equal the loads to a particular type of pavement failure or amount of damage for a standard wheel load and tire pressure divided by the number of loads to the same pavement failure or amount of damage for a given wheel load and tire pressure.

4. What can be done to make flexible pavements more resistant to tire pressure-related damage?

Tire pressure is crucial in determining stresses near the surface of a flexible pavement. High tire pressures, thus, necessitate high-quality materials in the upper layers of the flexible pavement. Asphalt overlays of concrete pavements may also be highly impacted by increased contact pressures.

The same preliminary design, mix design, construction, and maintenance practices that have been used to make flexible pavements more resistant to rutting, stripping, and cracking can also be applied to ensuring AC pavements are resistant to higher tire pressures. These practices can be found in FHWA Technical Advisory T5040.27, "Asphalt Concrete Mix Design and Field Control," March 10, 1988, and in such study reports and manuals as the following:

"Asphalt Pavement Rutting Western States," Western Association of State Highway and Transportation Officials, original report, May 1984, and followup report, February 1988.

"Asphalt Pavement Rutting and Stripping Report," FHWA Ad Hoc Task Force, August 14, 1987.

"Hot-Mix Bituminous Paving Manual," FHWA, May 1984.

Some important factors brought out by this paper and by the referenced materials which may help prevent failures under high tire pressure conditions are as follows:

(1) Use the FHWA Traffic Monitoring Guide and weigh-in-motion (WIM) equipment to obtain accurate design axle loadings.

(2) Follow the recommendations found in the FHWA Technical Advisory T5040.27 "Asphalt Concrete Mix Design and Field Control" with particular emphasis on the suggested eight steps to be followed in pavement rehabilitation design.

(3) Stripping is often a primary cause of rutting which is aggravated by high tire pressures. Be sure to consider an anti-strip additive when high volumes of trucks are anticipated.

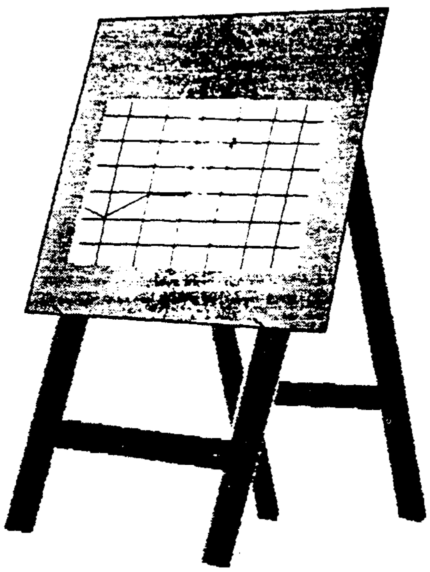
(4) Use the 0.45 power gradation chart to select the proper aggregate gradation for optimum mix density, stability, and voids.

(5) Follow recommended good engineering procedures pertaining to drainage, site control, choice of asphalt, choice of aggregate, mix design, base and subbase design, plant operation, construction practices, quality control, and maintenance.

DRAFT

A DISCUSSION OF

DISCOUNT RATES FOR ECONOMIC ANALYSIS OF PAVEMENTS



by

Peter Z. Kleskovic

Highway Engineer

FHWA

Pavement Division

Feburary 1990

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A DISCUSSION OF DISCOUNT RATES FOR ECONOMIC ANALYSIS OF PAVEMENTS

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GENERAL

In a life cycle cost analysis, a discount rate is needed to compare costs occurring at different points in time. The discount rate reduces the impact of future costs on the analysis, reflecting the fact that money has a time value. In the private sector, money that is not spent today can be invested to earn some rate of return. In the public works sector, where needs usually exceed the available funds, savings from one project can be used to build another project. This results in additional benefits to the public.

The factors that determine interest rates for bonds include inflation, risk, liquidity and tax liabilities. Removing these factors should result in a real interest rate that represents the true time value of money. In the engineering economics literature, this rate is known as the discount rate.

There continues to be discussion about what rate to use when evaluating alternative pavement strategies. In a 1987 survey, State Highway Agencies used rates ranging from 0 to 9 percent. Of the 27 responses, the median discount rate was 4 percent, which was used by 26 percent of the responding States. In total, 59 percent of the responding States used a discount rate in the range of 3 to 5 percent, with 19 and 22 percent either below or above this range, respectively.

The discount rate can affect the outcome of a life cycle cost analysis in that certain alternatives may be favored by higher or lower rates. High rates favor alternatives that stretch out costs over a period of time, since the future costs are discounted in relation to the initial cost. A low rate hurts these alternatives since future costs are added in at almost face value. In the case of a discount rate equal to 0, all costs are treated equally regardless of when they occur. Where alternative strategies have similar maintenance, rehabilitation and operating costs, the discount rate will have a minor effect on the analysis and initial costs will have a larger effect.

This paper documents a review and analysis of economic data in order to determine an appropriate discount rate to use in economic analyses of pavements. Interest and inflation data was assembled for the period of 1950 to 1987. Discount rates were then computed by subtracting the inflation rates for each year from the corresponding interest rates. Most of the interest and inflation data was obtained from the Economic Report of the President, February 1988. The Producer Price Index data came from the Handbook of Cyclical Indicators. The Federal-aid Price Index data came from the 1st Quarter 1974 and the 3rd Quarter 1987 "Price Trends for Federal-Aid Highway Construction."

INTEREST RATES

Table 1 presents six interest rates between 1950 and 1987. They are:

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YEAR	3 YEAR TREASURY BONDS	BANK PRIME RATE	Aaa CORP BONDS	Baa CORP BONDS	HIGH GRADE MUNICIPAL BONDS	FEDERAL FUNDS RATE.
1950		2.1	2.6	3.2	2.0	
51		2.6	2.9	3.4	2.0	
52		3.0	3.0	3.5	2.2	
53	2.5	3.2	3.2	3.7	2.7	
54	1.6	3.1	2.9	3.5	2.4	
55	2.5	3.2	3.1	3.5	2.5	1.8
56	3.2	3.8	3.4	3.9	2.9	2.7
57	4.0	4.2	3.9	4.7	3.6	3.1
58	2.8	3.8	3.8	4.7	3.6	1.6
59	4.5	4.5	4.4	5.1	4.0	3.3
1960	4.0	4.8	4.4	5.2	3.7	3.2
61	3.5	4.5	4.4	5.1	3.5	2.0
62	3.5	4.5	4.3	5.0	3.2	2.7
63	3.7	4.5	4.3	4.9	3.2	3.2
64	4.0	4.5	4.4	4.8	3.2	3.5
65	4.2	4.5	4.5	4.9	3.3	4.1
66	5.2	5.6	5.1	5.7	3.8	5.1
67	5.0	5.6	5.5	6.2	4.0	4.2
68	5.7	6.3	6.2	6.9	4.5	5.7
69	7.0	8.0	7.0	7.8	5.8	8.2
1970	7.3	7.9	8.0	9.1	6.5	7.2
71	5.7	5.7	7.4	8.6	5.7	4.7
72	5.7	5.3	7.2	8.2	5.3	4.4
73	7.0	8.0	7.4	8.2	5.2	8.7
74	7.8	10.8	8.6	9.5	6.1	10.5
75	7.5	7.9	8.8	10.6	6.9	5.8
76	6.8	6.8	8.4	9.8	6.5	5.0
77	6.7	6.8	8.0	9.0	5.6	5.5
78	8.3	9.1	8.7	9.5	5.9	7.9
79	9.7	12.7	9.6	10.7	6.4	11.2
1980	11.6	15.3	11.9	13.7	8.5	13.4
81	14.4	18.9	14.2	16.0	11.2	16.4
82	12.9	14.9	13.8	16.1	11.6	12.3
83	10.5	10.8	12.0	13.6	9.5	9.1
84	11.9	12.0	12.7	14.2	10.2	10.2
85	9.6	9.9	11.4	12.7	9.2	8.1
86	7.1	8.3	9.0	10.4	7.4	6.8
87	7.7	8.2	9.4	10.6	7.7	6.7
1950-87	6.4[a]	7.0	6.8	7.8	5.3	6.3[a]
1950-59	3.0[a]	3.4	3.3	3.9	2.8	2.5[a]
1960-69	4.6	5.3	5.0	5.7	3.8	4.2
1970-79	7.2	8.1	8.2	9.3	6.0	7.1
1980-87	10.7	12.3	11.8	13.4	9.4	10.4

a. Average is for period of available data.

Table 1. Interest Rates (1950 - 1987)

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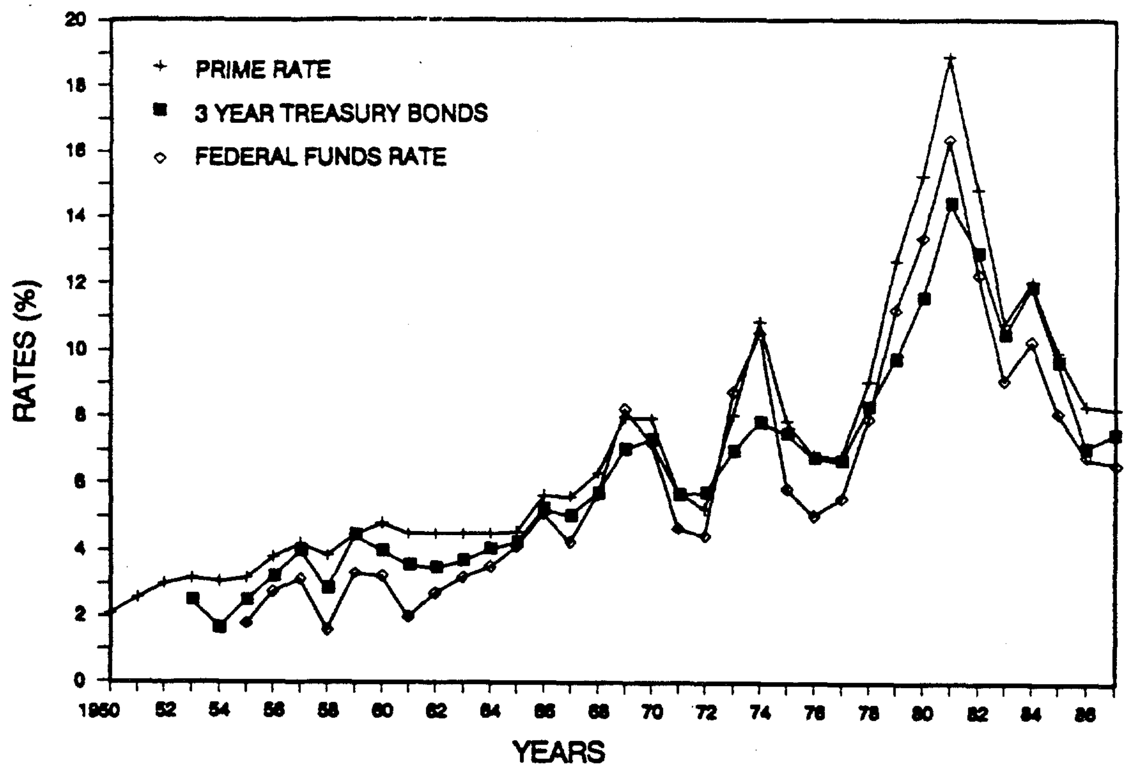

Figure 1. Three Interest Rates

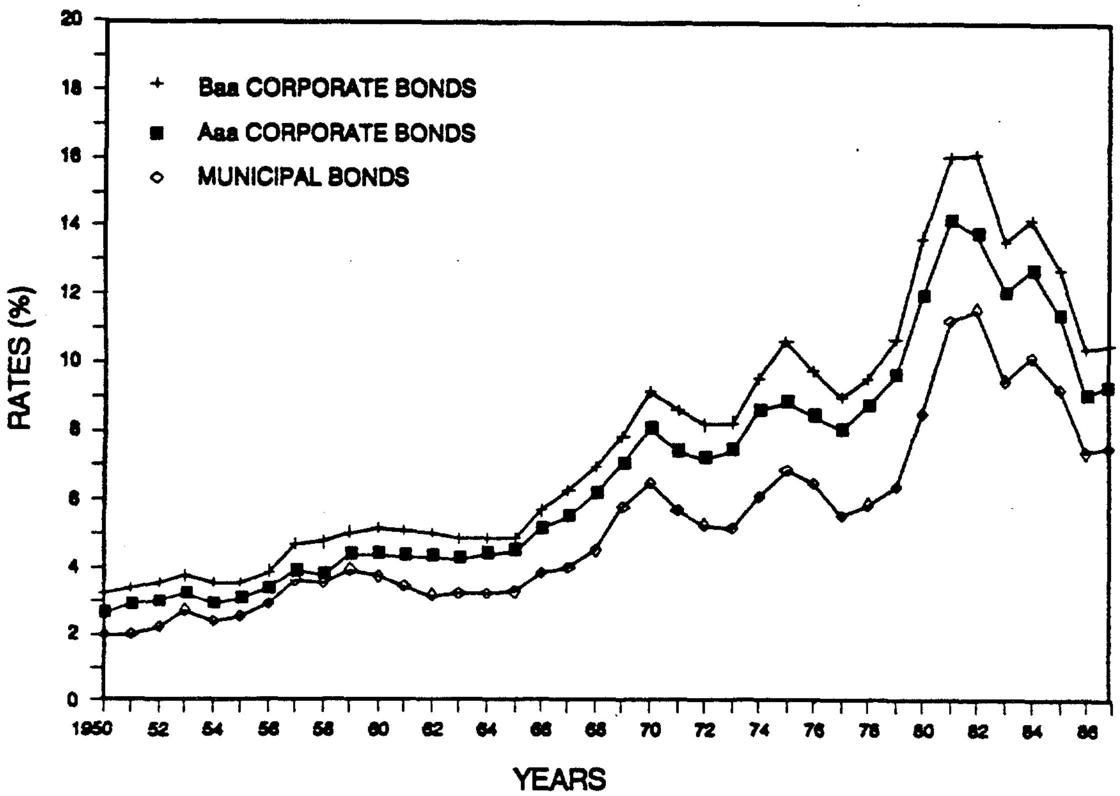

Figure 2. Three Interest Rates

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Treasury Bonds (3 Years): Securities backed by the taxing power of the US Government and exempt from State and local taxes.

Bank Prime Rate: The rate banks charge their most credit worthy customers for short term loans.

Aaa Corporate Bonds (Moody's): Highest graded bonds.

Baa Corporate Bonds (Moody's): Lower medium graded bonds.

High Grade Municipal Bonds (S&P): Bonds of states, cities, or counties. They are often exempt from federal, state and local taxes.

Federal Funds Rate: The interest rate on overnight loans between banks.

These six rates are plotted on Figures 1 and 2. For clarity, only three rates are shown on each figure. Although, the individual rates vary, all follow a similar pattern. Of the six interest rates, the Baa corporate bond usually was the highest. The fairly consistent difference between Baa and Aaa bonds is a measure of the higher risk that Baa bonds carry. Treasury and municipal bond rates are usually lower than the two corporate bonds or the prime rate, again because of their lower risk. Municipal bonds usually have lower rates than Treasury bonds, because of their generally tax exempt status.

Figure 3 is a plot of the high, average, and low value of the six rates for each year. In 1950, these rates ranged from 2 to 3.2 percent. They rose at a slow rate till about 1965, when the range was from 3.3 to 4.9. Since then,

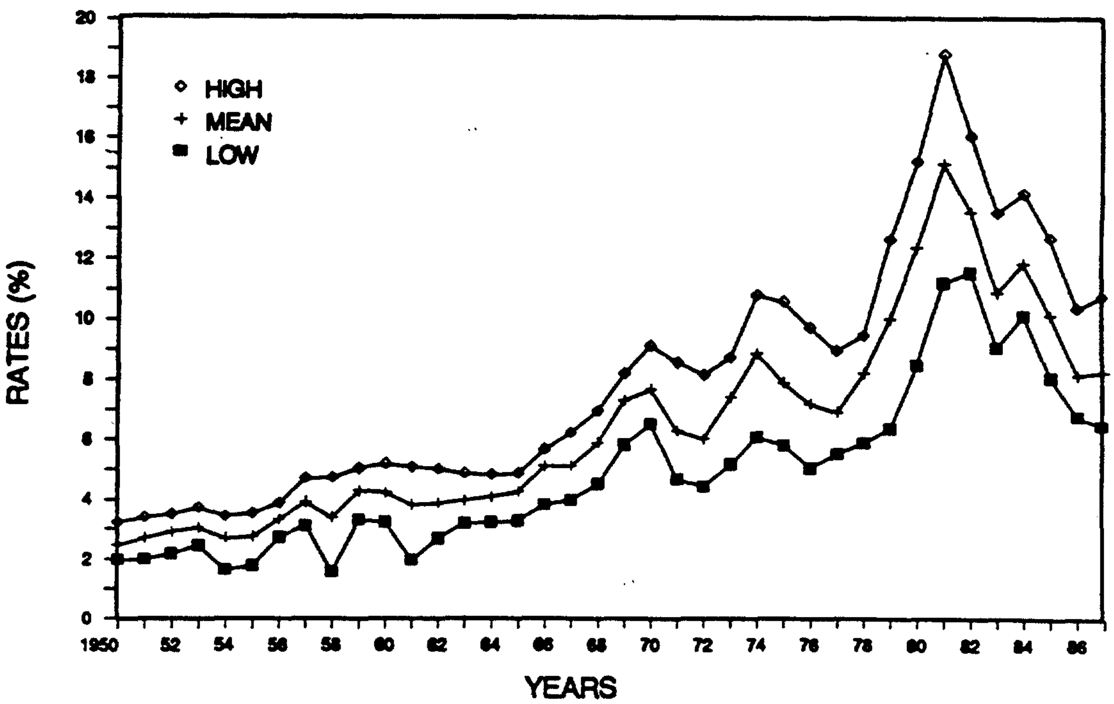

Figure 3. High/Low Range of Six Interest Rates

rates have peaked three times, 1970, 1974 and 1981. In 1981, rates reached their highest peak in recent times, ranging from 11.2 to 18.9 percent, and averaging 15.2 percent. These rates have dropped since this peak, with a range of 6.7 to 10.6 percent and an average value of 8.4 percent in 1987.

INFLATION RATES

Table 2 presents inflation rates as measured by the year to year rate of change in four indexes. They are:

Implicit G.N.P. Price Deflator: Index of average price level of all final goods and service. Used to convert current-dollar GNP to constant-dollar GNP.

Composite Index (FHWA): Index composed of six indicator items including excavation, pavement surfaces and structural elements. These are reported in the "Price Trends for Federal-aid Highway Construction."

Consumer Price Index: Measure of the average level of prices over time in a fixed market basket of goods and services.

Producer Price Index (all commodities): Measures average changes in prices received by commodity producers.

Figure 4 plots the yearly rate of change for three of the inflation indexes from 1950 to 1987, the Implicit G.N.P. Price Deflator, the Consumer Price Index, and the Producer Price Index. The three inflation rates show similar trends to the interest rate curves. During most of the 1950's and early 1960's, inflation rates were low, generally below 4 percent. Inflation started to rise during the mid 1960's and into the early 1970's. Major increases in inflation occurred in 1973-1974 and during the late 1970's. Inflation rates have generally fallen since 1980. The Producer Price Index is the most volatile of the three rates on Figure 4, generally having either the lowest or the highest yearly rates of change.

The rate of change in the Composite Index from the "Price Trend for Federal-aid Highway Construction" is shown on Figure 5. Although, this index tracks the other inflation indexes it fluctuates over a much wider range.

DISCOUNT RATES

Discount rates were computed from the above data by subtracting the inflation rates from the interest rates for each year. Since data was assembled for six different interest rates and four inflation rates, there are twenty-four possible discount rates that could be considered. To make this effort more manageable, only nine discount rates were computed using the following combinations of interest and inflation rates:

Interest Rates	Inflation Rates
Treasury Bonds (3 Years)	Implicit G.N.P. Deflator
Aaa Corporate Bonds	Consumer Price Index
Municipal Bonds	Composite Index (FHWA)

YEAR	IMPLICIT GNP PRICE DEFLATOR	COMPOSITE INDEX (FHWA)	CONSUMER PRICE INDEX	PRODUCER PRICE INDEX COMMODITIES
1950	2.0		1.0	3.9
51	4.8	22.8	7.9	11.4
52	1.5	2.8	2.2	-2.7
53	1.6	-3.7	0.8	-1.4
54	1.6	-5.7	0.5	0.2
55	3.2	-2.8	-0.4	0.2
56	3.4	13.1	1.5	3.3
57	3.6	4.4	3.6	2.9
58	2.1	-2.4	2.7	1.4
59	2.4	-4.2	0.8	0.2
60	1.6	-2.3	1.6	0.1
61	1.0	0.8	1.0	-0.4
62	2.2	3.8	1.1	0.3
63	1.6	2.3	1.2	-0.3
64	1.5	0.8	1.3	-0.2
65	2.7	3.7	1.7	2.0
66	3.6	6.5	2.9	3.3
67	2.6	4.1	2.9	0.2
68	5.0	3.5	4.2	2.5
69	5.6	8.2	5.4	3.9
70	5.5	12.2	5.9	3.7
71	5.7	4.8	4.3	3.3
72	4.7	5.1	3.3	4.5
73	6.5	10.3	6.2	13.1
74	9.1	36.0	11.0	18.9
75	9.8	0.4	9.1	9.2
76	6.4	-3.4	5.8	4.6
77	6.7	7.1	6.5	6.1
78	7.3	19.4	7.7	7.8
79	8.9	19.4	11.3	12.6
80	9.0	14.3	13.5	14.1
81	9.7	-3.9	10.4	9.2
82	6.4	-6.3	6.1	2.0
83	3.9	-0.2	3.2	
84	3.7	5.8	4.3	
85	3.2	11.0	3.6	
86	2.6	-0.3	1.9	
87	3.0		3.7	
1950-87	4.4	5.2 [a]	4.3	4.2 [a]
1950-59	2.6	2.7 [a]	2.1	1.9
1960-69	2.7	3.1	2.3	1.1
1970-79	7.1	11.1	7.1	8.4
1980-87	5.2	2.9 [a]	5.8	8.4 [a]

a. Average is for period of available data.

Table 2. Yearly Inflation Rates (1950-1987)

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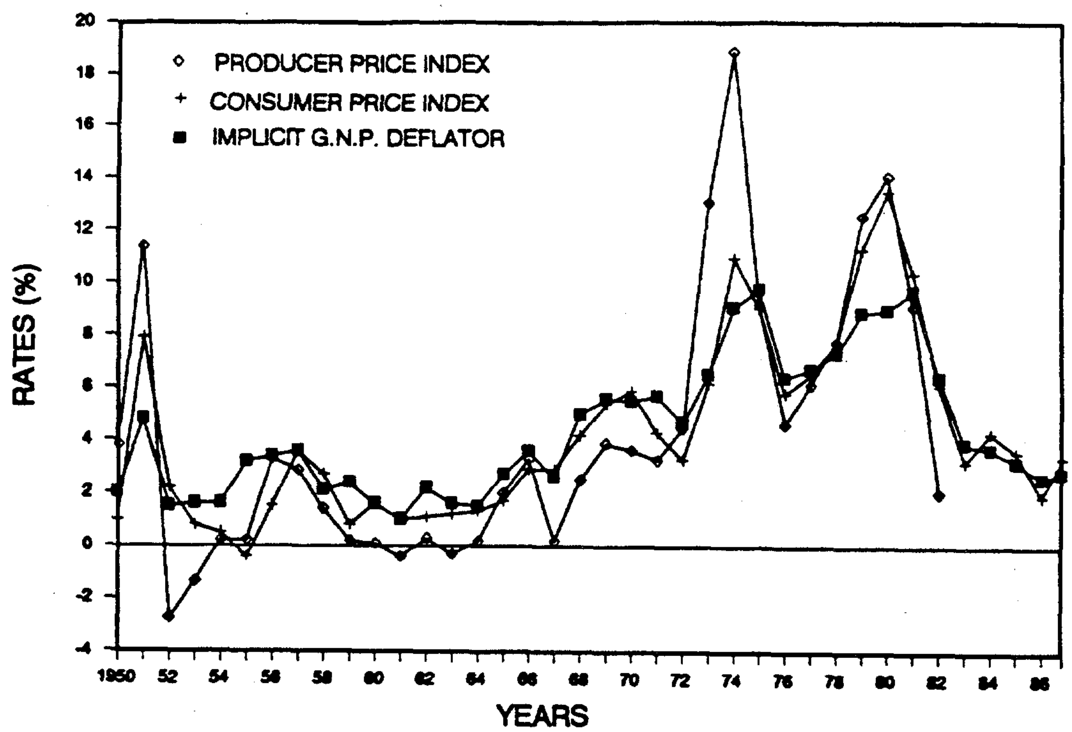

Figure 4. Year to Year Rate of Change in Three Inflation Indexes.

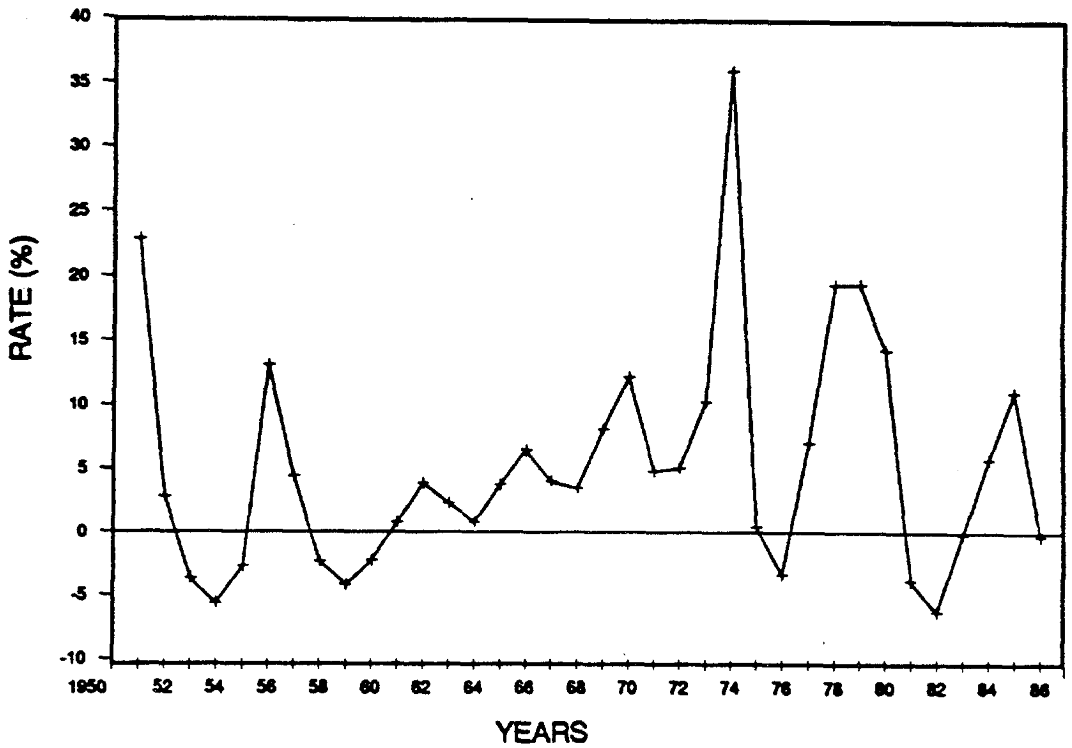

Figure 5. Year to Year Rate of Change in Composite Index (FHWA)

It is recognized that the discount rates computed from these sets of data may not be the most theoretically appropriate for an analysis of pavements. The purpose was to determine if discount rates varied over time or if they held fairly constant.

It was expected that the different discount rates would generally follow parallel paths over time, but that individual curves would be higher or lower based on other factors, such as, risk. The three interest and three inflation rates that were used in the analysis were chosen because they were less volatile than the other rates. The exception was the Composite Index (FHWA) which is highly volatile, but was used in the analysis because of its relationship to the highway program.

The nine discount rates are plotted in Figures 6, 7 and 8. The six curves in Figures 6 and 7 followed similar tracks with time. The curves on Figure 8, which are based on the Composite Index (FHWA), follow similar paths but with highly exaggerated movements. The inflation component of the discount rates on Figure 8 completely overpower the effect of the different interest rates.

Figures 6 and 7 show that the discount rate is not fixed over time. The 1960's are the only period in recent times where discount rates remained fairly constant and relatively low. During the 1970's, discount rates were very unsteady due to the surges in inflation that occurred during the middle

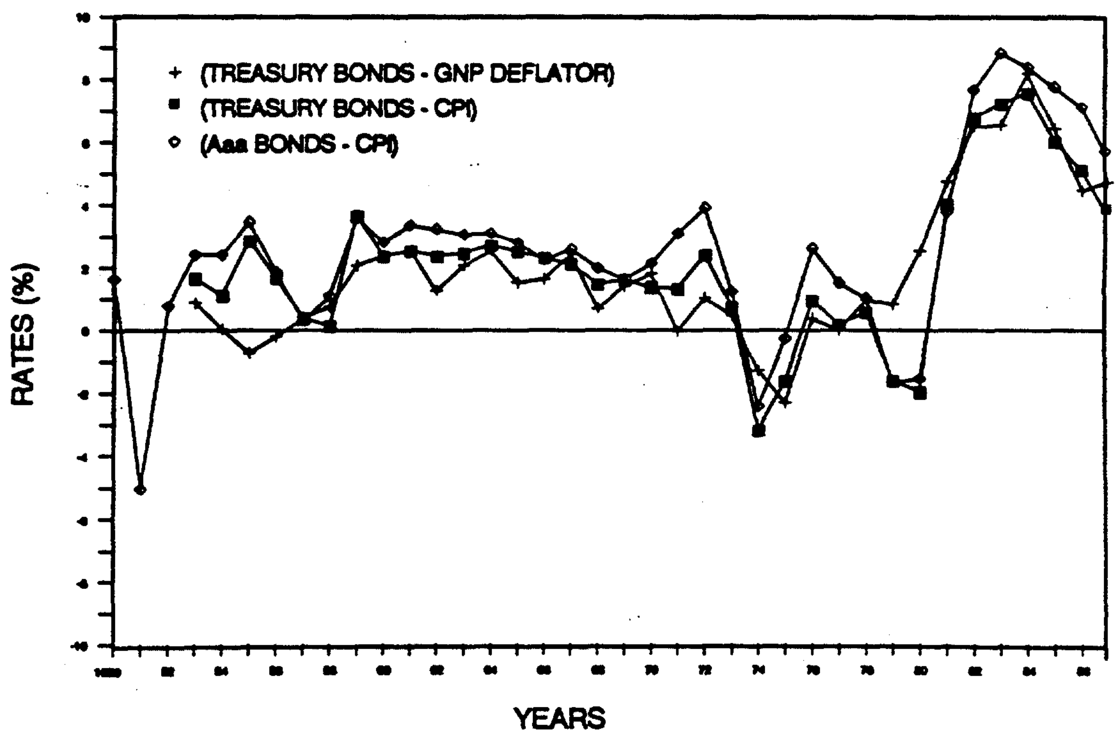

Figure 6. Discount Rates

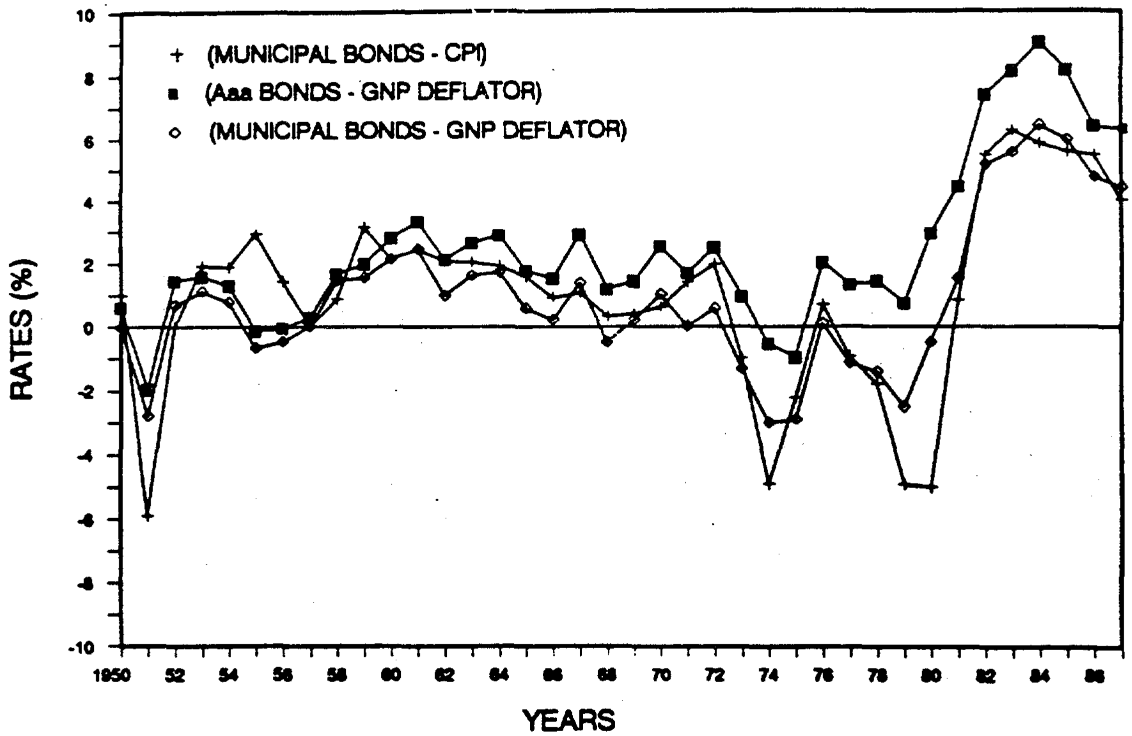

Figure 7. Discount Rates

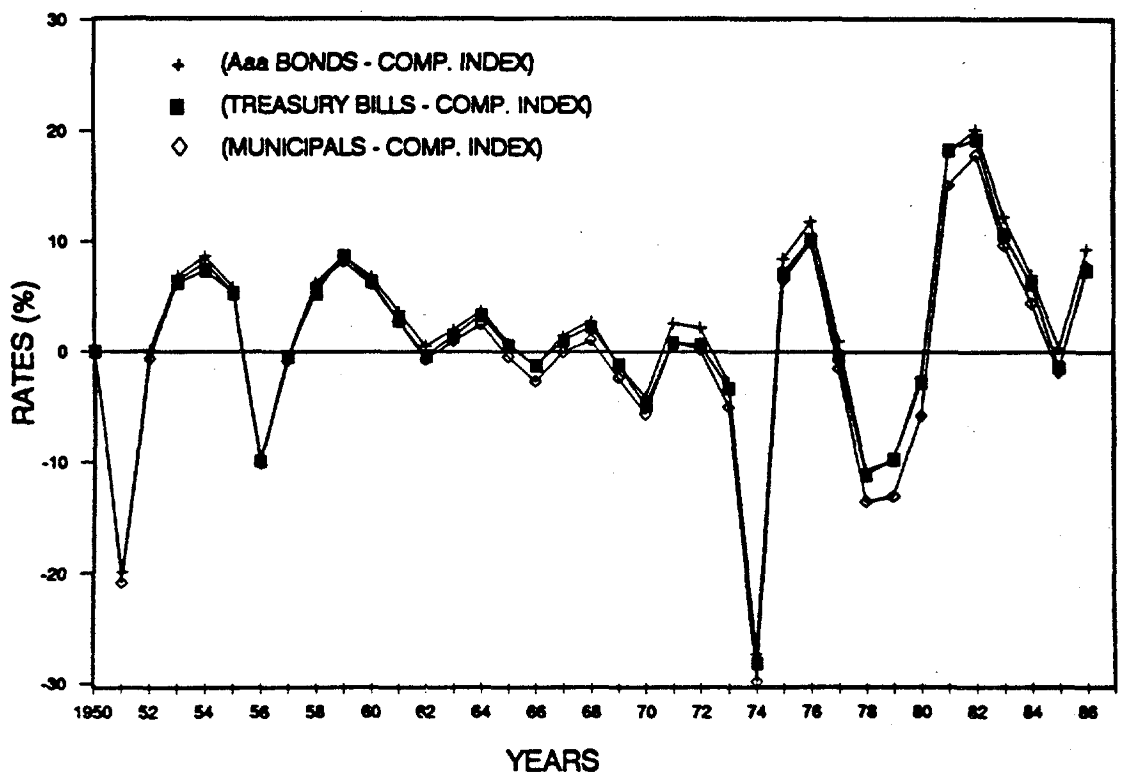

Figure 8. Discount rates

and end of the decade. During these inflationary surges, the economy was subjected to negative discount rates. During the 1980's, discount rates have been unusually high due to relatively low inflation rates and high interest rates.

The long term trend of relatively low discount rates with higher rates more recently is further shown in Table 3. From 1953 to 1987, the average value for each of the 6 computed discount rates fell in a range from 1.0 percent to 2.8 percent. However, from 1980 to 1987, they ranged between 3.5 percent and 6.6 percent. A frequency distribution for each decade from 1950 to 1987 is shown on Figure 9. These distributions indicate that during most of this period, a low discount rate would be appropriate, on the order of 1 to 2 percent. During, the 1980's, a higher rate of about 6 percent would appear to be appropriate. However, it is important to note that discount rates have generally declined from about 1983-1984. In 1987, the six discount rates fell in a range of 4.0 to 6.4 percent and they averaged 4.9 percent.

Conclusions and Recommendations

The question arises as to what is the appropriate discount rate to use in an economic analysis of pavements. The following points are offered for use in adopting a particular value:

1. The difference between interest rates and inflation rates does not remain constant over time. Therefore, it is not possible to identify a unique discount rate which will always be correct. As shown on Figures 6 and 7, there were very drastic changes in discount rates during the late 1970's and the early 1980's. It is clear that the selection of an appropriate rate should not be based on unusual economic conditions which may occur for a relatively short period of time.

2. Over the long run, discount rates have been relatively low, on the order of 1 to 2 percent. During the early and mid 1980's, these rates have been in a range of 5 to 6 percent. They have been declining from 1983-1984.

3. Future interest and inflation rates cannot be reliably predicted over a long period of time, such as 30 years. Whether discount rates will return to their long term range of 1 to 2 percent or whether they will remain relatively high is unknown. Conditions in the US economy may lead to continued higher discount rates for the near future.

4. Since we cannot accurately forecast discount rates for long periods of time, a conservative approach would be to adopt a value somewhere between the high and the low range. A reasonable value might be in the range of 3 to 5 percent. It is perhaps on this basis that a discount rate of 4 percent is commonly used in pavement life cycle cost analyses. Such a range recognizes that discount rates of 7 or 8 percent have been relatively rare in this country and have lasted for only a short period of time. Additionally, we have had high discount rates for almost a decade. It is probably unrealistic to assume that they will return in the short run to a range of 1 to 2 percent.

5. Once a discount rate has been selected, Agencies may wish to conduct a

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YEAR	TREASURIES MINUS CPI	TREASURIES MINUS GNP DEFLATOR	Aaa BONDS MINUS CPI	Aaa BONDS MINUS GNP DEFLATOR	MUNICIPALS MINUS CPI	MUNICIPALS MINUS GNP DEFLATOR
1950			1.6	0.6	1.0	0.0
51			-5.0	-1.9	-5.9	-2.8
52			0.8	1.5	0.0	0.7
53	1.7	0.9	2.4	1.6	1.9	1.1
54	1.1	0.0	2.4	1.3	1.9	0.8
55	2.9	-0.7	3.5	-0.1	2.9	-0.7
56	1.7	-0.2	1.9	0.0	1.4	-0.5
57	0.4	0.4	0.3	0.3	0.0	0.0
58	0.1	0.7	1.1	1.7	0.9	1.5
59	3.7	2.1	3.6	2.0	3.2	1.6
60	2.4	2.4	2.8	2.8	2.1	2.1
61	2.5	2.5	3.4	3.4	2.5	2.5
62	2.4	1.3	3.2	2.1	2.1	1.0
63	2.5	2.1	3.1	2.7	2.0	1.6
64	2.7	2.5	3.1	2.9	1.9	1.7
65	2.5	1.5	2.8	1.8	1.6	0.6
66	2.3	1.6	2.2	1.5	0.9	0.2
67	2.1	2.4	2.6	2.9	1.1	1.4
68	1.5	0.7	2.0	1.2	0.3	-0.5
69	1.6	1.4	1.6	1.4	0.4	0.2
70	1.4	1.8	2.1	2.5	0.6	1.0
71	1.4	0.0	3.1	1.7	1.4	0.0
72	2.4	1.0	3.9	2.5	2.0	0.6
73	0.8	0.5	1.2	0.9	-1.0	-1.3
74	-3.2	-1.3	-2.4	-0.5	-4.9	-3.0
75	-1.6	-2.3	-0.3	-1.0	-2.2	-2.9
76	1.0	0.4	2.6	2.0	0.7	0.1
77	0.2	0.0	1.5	1.3	-0.9	-1.1
78	0.6	1.0	1.0	1.4	-1.8	-1.4
79	-1.6	0.8	-1.7	0.7	-4.9	-2.5
80	-1.9	2.6	-1.6	2.9	-5.0	-0.5
81	4.0	4.7	3.8	4.5	0.8	1.5
82	6.8	6.5	7.7	7.4	5.5	5.2
83	7.3	6.6	8.8	8.1	6.3	5.6
84	7.6	8.2	8.4	9.0	5.9	6.5
85	6.0	6.4	7.8	8.2	5.6	6.0
86	5.2	4.5	7.1	6.4	5.5	4.8
87	4.0	4.7	5.7	6.4	4.0	4.7
1953-87	2.1	1.8	2.8	2.6	1.2	1.0
1953-59	1.6	0.4	2.2	1.0	1.7	0.5
1960-69	2.3	1.8	2.7	2.3	1.5	1.1
1970-79	0.1	0.2	1.1	1.2	-1.1	-1.1
1980-87	5.0	5.6	6.0	6.6	3.5	4.1

Table 3. Discount Rates

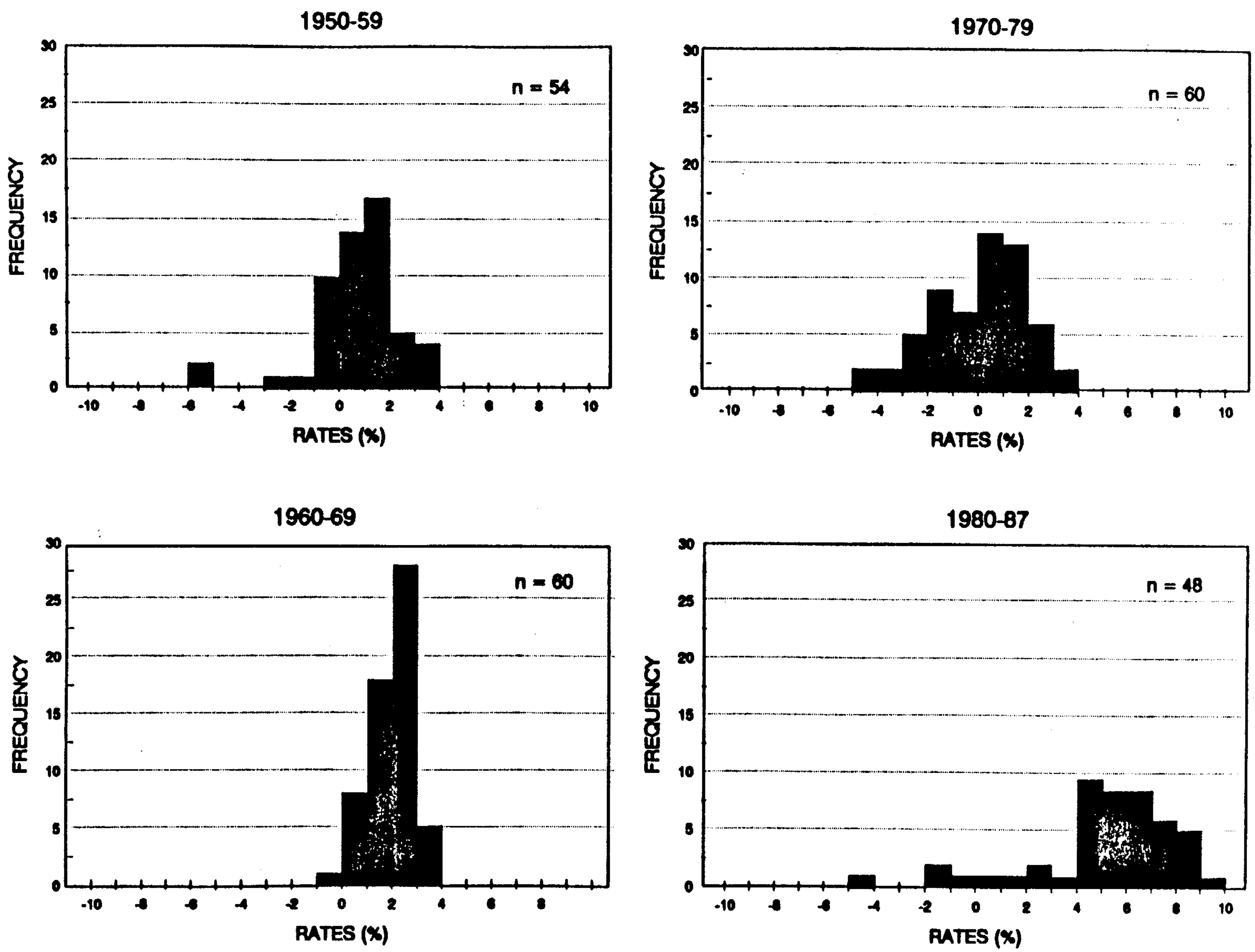


Figure 9. Frequency Distributions for Six Discount Rates by Decade

sensitivity analysis by calculating Present Worths or Equivalent Uniform Annualized Costs using several discount rates. It gives an indication as to how sensitive the outcome of the analysis is to the discount rate. If one alternative is favored over a range of discount rates, the agency can have confidence that the analysis has truly identified the least cost alternative. It is important however to emphasize, that the sensitivity analysis should not be used for changing discount rates on a project by project basis. However, they can help in the selection of the particular alternative that will be built.

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: Resilient Modulus Testing Equipment

Date: 24 1988

From: Chief, Pavement Division

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators

Attached is a summary of responses to our November 16, 1987, memorandum on the above subject. A listing of manufacturers of resilient modulus testing equipment used or proposed for use by State highway agencies (SHA's) is included as an attachment to the summary. There are currently 24 SHA's that are or soon will be performing laboratory resilient modulus testing on unbound and/or bound material. Most SHA's are using laboratory resilient modulus for research purposes only. The equipment used and the cost of that equipment is quite variable as can be seen in the attached summary.

As you are aware, the definitive material property used to characterize roadbed soil and to assign layer coefficients (flexible pavements) for pavement design in the "AASHTO Guide for Design of Pavement Structures" (1986 Guide) is the resilient modulus.

The 1986 Guide recommends that low stiffness materials, such as natural soils, unbound granular layers and even stabilized layers and asphalt concrete be tested using resilient modulus test method AASHTO T274. Although the testing apparatus for each of these types of materials is basically the same, there are some differences, such as the need for triaxial confinement for unbound materials.

The 1986 Guide also states that the bound or higher stiffness material such as stabilized bases and asphalt concrete may be tested using the repeated-load indirect tensile test (ASTM D4123). Appendix F to the 1986 Guide notes that ASTM D4123 provides an <u>estimate</u> of the modulus of asphalt concrete and other relatively low-strength materials under simulated field-loading conditions. The estimate may or may not correlate well with the resilient modulus value obtained using AASHTO T274.

The resilient modulus values can be used directly for the design of flexible pavements, but must be converted to a modulus of subgrade reaction (k value) for design of rigid or composite pavements.

Some of the manufacturers equipment listed in the attachment does not include apparatus needed for triaxial confinement of a specimen. Many States modify standard test procedures for reasons of practicality and speed. In order to learn more about the advantages and disadvantages of the currently used resilient modulus testing equipment, we suggest that SHA's call or write the various State contact persons listed in the attached summary as well as the equipment manufacturers.

We are currently working with the State of Washington to produce a videotape using WSDOT's resilient modulus testing equipment. The tape will outline the AASHTO and ASTM resilient modulus test procedures. It will also include WSDOT's test procedures and explain how and why they deviate from the AASHTO and ASTM test procedures. We anticipate that the tape will be available for distribution later this year.

As noted earlier, the 1986 Guide uses resilient modulus to characterize soil support and to assign asphalt pavement layer coefficients. It further stresses, the need for a more rational approach to incorporate material engineering properties into the asphalt mixture design process. A number of research studies are being conducted by FHWA, NCHRP, and SHRP in this area. We will keep you informed as results become available.

We feel that the information included with this memorandum would be helpful to those States contemplating the use of laboratory resilient modulus, as well as those States which are currently doing work in this area. Sufficient copies of this memorandum and attachments have been provided for distribution to the division offices and their appropriate State counterparts.

If you have any questions, please contact Messrs. Tom Fudaly at 366-1338 or Dan Mathis at 366-1340.

Norman J. Van Ness

SUMMARY OF RESILIENT S EQUIPMENT USAGE BY REGION

REGION 1

	State	Primary Use	Brand & Model	Purchase Date	Purchase Amount	State Contact	Telephone Number
*	Maine	Research	Hicks & Vincent IB	1987	$38,000	Warren Foster	(207) 289-5668
**	New Hampshire	Design (Bound mat'l)	Retsina, Mark VI	1987	$18,000	Phil McIntyre	(603) 271-3151
	New York	Design	SBEL Co.	1980	$22,000	David Suits	(518) 457-4704

* Maine will soon be performing laboratory resilient modulus testing for research work and eventually hope to use laboratory results for design.

** New Hampshire will soon be performing laboratory resilient modulus testing on bound material for use in design.

REGION 3

	State	Primary Use	Brand & Model	Purchase Date	Purchase Amount	State Contact	Telephone Number
	Maryland	Research (Bound and unbound mat'l)	MTS, 410, 413, 414, 422, 464	1983	$100,000	Michael Arastek	(301) 321-3560
	Pennsylvania	Research and Design (Bound mat'l)	Retsina, Mark IV	1981	$10,600	Prithus Kandhal	(717) 787-5229
*	Virginia	Research (Bound mat'l)	Retsina, Mark II	1980	$5,000	Bill Maupin	(804) 293-1948
	West Virginia	Research and Design	MTS (see attachment 1)			Berke Thompson	(304) 348-3664

* The Virginia Research Council performs the research work for the Virginia DOT. They are in the process of obtaining a Retsina Model Mark VI for approximately $15,000.

REGION 4

State	Primary Use	Brand & Model	Material Tested	Purchase Date	Purchase Amount	State Contact	Telephone Number
Florida	Research	MTS, 312.31 MTS, 312.21 (see attachment 2)	Asphaltic Concrete Soils	1975 1975	$50,000 $48,000	Larry Smith or Gale Page	(904) 372-5304
Georgia	Research	(See attachment 3)	Bound	1975 & 1986	$10,000	William Webb	(404) 363-7546
* Kentucky	Research	Structural Behavior Engr. Lab.(SBEL) STD-1000	Bound and Unbound	1974	$5,200	David Allen, University of Kentucky	(606) 257-4513
Miss.	Research	Retsina, Mark IV	Asphaltic Concrete	1980	$10,000	Joe Scheffield	(601) 359-1174

* Kentucky Transportation Cabinet to purchase custom-made model from Materials Testing System (MTS) Minneapolis, MN, for $119,000 by June 1988.

R 5

State	Primary Use	Brand & Model	Purchase Date	Purchase Amount	State Contact	Telephone Number
Illinois	Research	(Built their own device)		-------	Jake Dhamrait	(217) 782-7206
Michigan	Research (Bound mat'l)	MTS	------	-------	Jack DeFoe	(517) 322-5711
Minnesota	Research	MTS 810	1980	$99,500	George Cochran or Neil Magee or Dave Rattner	(612) 296-7134 (612) 296-7848 (612) 296-9740

REGION 6

State	Primary Use	Brand & Model	Purchase Date	Purchase Amount	State Contact	Telephone Number
* Texas	Research (Bound mat'l)	Retsina Mark IV	1975	-------	Paul Krugler	(512) 465-7603
New Mexico	Research (Bound and unbound mat'l)	Custom made by University of Oregon	1982	$45,000	John Tenison	(505) 827-5565

* The Bituminous concrete section of DHT Materials and Test Division (D-9) occasionally does resilient modulus testing. It is not done routinely and done only when additional information about a mix is needed.

REGION 7

State	Primary Use	Brand & Model	Purchase Date	Purchase Amount	State Contact	Telephone Number
* Kansas	Research and Design	Cox and Sons, Inc. C5-4000KA	1984	$57,000 plus $30,000 accessories	Glenn Fager	(913) 296-7410

An additional resilient modulus testing machine has been purchased by KDOT and will be received in early 1988. This unit was manufactured by Research Engineering and is a component type system. The load frame is Model RE-CLF-5000 and the Air Electric Loader is Model No. RE-CL-82. This unit is operated through an IBM PC-AT processor. The cost of this unit is $36,000.00 with up to an additional $5,000.00 in accessories. When this unit is brought on line at KDOT, it will be used primarily for the design of pavement structures. The contact person at KDOT for this unit is Mr. Jeff Frantzen. His telephone number is (913) 296-3008.

REGION 8

State	Primary Use	Brand & Model	Purchase Date	Purchase Amount	State Contact	Telephone Number
Colorado	Research	Retsina Mark III	1974	$6,000	Lex O'Connor or Dick Hines	(303) 757-9449 (303) 757-9724
Utah	Design	MTS	1972	-------	Wade Betenson	(801) 965-4303

REGION 9

State	Primary Use	Brand & Model	Purchase Date	Purchase Amount	State Contact	Telephone Number
California	Research	Retsina, Mark II	1974	$5,600	Robert Doty	(916) 739-2361

Nevada - Will soon purchase a device by "Research Engineering" for research purposes. The State Contact will be Pat Schoener or Ted Beeston (702) 885-5875.

REGION 10

State	Primary Use	Brand & Model	Purchase Date	Purchase Amount	State Contact	Telephone Number
Alaska	Research	Hicks and Vincent	1987	$45,600	Eric Johnson	(907) 338-2121
Washington	Research & Design	Hicks and Vincent IA	1983	$29,000	Newton Jackson	(206) 753-7110
* Oregon	Research	Retsina Mark IV	1980	-------	Dick Dominick	(503) 388-2621

* Oregon has recently ordered equipment manufactured by "Research Engineering" at a cost of $33,600.

West Virginia DOH Resilient Modulus Equipment List

Equipment Manufacturer and Type	Model Number	Date Purchased	Cost
MTS Inc. Material Test System	810 Series	Dec. 1971	$63,923
MTS Inc. 22 Kip Load Frame	Not Available	Late 1983	$3,500 estimated
Schaevitz Engineering Co. Linear Variable Differential Transformer	100MHR Range +/- 0.100 inch	May 1982	$250
Research Engineering Co LVDT Clamps (2 each)	RE-PRC	May 1982	$305 each
Wavetech Inc. Function Generator	186	Not Known	$350
Air Compressor	(Air compressor set up for entire lab is used)		
Blue M. Inc. Construction Temperature Oven	OV490-I	May 1963	$330
Hobart Manufacturing Co. Mixer	C-1[illegible]T	Oct. 1970	$495
Hewlett Packard Co. Oscilloscope	1702A	Late 1974	$6,000
Research Engineering Co. Triaxial Chamber	RE-SA-150	May 1982	$2,920
Mettler Co. Balance	P11N	Dec. 1977	$1,940
Soiltest Inc. Membrane Expander	No number	May 1982	$85
Soiltest Inc. Membranes	T-614	Purchased as needed	$60/doz.

Florida DOT Resilient Modulus Equipment List

System No. 1 - Asphalt Test System

Consists of the following:

1. Load frame - 55 Kip (M.T.S.), Model No. 312.31

2. Activator - 22 Kip (M.T.S.), Model No. 204.63

3. Hydraulic service manifold - (Series 284) (M.T.S.), Series 284

4. Load Cell 10 metric ton - (M.T.S.) - Asphalt, Model No. 661.21A-03

5. Load Cell 1500 D.G.F - (M.T.S.) - Model No. 661.13A-05

6. Temperature control chamber - (Thermotron Corp.), Model No F-3-Ch-Co_2

7. Split Tension Load Frame - (Custom Made)

8. Electronic Console (M.T.S.)

   A. 409 Temp. Control panel
   B. 430 digital indicator panel
   C. 417 counter panel
   D. 410 digital function generator
   E. 442 controller arranged with following modules:
      (a) Serve Control - Model 440.13
      (b) Valve Driver - Model 440.14A
      (c) Feed Back selector - Model 440.31
      (d) Limit detector - 440.41
      (e) A.C. conditioner - Model 440.22
      (f) D.C. conditioner - Model 440.21
   F. 410 pulse sequence panel
   G. 413 Master control panel

9. Gould Brush Recorder - Model -1111-1707-120, consists of the following modules:

   A. D.C. Amplifiers - Model 13-4215-32 (2 each)
   B. Transducer - Model 13-4218-00
   C. Carrier Amplifier - Model 13-4212-02

   Date of Purchase - 1975
   Cost - $50,000

Equipment List

System No. 2 - Soils Test System

Consists of the following:

1. Load frame - 22 Kip (M.T.S.), Model No. 312.21

2. Actuator - 3.3 Kip (M.T.S.), Model No. 204.51

3. Hydraulic service manifold (series 284) (M.T.S.), Series 284

4. Load cell - 500 K.G.F. (M.T.S.) (w/protector), Model No. 3170

5. Triaxile chamber - (Wykeham-Farrance Eng.), Model No. 11006

6. Electronic console (M.T.S.)

   A. 417 counter panel
   B. 410 digital function generator
   C. 442 CONTROLLER - (arrangement is same as system No. 1)
   D. 410 pluse sequence panel
   E. 413 Master Control panel

7. Gould Brush Recorder - arrangement same as system no. 1
   - Model No. -1111-1707-120

   Date of Purchase - 1975
   Cost - $48,000

Hydraulic Power Supply (MTS)

3000 psi capacity
21 gpm
Model No. 510.21B

Date of Purchase - 1985
Cost - $14,300
Note: This Hydraulic Power Supply is capable of supplying both the Asphalt and Soil Test systems with 3000 psi

GEORGIA DOT RESILIENT MODULUS EQUIPMENT LIST

Part	Manufacturer
2 Triaxial Cells	Soiltest
1 Load Frame for 2 Samples	GA DOT - Office of Materials and Research Machine shop
1 Strip Recorder (Brush 2 Channel)	Gould Instruments
4 Pressure Regulators (Model #40-100)	Moore Products Company
4 Pressure Gauges	
4 LVDT's (Transducers Model #SS-203)	G. L. Collins
2 Bellofram (Size 4)	Bellofram Products Company
2 Mufflers	
2 Recycling Timers (Model #CX400)	Eagle Signal Controls
4 Revolution Counters	
2 24 Volt Power Supplies	GA DOT - Research
Miscellaneous Plumbing and Electrical Materials	

Resilient Modulus Equipment Manufacturers

The following is an alphabetical listing of manufacturers of resilient modulus testing equipment that is currently used or proposed for use by the SHA's:

Cox and Sons, Inc.
P.O. Box 674
Colfax, California 95713
Phone: (916) 346-8322

Hicks & Vincent (H&V)
Material R and D
3187 NW Seneca Place
Corvallis, Oregon 97330
Phone: (503) 757-1293

Material Testing System (MTS)
P.O. Box 24012
Minneapolis, Minnesota 55424
Phone: (612) 937-4000

Research Engineering
2640 Dundee Road
San Pablo, California 94806
Phone: (415) 223-4798

Retsina Company
601 Brush Street
Oakland, California 94607
Phone: (415) 268-0821

Structural Behavior Engineering
Laboratories, Inc. (SBEL)
P.O. Box 23167
Phoenix, Arizona 85063
Phone: (602) 272-0274

LONGITUDINAL JOINT CONSTRUCTION AND EDGE DROP-OFFS

STATE-OF-THE-PRACTICE

by

Steve A. Call

Highway Engineer Trainee

FHWA

Pavement Division

March 1989

INTRODUCTION

In January of 1987 a questionnaire dealing with longitudinal joint construction with asphalt pavements was sent out to all State highway agencies by the Transportation Research Board Committee on Flexible Pavement Construction. Forty-five agencies responded to the survey. The questionnaire asked if step-offs (drop-offs) were routinely permitted overnight or longer, before placement of the adjacent mat, on either new construction or on resurfacing projects. Questions followed concerning conditions under which drop-offs were allowed, joint construction techniques, and alternate procedures used. A compilation of the responses to the questionnaire was made in May of 1988 by C.S. Hughes. It included his conclusions and recommendations (see appendix).

Since this questionnaire was sent out there has been much interest and activity in the area of longitudinal joint construction. In addition, many State highway agencies have been encouraged to, and are trying to, develop pavement edge drop-off policies. This paper is an attempt to update and add to the information gathered in the 1987 survey, in order to provide a "state-of-the-practice" report.

As was indicated by the results of the 1987 survey, longitudinal joint construction practices vary from State to State. It is not the intent of this paper to evaluate the various construction practices of the States, but rather to provide information on what different States and regions are doing to mitigate the hazards created by edge drop-offs. To set the stage for this information, a literature review is given detailing the results of the most recent studies concerning the safety aspects of drop-offs.

LITERATURE REVIEW OF SAFETY RELATED ASPECTS OF PAVEMENT EDGE DROP-OFFS

An edge drop-off occurs when there is a vertical difference in height between adjacent road surfaces. Drop-offs may occur as a result of paving or resurfacing operations, or milling or other types of excavation work. They also may occur as the result of the deterioration of an adjacent surface. The hazard results when a driver of a vehicle crosses over the drop-off, dropping his wheel(s) down onto the lower surface, and then tries to steer back up onto the higher surface. An overcorrection may result in loss of vehicle control, while a gradual correction may result in the phenomena known as "scrubbing." Scrubbing occurs when the steering angle is insufficient to overcome the opposing force of the face of the drop-off, hence, "scrubbing" of the side of the tire occurs along the drop-off face. Once sufficient steering angle is imparted, the wheels mount the pavement edge and, in the absence of an opposing force, the vehicle has a sudden change of direction, often times causing lane exceedance or loss of control. As the height of the drop-off increases, so does the severity of the situation. For this reason engineers have tried to determine the height of drop-offs at which mitigating action needs to be initiated.

Current literature cites four major studies that have been conducted by various agencies since 1976 pertaining to vehicle responses to an edge climb maneuver. These studies, in chronological order, are:

- The Effect of Longitudinal Edge of Paved Surface Drop-off on Vehicle Stability, E. Nordlin, D. Parks, R. Stoughton, and J. Stoker, California Department of Transportation, 1976.

- Vehicle Controllability in a Pavement/Shoulder Edge Climb Maneuver, R. Klein, W. Johnson, and H. Szostak, Society of Automobile Engineers Technical Paper Series, 1978.

- Pavement Edges and Vehicle Stability- A Basis For Maintenance Guidelines, Don Ivey and Richard Zimmer, Texas Transportation Institute, 1982.

- Pavement Edge Drop, Paul Olson, Richard Zimmer, and Val Pezoldt, University of Michigan Transportation Research Institute, 1986.

This paper will summarize both the test procedures and the findings of these four studies. It is important to note that testing procedures differed in the four studies because each of the studies had different goals. As the pool of knowledge grew, the procedures also evolved somewhat. It is recognized now that when a vehicle drives over a drop-off, through inattention, recovery from a scrubbing condition is the most difficult form of recovery and therefore should ultimately be the determining factor in the conclusions.

In Nordlin's study 50 tests were conducted using professional drivers in four different vehicle types: small, medium, and large passenger cars, and a full-sized pickup truck. Three different drop-off heights were used- 1.5, 3.5, and 4.5 inches. Vehicles were driven from an A.C. shoulder onto either an A.C. or soil surface, and returned to the A.C. shoulder at speeds of 60 mph and at angles of less than 10 degrees. In these tests either two or four wheels were dropped off the shoulder and then returned. This study did not examine the pavement edge scrubbing condition and used only vertical drop-offs. In addition the combined width of the lane and shoulder was 17 feet, allowing more room for recovery.

Nordlin found that although experiencing a "significant jolt and accompanying front end noises" at the larger drop-off heights, there was no real problem with vehicle stability, no deviation in vehicle trajectory, and no encroachment into adjacent lanes. Less than one wheel revolution was required for the first wheel to mount the various drop-off heights.

In Klein's study three different size passenger cars were used in closed loop tests. The car's two right wheels were gradually dropped 4.5 inches onto an earth shoulder. Pylons were used to keep the wheels close to the pavement edge increasing the chance for scrubbing. Klein tested only vertical edge drops in his study. The drivers were told to drive at constant speeds, increasing from 25 to 55 mph in 5 mph increments on successive runs. Twenty-two naive (non-professional) subjects were used on 73 runs. On 34 runs the tires did not scrub, but on the other 39 they did. On the non-scrubbing runs there were no lane exceedances, but over half (22 of 39) of the scrubbing runs resulted in lane exceedances. Klein found that a correlation existed between vehicle speed and lane exceedance. Each vehicle had a critical speed at which recovery from shoulder climbs became difficult (83% failure). In the two smaller cars the critical speeds were 30 and 32 mph. In the larger car the critical speed was 42 mph.

In the open looped test Klein used four test vehicles and drop-off heights ranging from 2 to 4.5 inches. Once again the most hazardous results occurred during scrubbing. Whether or not a vehicle was able to climb a drop-off was a function of closing velocity. On a graph of closing velocity (the component of velocity perpendicular to the pavement edge) verses drop-off height, it was shown that, at a height of about 4 inches, closing velocity needed to climb the pavement edge increased sharply. For this reason 4 inches was suggested as a maximum drop-off height. Five inches was determined to be the maximum height that could be climbed due to the undercarriage characteristics of vehicles and side forces on the wheel.

Don Ivey's study built upon the preceding two studies by Nordlin and Klein. He used drop-off heights of 1.5, 3, and 4.5 inches with various edge shapes including a 45 degree taper and a vertical edge. He used, as previously, three passenger cars and a pickup for test vehicles. Different types of drivers were used ranging from professional to naive, however only the professional driver drove the complete matrix of tests. Test speeds of 35, 45, 55 mph were driven with three different vehicle positions: scrubbing, two wheels off the pavement, and four wheels off the pavement. Ivey used a subjective rating system which had the driver rate the difficulty of the edge climb maneuver, however only the professional driver was used to rate the various climbing maneuvers since he was the only one to drive the complete matrix of tests.

Ivey found that the professional driver handled all the variations easily except for the 4.5 inch edge drop, on the vertical edge, in the scrubbing condition. It was therefore concluded that the 4.5 inch edge drop was unsafe at speeds as low as 35 mph. The 45 degree angle was safe, even at a 6-inch drop, at speeds up to 55 mph. Velocity, drop-off shape, and proximity to the edge were the factors with the greatest influence on safety.

Paul Olson's study used most of the same variables that the former studies used (i.e. vehicle type, velocity, edge shape, shoulder type, and edge drop heights). His investigation, however, was "primarily concerned with evaluating the performance of ordinary (naive) drivers on their first encounter with the edge drop". He also examined "subject learning" and found that its effects were minimal. The criteria he used to determine the safety of each maneuver was lane exceedance beyond a 12.5-foot lane with the drop at the edge of the lane.

Olson found that 4.5-inch vertical drops could not be negotiated by the naive subjects safely at speeds as low as 20 mph. The 3 inch vertical drop could be negotiated at speeds of between 20 and 25 mph in smaller cars and 30 mph in the largest passenger car. No safe maximum height was defined for speeds greater than 25 mph. Using the 45 degree bevel edge, virtually all runs at heights up to 4.5 inches were made without intruding beyond the lane adjacent to the edge drop at speeds up to 55 mph. The beveled edge was a suggested treatment at higher speeds. Finally, he concluded that height, not shoulder material, was the controlling factor, and that small cars had more difficulty than large cars. The results of Olson's study suggest that the recommendations of other studies are not adequate for high speed facilities, if the determining factor is recovering from a scrubbing condition. Maximum vertical heights of edge drops on these facilities should be less than 3 inches, although how much less has not been determined. Future studies should address this issue.

PAVEMENT EDGE DROP-OFF POLICY- STATE OF THE PRACTICE

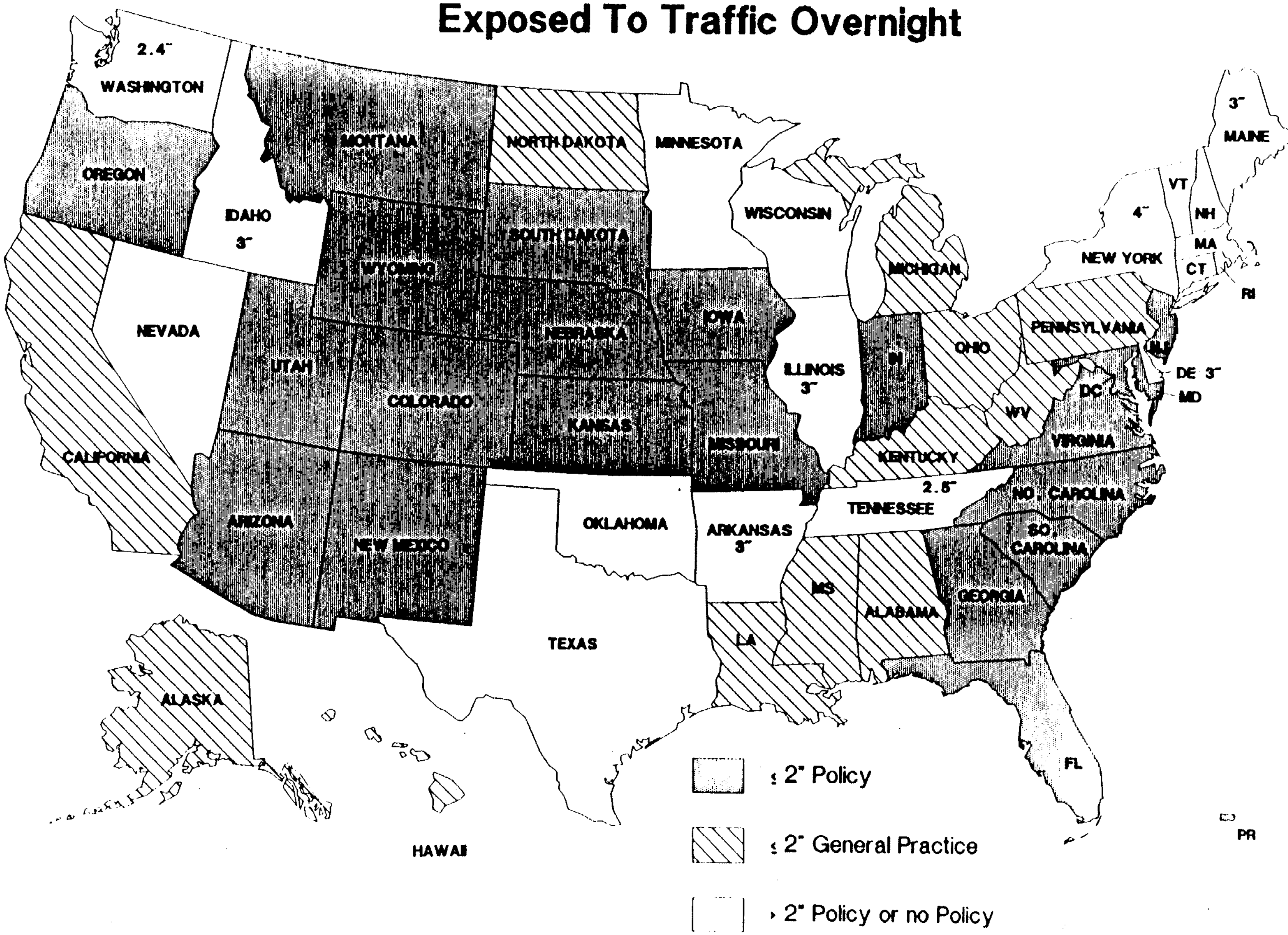
In the memorandum issued December 1, 1986, from the FHWA Construction and Maintenance Division, it states that drop-offs "greater than 2 inches, left overnight, and immediately adjacent to traffic, have high accident potential." The C&M Division recommended corrective action or a combination of actions for drop-offs greater than 2 inches (see appendix). The memorandum "strongly encouraged" the regions to work with the states in developing pavement drop-off policies.

The following figure illustrates the "state-of-the-practice" in the United States in regards to the 2-inch drop-off level. The figure divides the States into three groups. The first group consists of those States that have formal drop-off policies that allow a 2-inch or less maximum drop-off in work zones exposed to traffic overnight or require a taper for drop-offs exceeding 2 inches. The second group consists of those States that have not formulated a formal policy, but their general practice meets the requirements of the first group. The third group consists of those States that have a policy allowing greater than 2 inches, or have no policy at all.

This information was obtained from surveys conducted by TRB and various regions, and supplemented by information obtained from telephone conversations with regional and division personnel. It is noteworthy that in some cases where there was more than one source available, there was a lack of agreement as to policy or practice. In these cases, preference was given to sources citing State Specifications or Codes.

A more detailed summary of each State's position concerning drop-offs follows. The States are organized according to FHWA regions so as to show patterns on a regional basis. While the information is not detailed, in some cases, each State is represented and the summary takes advantage of as many sources as possible, given the time constraints imposed.

Maximum Vertical Drop Off Exposed To Traffic Overnight



REGION 1

With a few exceptions, the States in Region 1 do not have formal pavement drop-off policies.

In the State of **Connecticut** edge drop-offs are not considered to be a problem. For the most part traffic is kept off the joint area, using the rest of the roadway. With multiple lifts, the pavement in adjacent lanes is matched before beginning the next lift.

The State of **Maine** uses channelizing devices spaced every 50 feet when the drop-off exceeds 3 inches in vertical height. On a resurfacing project creating drop-offs of less than 3 inches, channelizing devices are placed 2 feet outside of the edge of the pavement at 600 foot intervals with the MUTCD, W8-9 "low shoulder" signs every 1/2 mile. When the drop-off is greater than 3 inches, 4 feet of shoulder material is required to be placed with channelizing devices placed as stated before. The speed limit on such projects is set at 45 mph.

Massachusetts has elected not to adopt a drop-off policy because "in some instances such a policy would create more problems than it would solve." Instead it was decided that each traffic control plan should place special attention to drop-offs in work areas and individual needs should be carefully evaluated.

The State of **New Hampshire** has no specific height requirements, but specifications state that open excavations shall not be exposed overnight, on weekends, or on holidays. No guidelines for resurfacing projects were given, but the State feels that they have few drop-off situations because of their specifications, and attention given by project personnel.

New Jersey has the strictest policy in the Region, requiring a gravel wedge at a slope of 6:1 when adjacent excavation is greater than 2 inches. On resurfacing, adjacent lanes of pavement are matched every 1500 feet. Lift thicknesses are 2 inches. They also use a longitudinal wedge joint design. Appropriate signing and a double yellow line is required on their resurfacing projects to keep traffic off the joint.

The State of **New York** has not adopted a formal drop-off policy. The State relies on a section in its Standard Specifications. It was requested that NYSDOT develop a special specification dealing with drop-offs. This issue is still unresolved at this time.

Rhode Island has not developed a formal policy because they did not feel that drop-offs were a problem. It is general practice, with drop-offs greater than 4 inches, to require either a 4:1 transition slope or a median barrier.

Vermont does not have a specific policy on drop-offs, but does require the pavement to be matched in adjacent lanes by the end of the day. This issue was to be discussed prior to the start of the 1988 construction season.

In **Puerto Rico** the standard specifications state that pavement repairs and construction on both bituminous and PCC pavements will be initiated and completed during the same working day. This eliminates unnecessary drop-offs along and adjacent to travel lanes. Where isolated or continous excavation is expected as part of the construction project, appropriate channellizing devices are specified. No height specification for drop-off are given.

(The information for this summary was obtained from a survey conducted by TRB and a survey taken by FHWA Region 1.)

REGION 3

The State of **Delaware** has no height specifications for drop-offs on milling type projects, however, they said that drop-offs of 3 or more inches do occur. On resurfacing projects, drop-offs of 1 inch or more require signing. Drop-offs of between 2 to 6 inches require cones or vertical panels and are tolerated for the length of one days paving operation. Drop-offs of greater than 6 inches require concrete barriers when within 10 feet of the traveled way and require barricades when outside of 10 feet.

Maryland requires pavement in adjacent lanes to be matched by the end of the working day when vertical drop-offs exceed 2 inches. When drop-offs are less than 2 inches pavement must be matched within 24 hours. Reduced speed limits are enforced within construction work zones. They said that nothing was mentioned in their specifications for excavation work.

In **Pennsylvania** longitudinal edge drop-offs are generally limited to 25 feet in length at the end of each days work, and a maximum of 2 inches in height. This does vary from district to district.

Virginia requires that pavement having drop-offs greater than 2 inches, have lanes of adjacent pavement matching by the end of a days operation. Appropriate signing is required when drop-offs occur.

West Virginia generally sets 2 inches as the maximum drop-off allowed although it has no formal policy. They generally do not prevent traffic from crossing the longitudinal joint.

(The information for this summary was obtained from a questionnaire sent out by TRB and from specifications from the States of Virginia and West Virginia.)

REGION 4

Although not all the States in Region 4 have adopted a formal policy mitigating pavement drop-offs, they, at least in general practice, have strict limits.

Alabama generally does not permit drop-offs of more than 2 inches to exist overnight. If they occur a temporary 1:1 longitudinal taper joint is required and is later removed when paving resumes.

Florida sets a maximum height of 1.5 inches for drop-offs that traffic is expected to cross. This may be increased to 2 inches for low speed situations. Where traffic is not expected to cross, less than 2 inch drop-offs require warning signs only. Drop-offs between 2 to 4 inches require drums, panels, or barricades. With drop-offs greater than 4 inches either positive separation or a 3:1 wedge is required. For temporary conditions, drop-offs greater than 4 inches may be protected by drums, panels, or barricades for short distances, during daylight, while work is being performed.

The State of **Georgia** requires pavement on the Interstate system to be matched in adjacent lanes by the end of the next day. They set 2 inches as the maximum height allowable for drop-offs exposed to traffic. They also require appropriate signing where drop-offs occur.

Although **Kentucky** does not have a formal policy concerning drop-offs, they said that projects with drop-offs are generally closed to traffic.

Mississippi generally allows drop-offs of up to 2 inches without protective devices and requires protective devices at drop-offs greater than 2 inches.

North Carolina has set 2 inches as the maximum drop-off height allowed. All paving projects in the State must have adjacent lanes of pavement matched within 24 hours. Use of the W8-9a sign is required when traffic is exposed to drop-offs.

South Carolina sets 1.5 inches as the maximum drop-off height they will allow on resurfacing projects. They also require warning signs.

Tennessee does not allow night traffic on projects where drop-offs occur. Pavement must be matched in adjacent lanes within 24 hours. Warning lights and barrels are required when the drop-off exceeds 2.5 inches.

(Information for this summary was obtained from a survey conducted by TRB and information provided by Region 4.)

REGION 5

A survey of the States in Region 5 was conducted in 1987 and was confirmed by telephone conversation in February of 1989.

Standard Specifications in **Illinois** require that drop-offs of 3 or more inches at the edge of the pavement be protected by type I or II barricades at 100-foot intervals when they were greater than 4 miles in length. This applies to both resurfacing and excavation and milling type projects. The pavement in adjacent lanes is required to be matched before the next lift is placed, and within 24 hours. Appropriate signing is required and no open trenches greater than 3 inches are allowed to exist overnight.

Indiana requires, on resurfacing projects only, that barricades be placed where drop-offs exceed 2 inches adjacent to the pavement. Up to 3-inch drop-offs are permitted outside the shoulder. These specifications are contained in the Contract Special Provisions. All other situations are covered in the Traffic Control Plan. Deep excavations at the edge of the pavement require temporary concrete barriers to separate them from the traveled way.

The State of **Michigan** does not have a formal policy, but has specifications that state that low shoulders be delineated and that hazards be removed as soon as possible. Pavement in adjacent lanes must be matched by the end of the day or else warning signs must be provided and barricades placed every 100 feet to delineate the traveled way. They frequently make use of a longitudinal taper joint when drop-offs are expected to be under traffic.

The State of **Minnesota,** likewise, does not have a formal pavement drop-off policy, but as a general practice allows drop-offs under 2 inches to be left untreated unless the drop-off occurs between lanes, then warning signs are required. Drop-offs between 2 and 4-6 inches (varies between districts) are signed as low shoulders and **may** be delineated with channelizing devices. Drop-offs over 4-6 inches are signed and delineated with channelizing devices. In most cases adjacent lanes of pavement must be matching by the end of the day. Excessive drop-offs require the use of concrete barriers.

Ohio has no official drop-off policy, however, drop-offs are considered and discussed during the development of the traffic control plan. Their specifications allow for a maximum 2-inch drop-off and require pavement in adjacent lanes to be matched within 24 hours after placement. Open trenches are protected by barrels. Ohio has utilized, on many occasions, all the techniques discussed in the 1985 memo from the C&M Division.

The State of **Wisconsin** does not have a drop-off policy, but as a general rule, uses the provisions in the MUTCD. These are included in the contract plan.

(The information in this summary was obtained from surveys conducted by TRB and Region 5.)

REGION 6

The States in Region 6 follow no definite pattern when it comes to mitigating drop-offs in work zones.

The State of **Arkansas** allows a maximum drop-off of 3 inches on the centerline pavement edge and 4 inches maximum at the edge of the shoulder. When resurfacing lifts are less than 1 inch no treatment is necessary. Between 1 and 3 inches, at the centerline, an uneven lane sign (WSP-1) is required. At the shoulder edge, a drop-off of between 1 and 4 inches requires that a drop-off sign (WSP-2) be used. Adjacent lanes must be matched within 24 hours unless an emergency arises.

Although there is no formal policy in **Louisiana**, as a general rule, drop-offs of less than 2 inches are allowed to exist without any treatments while drop-offs of greater than 2 inches require matching lanes of pavement by the end of the day. They are currently looking at a policy patterned after one being developed by the State of Oklahoma.

The State of **New Mexico** requires a 6:1 taper on the edges of lifts greater than 1.5 inches in vertical height. At heights greater than 3 inches they require panels or barrels in addition to the taper. Adjacent lanes of pavement are usually matched within 24 hours.

Oklahoma, at present, has no drop-off policy in construction work zones. The State is currently developing a policy based on the state-of-the-practice in other States.

Because of the size of the State and the decentralized nature of the State DOT, **Texas** does not have one single pavement drop-off policy. Each district sets their own standards which they will follow, so practices vary throughout the State. Some districts are making use of the longitudinal taper joint.

(The information in this summary was obtained from a survey conducted by TRB and telephone conversations with each FHWA Division Office's Pavement Specialist.)

REGION 7

In a memorandum dated May 13, 1988, Region 7 strongly encouraged States in that Region to develop policies, mitigating pavement edge drop-offs, conforming to the following guidelines:

1) For "vertical drop-offs of 1 to 2 inches in height. . . consideration should be given to providing appropriate signing and delineation, and limiting drop-off length and time of exposure."
2) Drop-offs from 2 to 4 inches should have a slope of 1:1 or flatter with appropriate warning signs and delineation.
3) Drop-offs over 4 inches should have a 3:1 or flatter drop-off slope and obstruction free area or positive separation.
4) A pavement edge that traffic is expected to cross should not have an effective height greater than 1 inch. Greater heights (up to 3 inches) should be treated with a wedge slope of no steeper than 3:1. The TCP's should provide for a reduced speed limit of 35 mph.

The Region further stated that each situation should be thoroughly and individually analyzed, taking into account cross section features, traffic volume and mix, speed, and practicality and feasibility of the treatment options.

The four States in this Region, **Iowa, Kansas, Missouri, and Nebraska** have essentially complied with the guidelines recommended by the Region. The State of Kansas has proposed that all lifts have a 1:1 wedge and uses channelizing devices at spacings equal to twice the speed limit. The State of Missouri allows a 2-inch height differential (their maximum lift thickness is 1 3/4) before they require any kind of treatment on both traversable and non-traversable sections.

(The information for this summary was obtained from a survey conducted by TRB and a survey conducted by Region 7.)

REGION 8

Most of the States in Region 8 have developed a formal policy mitigating edge drop-offs.

The State of **Colorado** allows a 1 inch, untreated, maximum drop-off height. Any drop-off exceeding 1 inch, and exposed to traffic, must use a 3:1 slope joint at the longitudinal edge. They also require appropriate signing throughout projects where drop-offs occur.

In the State of **Montana** all longitudinal joints greater than 3/4 of an inch in height must have a 5:1 tapered longitudinal joint.

In **North Dakota,** although there is no policy, drop-offs are generally limited to 1.5 inches in height and pavement in adjacent lanes must be matched within 24 hours.

The State of **South Dakota** has a policy limiting the height of drop-offs to 2 inches and requiring adjacent lanes of pavement to be matched within 24 hours. On multi-lane highways traffic is kept off the joint entirely. Appropriate signing is required where ever drop-offs occur.

Although **Utah** allows up to 4-inch drop-offs, pavement in adjacent lanes must be matched by the end of the day so that no drop-off is left exposed overnight. A sloped 3:1 wedge at the longitudinal joint is sometimes used.

In the State of **Wyoming** any paving operation that creates a drop-off of more than 1 inch shall have pavement in adjacent lanes be matching by the end of the day. In situations where this is not possible a 3:1 longitudinal sloped joint is used.

(The information for this summary was obtained from a survey conducted by TRB and from Wyoming State specifications.)

REGION 9

No State in Region 9 has developed a formal policy mitigating the hazard of pavement drop-off.

The State of **Arizona** uses a 4:1 wedge joint at the longitudinal pavement edge between adjacent lifts. A study performed by Arizona DOT has shown that superior compaction is obtained at the joint with this technique. They use warning signs when the vertical difference between lanes is between 1 to 3 inches and cones, drums, or barricades when the difference is greater than 3 inches.

California is currently working on a drop-off policy for their State. As a general practice they allow a maximum drop-off of .15 feet (1.8 inches) between lifts. They require appropriate signing where drop-offs exist.

Although they do not have a formal policy, the State of **Hawaii** generally does not allow drop-offs to exist overnight by requiring the full travel way to be paved daily. There is usually no more than a 3 inch height difference between lifts. The longitudinal sloped joint is sometimes used at the discretion of the engineer.

The State of **Nevada** has no formal policy concerning maximum allowable drop-off height. The length of an exposed drop-off can not extend beyond the length of 1 days paving. Appropriate signing is required on projects where drop-offs exist.

(Information for this summary was obtained from a survey conducted by TRB and from Region 9 Pavement Specialist.)

REGION 10

The States in Region 10 treat edge drop-offs differently.

Alaska at present has no formal policy dealing with drop-offs, however they are currently working on one. As a general practice they allow drop-offs to exist for one day's paving operation and allow for a maximum drop-off height of 2 inches.

The State of **Idaho** has no formal policy concerning edge drop-off heights. Drop-offs are handled on a job-by-job basis at the discretion of the engineer. They do require appropriate signing where drop-offs exist. On resurfacing projects the lifts are generally 3 inches thick. In the past Idaho has used the sloped longitudinal joint, but it is not now included in the specifications.

In **Oregon** if the drop-off height is greater than 2 inches then the pavement in adjacent lanes must be matched by the end of the day or a 10:1 sloped wedge must be used at the longitudinal joint. The joint is then cut back to a vertical face when paving resumes. If the drop-off is between 1 and 2 inches in height then adjacent lanes of pavement must be matched within 24 hours.

In **Washington** the general practice is to have drop-offs not exceed .20 feet (2.4 inches) in height where exposed to traffic. When drop-offs exceed this height channelizing devices are required. The State requires pavement in adjacent lanes to be matched within 24 hours. They also require appropriate signing where exposed drop-offs exist.

(The information for this summary was obtained from a survey conducted by TRB and information collected by Region 10.)

CONCLUSION

In general, the various State highway agencies have attempted to set some limits in height and length for drop-offs on resurfacing projects. Recently these limits have come in the way of formal policies issued by the State. Forty percent of the States have developed formal policies at this point in time, with several more currently working on such policies. In nearly all cases, these policies conform with the suggestions in the memorandum issued by the Construction and Maintenance Division in December of 1986. While these policy statements mostly refer to resurfacing projects, it is felt that the 2-inch criteria could be used as a standard for milling and excavation type projects and even as a criteria for maintenance.

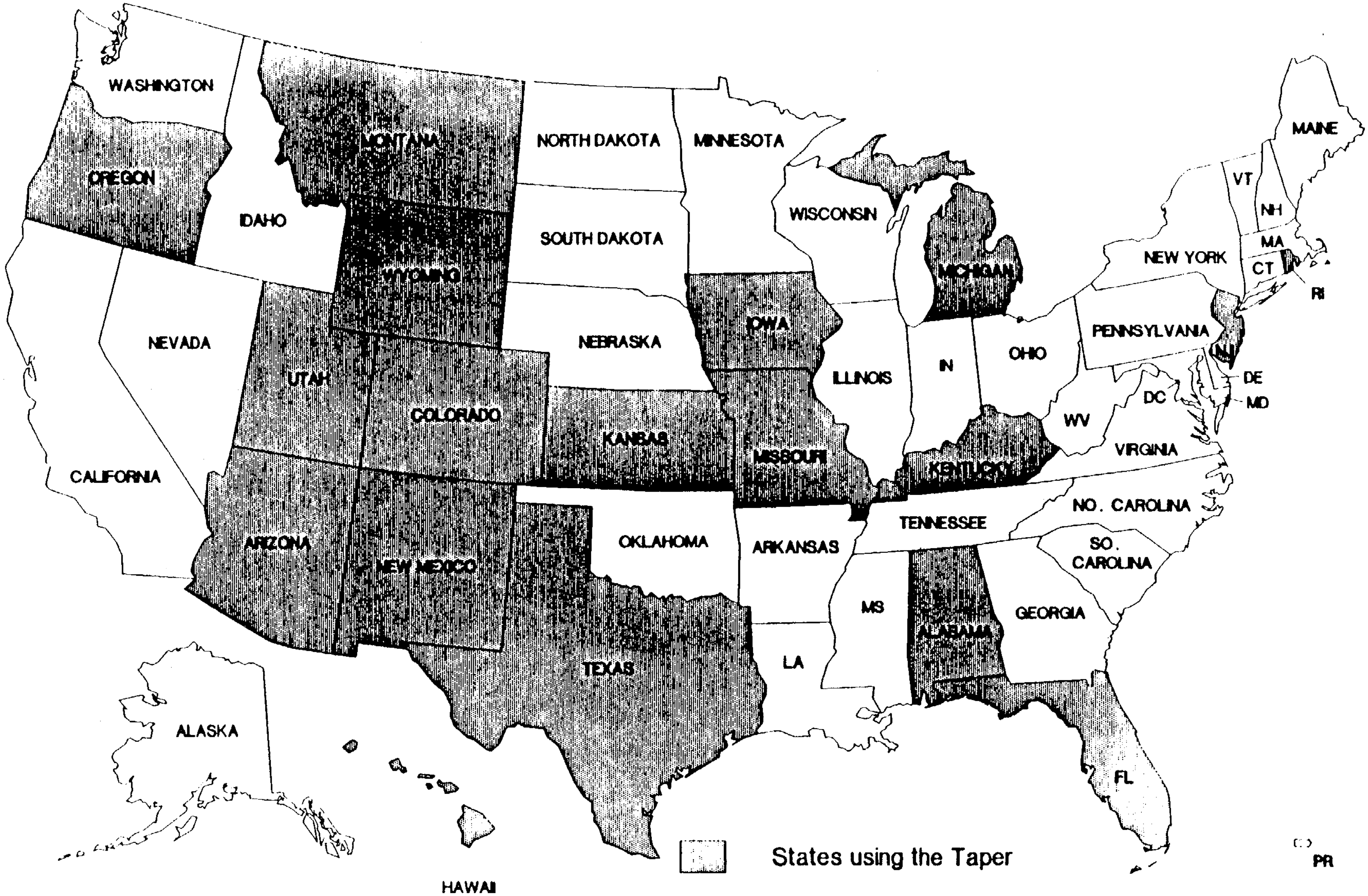
LONGITUDINAL JOINT CONSTRUCTION METHODS

The original intent of the TRB questionnaire sent out to State agencies in 1987, was to determine the state of the practice with longitudinal joint construction with flexible pavements. One practice that is growing in popularity is the use of a longitudinal wedge joint between adjacent lifts of asphalt. Several States already use a tapered edge when a longitudinal edge is exposed to traffic (see figure 2). Studies have demonstrated (see literature review) the safety benefits from the use of such a treatment. In many cases before the adjacent lane is placed, the wedge is cut back to a vertical edge for the joint between lifts. Recently some state highway agencies, namely Arizona and New Jersey, have experimented with the use of the tapered edge as the joint itself as opposed to the more common vertical butt joint. In the research which has been performed, both States claim to get a superior joint with the tapered edge, or "wedge edge." Higher and more uniform densities have been consistently obtained in the area of the joint which is believed will result in a longer pavement life. The tapered joint is expected to yield improved rideability because fewer transverse joints would be required in the pavement. This is because the pavers would not be required to be pulled back at specified lengths for the paving of adjacent lanes, in order to maintain matching pavement requirements normally associated with the use of vertical butt joints.

The State of Arizona originally used a 6:1 sloped wedge, but this has changed to a 4:1 wedge. It is formed by a sloping shoe attached to the paver in order to form the joint. The face of the wedge joint is compacted with a pneumatic tired roller, and then the adjacent lane is paved to finish the joint. The state of New Jersey uses a steel plate attached to the paver forming a wedge of 3:1 slope. The joint face is not compacted, but it is heated with an infrared heater immediately preceding the placement of the adjacent lift, for better bonding. For more information, the reader is encouraged to contact the previously mentioned State highway agencies.

From the research which has been performed to date in this area, the "wedge edge" appears to be a viable solution to the drop-off problem on paving and resurfacing type projects. Instead of creating problems with joint construction it has been shown to yield many desirable benefits.

Usage of Tapers as Longitudinal Edge Treatments



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U.S. Department of Transportation

Federal Highway Administration

Memorandum

Washington, D. C. 20590

Subject: Guidelines for Mitigating Pavement Dropoffs in Construction and Maintenance Work Zones

Date: DEC 1 1986

From: Chief, Construction and Maintenance Division
Office of Highway Operations

Reply to Attn of: HHO-31

To: Regional Federal Highway Administrators
Regions 1-10
Direct Federal Program Administrator

One of the problems noted during our 1986 construction reviews and work zone safety reviews involves pavement dropoffs adjacent to construction and maintenance activities. These dropoffs include those created by pavement or bridge deck removal work, shoulder excavations, and the placement of new layers of pavement. When not properly addressed, dropoffs may lead to an errant vehicle losing control resulting in property damage, injury, and possibly death. It was found that many States do not have any policy or guidelines addressing this hazardous situation. With the growing number of 3R/4R projects, there is potential for dropoff incidents to increase significantly.

To address this concern, information has been compiled and used to develop steps to mitigate potentially hazardous dropoffs. These suggested procedures are based on findings from recent research, current policies and guidelines from a number of States, and consideration of construction operations. The information presented here is not intended in any way to represent policy or to serve as a directive of the FHWA, nor does it represent or promulgate any new standard. Instead, this information is to provide guidelines to States in the development of their own dropoff policy.

Any dropoff is considered hazardous, but those greater than 2 inches, left overnight, and immediately adjacent to traffic have a high accident potential. For such situations, one or a combination of the following mitigating measures is recommended:

1. Specify that the contractor schedule resurfacing or construction operations such that no dropoff is left unprotected overnight, or, as a minimum, limit the length of the dropoff and the period of exposure.

2. If feasible, place steel plates to cover an excavation or trench. A wedge of material around the cover may be required in order to assure a smooth transition between the pavement and the plate. Warning signs should be used to alert motorists of the presence of steel plates particularly when the plates are on the travel lanes.

3. Place a wedge of material along the face of the dropoff. The wedge should consist of stable material placed at a 3:1 or flatter slope. Warning signs may be needed in advance and throughout the treatment. Pavement markings or markers are useful in delineating the edge of the travel lane.

4. Place channelizing devices along the traffic side of the hazard and maintain a 3-foot wide buffer between the edge of the travel lane and the dropoff. The minimum spacing of the devices in feet should be, at most, twice the speed in miles per hour. Dropoff warning signs should be placed in advance and throughout the dropoff treatment.

5. Install portable concrete barriers or other acceptable positive barriers with a 2-foot buffer between the barrier face and the traveled way. An acceptable crashworthy terminal or flared barriers are required at the upstream end of the section. For nighttime use, the barriers must be supplemented by standard delineation devices, i.e., paint, retroreflective tape, markers, or warning lights.

For dropoffs greater than 6 inches, recommendation 5 is strongly suggested if recommendations 1 or 2 are not feasible. Speed reduction measures need to be considered particularly for recommendations 4 and 5. Although these mitigating measures are directed to nighttime conditions, dropoffs must also be properly addressed during daylight operations.

We recognize that there may be some reluctance by the States to develop a dropoff policy or guidelines. The primary concern that has been stated in the past is that the development of such a policy would increase the potential for tort liability actions. It has however also been stated that the existance of properly developed policies and conformance to those policies can in fact provide the State with a good defense against tort liability. More important however, is that such policies will provide greater protection from accidents and injuries for the motorist.

We strongly encourage you to work with the States on the development of such policies. If any further information or technical assistance is needed, please contact us at your convenience.

James A. Story
for Bob B. Myers

TRANSPORTATION RESEARCH BOARD
COMMITTEE ON FLEXIBLE PAVEMENT DESIGN

COMPILATION OF QUESTIONNAIRE ON LONGITUDINAL JOINT CONSTRUCTION

The questionnaire on longitudinal joint construction was developed to determine practices and concerns of leaving an open joint when paving. The questionnaire focused primarily on safety to the traveling public and joint durability. A copy of the questionnaire is attached.

Responses were received from 45 states, 2 turnpike agencies, and 4 Canadian Provinces. The compilation of these responses follows.

Thirty-five agencies allow step-offs (open faces) for new construction and thirty-three allow them on resurfacing. Of the 26 agencies allowing this practice and having a maximum step-off, 62% have a maximum of 2"; 19% have a maximum of 1 1/2"; and only 15% allow 3" or more. Five agencies require a taper and this varies between 3:1 and 10:1. Twenty-nine agencies have a maximum time limit of 1 day or 24 hours over which to pave the adjacent lane. The others have no specified time limit.

The question addressing signing required answers which were somewhat hard to compile because the Manual of Uniform Traffic Control Devices (MUTCD) has no standard sign for a lane step-off or uneven paving. Therefore, misinformative signs or, more often, signs that are designed by the agency are used to alert the public of the step-off. Six agencies use the standard signs of Low Shoulder (W8-9A) or No Passing (W14-3). Fourteen agencies use special signs with 10 either stating or illustrating Uneven Pavement, 3 state Abrupt Edge and one says Center Line Drop Off. Thirteen agencies use no signs mentioning the step-off.

Of 32 agencies requiring special longitudinal joint techniques, one or more of the following techniques are used.

Matching shoe	53%
Tacked joint	53%
Cutback to vertical face	38%
Taper	19%
30' ski	6%
Joint heater	3%

Several agencies stated that tacking or cutting back to a vertical face was required, if necessary.

The agencies that do not allow an open joint require the contractor to move the paver back and square up daily when paving under traffic. For new construction, full width paving and paving in echelon are generally allowed as alternatives to moving the paver back daily.

No agencies reported any special density requirement on a joint. One is attempting to develop a joint density specification.

Thirty-eight of the agencies responded that they have no specified methods to prevent rounding of the joint edge by traffic. Nine (18%) do not allow any traffic on the joint at any time.

The question requesting the responders opinion as to hov hazardous a step-off is to various vehicles drev some interesting responses. One responder invoked the fifth amendment. The ratings are listed belov. Many responders assumed the "no" column, left in through a design flav, to mean "not hazardous" and thus resulted in an additional rating to that intended.

	Rating			
Hazardous to:	Extremely	Somevhat	Slightly	Not
Tractor Trailers	6	14	18	10
Passenger Cars	7	20	15	6
Compact Cars	16	19	10	3
Motorcycles	32	10	6	0

This response is in line vith anticipated results. Motorcycles and compact cars are thought to be the most affected and tractor trailers the least affected.

The question concerning special procedures or deviations did not drav any comments not already included in the compilation. Likevise, the question requesting special joint edge shapes only provided information on tapers, vhich has already been categorized.

CONCLUSIONS

1. Almost 2/3 of the agencies responding allov step-offs.

2. Tventy-five of the tventy-six agencies alloving step-offs, permit 1 1/2" or greater.

3. a) There is no standard sign for a lane step-off in the Manual of Uniform Traffic Control Devices (MUTCD).

   b) In the absence of a MUTCD approved sign, many different signs, some misinformative, are used.

4. The use of a matching shoe and a tacked joint are the tvo most often used special longitudinal joint requirements.

5. Most respondents feel that the hazard of a step-off affects motorcycles more than cars or trucks. Of cars or trucks, compact cars are felt to be most severely affected.

RECOMMENDATION

The only recommendation that is apparent from this compilation is that a need exists for the National Committee on Uniform Traffic Control Devices to approve a standard sign vhich can be used for pavements vith step-offs.

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: ACTION: Life-Cycle Cost Analysis

Date: SEP 15 1992

From: Chairman, PMCG

Reply to Attn of: HNG-42

To: PMCG Members (See Attached List)

A Life-Cycle Costing (LCC) Task Force has been formed in response to LCC interest expressed by the FHWA Research and Development Executive Board at its 1991-92 winter meeting. The Task Force consists of representatives from the Associate Administrators for Policy (HPP-12), Research (HNR-20), Program Development (HNG-42), Motor Carrier (HIA-20), and Administration (HCP-22). The Task Force mission is to develop recommendations for the Research and Development Executive Board on appropriate ways to incorporate LCC analysis into the Federal-aid highway program, as well as the necessary LCC research, development, and training needs.

Attached for your review and comments is a draft of the Task Force's preliminary study paper, "Life-Cycle Costing and Life-Cycle Cost Analysis: Applications Within FHWA and The Federal-aid Highway Program." We are scheduling a presentation and discussion period of the Task Force's initial effort at the next PMCG meeting. We are seeking PMCG reaction, input and suggestion for improvement necessary to obtain PMCG endorsement of a course of action prior to presenting the task force findings to the Executive Research Review Board on October 22.

We would appreciate receiving your comments by September 28. Mr. Jim Walls has been designated to coordinate this effort and is available to address any questions you may have or clarify any proposals contained in the preliminary study. Mr. Walls can be reached at 366-1339.

Louis M. Papet

PMCG Members:

Lou Papet	HNG-40
Richard Torbik	HEP-10
Tom Pasko	HNR-1
Doug Bernard	HTA-1
Madeline Bloom	HPP-1
Dave McElhaney	HPM-1
John Grimm	HIA-1
W. Mendenhall, Jr.	HRA-06
Byron Lord	HNR-20
Paul Teng	HNR-40
Don Fohs	HNR-30
Ted Ferragut	HTA-20
Dick McComb	HTA-2

Life Cycle Costing and Life Cycle Cost Analysis:

Applications Within

FHWA and The Federal-aid Highway Program

Preliminary Study
August 1992

Task Force Members:
Jim Walls HNG-42 (Pavements)
Byron Lord HNR-20 (Research)
Walt Manning HPP-12 (Policy)
Dennis Miller HIA-10 (Motor Carrier)
Frank Waltos HCP-32 (Contracts and Procurement)

Executive Summary

In response to interest expressed by the FHWA Research and Development Executive Board in Life-Cycle Costing (LCC), the Pavement Management Coordinating Group (PMCG) established an internal LCC Task Force consisting of representatives from the major affected Associate Administrators. The Task Force was specifically charged with developing recommendations on appropriate LCC research needs.

Fundamental to accomplishing its primary tasking, the Task Force had to first identify current and potential FHWA LCC applications along with some fundamental policy implications. The Task Force also looked at the LCC implication of the ISTEA. This paper includes the Task Force's preliminary efforts in this area.

In terms of its specific tasking on LCC research needs, this paper identifies relevant LCC issues and limitations. It lays out research approach options and a plan of action.

Based on its initial efforts, the Task Force proposes two separate but concurrent LCC efforts; an internal LCC policy development effort and a two-phase LCC contract research effort. The policy development effort, although internally directed, would most likely require some outside contractor support.

Under Phase I of the contract research effort, FHWA would contract with several companies to provide inter-disciplinary teams to define and clarify LCC issues and necessary research. Phase I work would include development of detailed work plans that address the identified LCC research needs. Under Phase II, FHWA would continue to fund a more limited number of multi-disciplinary research teams to actually conduct the more promising research activities identified in Phase 1.

The results of this proposed multi-phase research effort and the internal policy development effort would eventually be digested into FHWA guidance on LCC. This final step would most likely be done with in-house staff using consultant support.

The Task Force stresses from the onset that the outputs of life-cycle cost analysis (LCCA) are not decisions in themselves; but rather inputs into the decision making process.

A draft copy of this paper was circulated to the PMCG and discussed at the last July 14 PMCG meeting. The draft paper has been revised to incorporate their views and comments.

The Task Force at this point has not made contact with any of FHWA's partners and/or customers. Consistent with FHWA's outreach program, the Task Force suggests ~~that~~ appropriate outside groups be contacted before research funding decisions are made. Groups such as the American Trucking Association and the Association of American Railroads have conducted research in this area and are likely to have a keen interest in FHWA's efforts. Industry groups such as NAPA, AI, PCA, plus ARTBA would also be interested.

Introduction

A Life-Cycle Costing (LCC) Task Force was formed by Mr. Louis Papet, Chairman of the PMCG, in response to LCC interest expressed by the Research and Development Executive Board at its 1991 - 92 winter meeting. The Task Force is composed of representatives from the Associate Administrators for Policy (HPP-12), Research (HNR-20), Program Development (HNG-42), Motor Carrier (HIA-20), and Administration (HCP-22). Specific Task Force members include:

Jim Walls	HNG-42 (Office of Engineering, Pavements Division)
Byron Lord	HNR-20 (Office of Engineering, Highway Operations Research and Development, Pavements Division)
Walt Manning	HPP-12 (Office of Policy Development, Transportation Studies Division)
Dennis Miller	HIA-10 (Motor Carrier)
Frank Waltos	HCP-32 (Office of Contracts and Procurement Research and Special Programs Division)

The Task Force mission is to develop recommendations for the FHWA Research and Development Executive Board on appropriate ways to incorporate LCC analysis into the Federal-aid highway program, as well as the necessary LCC research, development, and training needs.

This study paper first defines LCC, LCC analysis, and cost effectiveness. It then discusses potential LCC applications with their implications. This discussion is followed by a summary of current policies and a look at new LCC mandates. General LCC technical and policy related issues and limitations are then discussed. In the closing sections, the paper discusses potential approaches to determining and conducting needed research and training necessary to implement LCCA, and finally, the last section presents recommendations on the preferred course of action.

Definitions

Current literature loosely defines life-cycle costing/life-cycle cost analysis as a form of economic analysis which focuses attention on determining the longer term economic implications of alternative strategies rather than merely the initial or front end costs of the immediate decision at hand. It is a tool that can be used to assist in making economically prudent long-term expenditure decisions, i.e., cost-effective investment decisions.

The Task Force believes the terms "life-cycle costing" and "life-cycle cost analysis" are synonymous. However, life-cycle cost analysis is more descriptive of the inherent analytical process and, as a result, the remainder of this paper uses the term life-cycle cost analysis (LCCA).

A related term, cost effectiveness, also has bearing in terms of FHWA Policy. Cost effectiveness is an economic related measure (generally a ratio) that describes how well an alternative meets a performance type objective in relation to the cost of achieving that performance. The cost component of cost-effectiveness measures should generally reflect life-cycle cost. The attractiveness of using cost-effectiveness measures is based on its ability to tie cost to performance. For example, a cost-effective measure in the safety area might be cost/accident reduced. In terms of pavements, it could be cost per ESAL carried until terminal serviceability is reached.

As well as defining what LCCA and cost effectiveness are, it is equally important to define what they are not. The Task Force stresses from the onset that the outputs of life-cycle cost analysis are not decisions in themselves; but rather inputs into the decisionmaking process.

LCC Applications

The Task Force sees two distinct areas where LCCA could be applied within FHWA, i.e., internal and external applications. .The FHWA can use internal applications to support decisionmaking at the national level. External applications are those related to the Federal-aid highway program. Within each area there are multiple application possibilities.

In terms of the Federal-aid highway program, there are several potential decision levels where highway agencies could apply LCCA. These decision levels include but are not necessarily limited to:

State Network Analysis - To evaluate total funding needs and to determine resource allocation levels for the various systems, project categories, or improvement types in relation to established system wide performance goals. The LCCA can also be incorporated into the various management systems required by the ISTEA.

Project Prioritization - To Compare the merits of funding one project in lieu of another.

Pavement Design - To assist in pavement type selection and to evaluate the marginal rate of return for providing premium in lieu of standard pavements.

Materials Specifications - To compare the use of imported premium aggregate versus lower quality, but locally available aggregate.

Total Quality Management - To evaluate the long-term impact of increased attention to quality control. For example, increased expenditure for research and testing equipment may quickly pay for itself.

Operational Analysis - To evaluate catch basin clean out policy, the type and application rates of de-icing chemicals, use of cathodic protection, etc.

Current LCC Policy

Internally, the FHWA already incorporates cost-effective considerations in terms of national level policy development and analysis of alternate investment strategies. The Associate Administrator for Policy incorporates many aspects of life-cycle costing analysis during development of the biennial report to Congress, "Status of the Nations Highway and Bridges." Some LCC principles have been and more will be included in cost allocation studies and in developing and evaluating legislative proposals.

Externally, the FHWA does not specifically require State highway agencies (SHA) to conduct life-cycle costing or economic analysis in support of either program or project level decisions as a precondition for federal-aid funding. This is not true for other US DOT Modal Administrations.

The Federal Transit Administration (FTA) requires development of cost-effectiveness measures based on life-cycle cost analysis in support of grant applications for Section 3 discretionary money. This requirement, called an Alternatives Analysis, must be conducted by applicants at the Draft EIS stage, and the results must be included in the Draft EIS. This Alternatives Analysis requirement has been in place for many years, and the FTA has developed and published specific procedural guidelines on how to conduct it.

In contrast, the FHWA has administered a formula based rather than a discretionary program and has encouraged rather than mandated LCCA in the State and local decisionmaking process affecting Federal-aid highway funds. While FHWA will continue to administer a predominately formula based program, FHWA now administers some discretionary programs. The LCC would appear to have a more substantive roll in discretionary programs.

The FHWA, in its pavement policy, requires SHA's to have a pavement management systems (PMS). In that policy, FHWA defines PMS as a set of tools for finding cost-effective strategies.

At its March 8-10 meeting, the Research and Technology Coordinating Committee developed comments on the FHWA R&T program. Among other comments, the committee noted that, ". . . the lack of attention to life-cycle costs and benefits is a major impediment to the utilization of highway related technologies. Particular effort should be made in the research program to develop novel, user-friendly, and robust methods and tools for life-cycle costing"

ISTEA LCC Provisions

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 specifically addresses LCC under sections 134(f)(12) and 135(c)(20). These sections require that the metropolitan and statewide planning processes incorporate consideration of several factors including "the use of life-cycle costs in the design and engineering of bridges, tunnels, or pavement."

Cost effectiveness is referenced in section 119, "Interstate Maintenance Program." Under subsection 4, it establishes eligibility when a "State can demonstrate . . . that such activities are a cost- effective means . . ."

The ISTEA also addresses LCCA in FTA's Section 3(i) program. The revisions both weaken and strengthen the application of LCC in FTA's Alternative Analysis. While the legislation specifically exempts certain metropolitan areas from Alternatives Analysis requirements, it strengthened the Alternative Analysis requirements in non exempted areas.

One aspect of the ISTEA that presents somewhat of a dilemma for LCCA is the requirement to develop and implement several management systems. While current experience reveals that PMS's can be used to foster systematic decisions based on life-cycle costs, few if any, explicitly incorporate user costs or the time value of money. Most focus on maximizing performance based on fixed budgets. Even in those highway agencies that have PMS's in which budget level and performance impact are directly related, the systems have little to do with ultimate budget decisions.

LCC Analysis Issues

Each LCCA application will, to varying degrees, have its own specific LCC issues. However, some of the more obvious fundamental issues include determining:

(a) the appropriate life cycle and analysis periods
(b) the alternatives that should be included
(c) the performance histories of the alternatives
(d) the cost factors to be included
(e) the actual costs of the various cost factors
(f) the appropriate discount rate

Procedural issues are also a concern. It include concerns over how:

(a) inflation is addressed?
(b) sensitive the results are to the discount rate?
(c) performance history variations are addressed?
(d) Agency Costs and User Costs are incorporated?
(e) SHAs can capture and re-invest user cost savings?

Technical, Policy and Procedural Issues and Limitations

Legitimate Subjective Inputs

Being a form of economic analysis, LCCA has all the strengths, weaknesses, and limitations of traditional economic analysis. Foremost among the weaknesses is the fact that LCCA includes many technical assumptions and policy related positions which directly influence the outcome of such analysis. The assumptions and policy inputs necessary to conduct an analysis can be very subjective and highly susceptible to criticism from all parties impacted by the analysis.

Technical assumptions and policy inputs must be clearly identified along with supporting rational. Rational limits or acceptable ranges should be established for technical inputs and policy related assumptions. Sensitivity analysis should be conducted within the acceptable ranges to evaluate the influence of the parameter being considered.

Alternative Development

Another important LCC issue is assuring consideration of a broad range of alternatives. The LCCA cannot be used to evaluate the economic wisdom of a particular alternative in and of itself. It can only evaluate the relative merits between alternatives. As such, incorporating all viable alternatives is essential. This should include promising new approaches and technology. Unfortunately, estimating the performance lives of alternatives, is at best, both an art and a science even when historical data is available. Untried but promising alternatives inherently incorporate greater risk than the tried and true. This additional risk has to be addressed.

Private industry incorporates risk through the selection of appropriate discount rates. Riskier projects (investments) require prospects of greater (generally 3-5% more) return. The SHA efforts in developing PM Systems and SHRP LTPP research will develop a better understanding of pavement performance relationships and should help in reducing risk.

Performance Equivalency

Implicit in economic analysis is the assumption that performance differences between alternatives can be clearly defined, captured, and reflected in the analytical results. While this is true for some aspects, it is not always the case. All alternatives which have the same "useful life," in terms of either years or loadings, do not necessarily provide equivalent performance over that "useful life."

For example, two competing pavement rehabilitation alternatives with the same pavement life, may very well deteriorate differently. If this is the case, then they will provide different levels of service over their useful lives, even if they reach the same terminal serviceability at the same time (see figure 1).

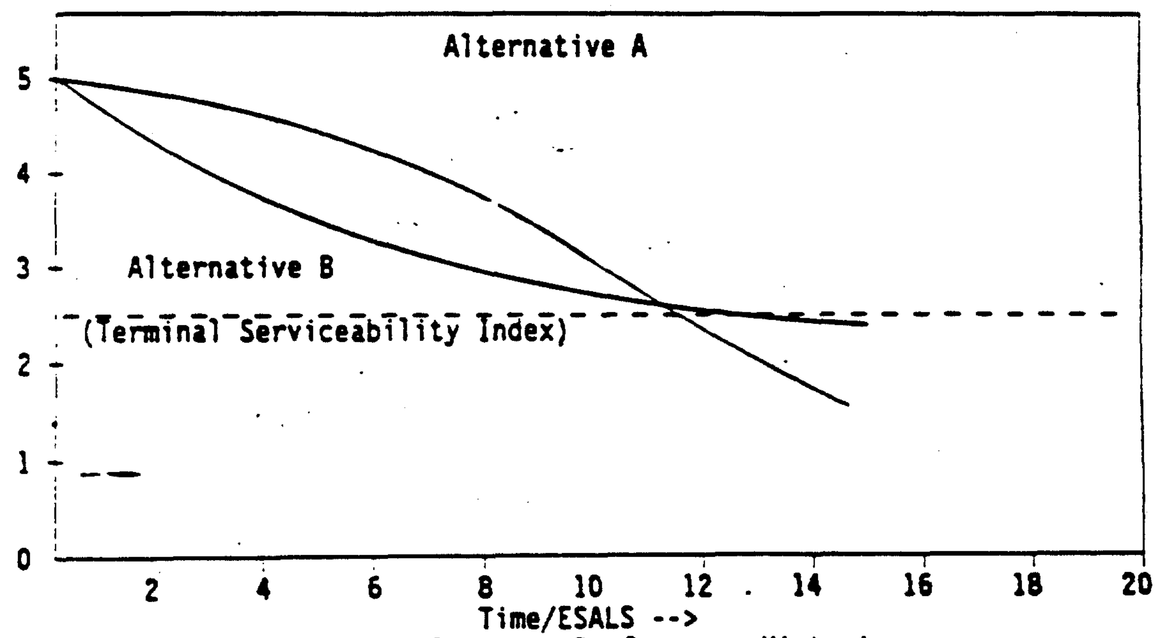

Figure 1 Pavement Performance Histories

Non-costable and Non-quantifiable

In any economic analysis, there are, generally speaking, non- costable and non-quantifiable elements that, none-the-less, need to be considered in the decision making process. The how and the degree to which the non-costable and non-quantifiable elements are addressed is a major issue. While broader scope analysis are more complete, they are not necessarily more accurate.

The degree to which current and future costs and benefits can be accurately estimated severely limit the ability of LCCA to distinguish between of alternatives when LCCA reveals little economic difference. When LCCA results are relatively close (within 10-20% of one another) relative risk and other considerations take on greater significance.

User Costs

Highway user costs, particularly travel time or delay cost, have been controversial. While they may be difficult to quantify and price, construction imposed traffic delays have become, and are likely to continue to be, an ever increasing burden imposed on the public.

Currently, highway agencies have little economic incentive to select alternatives that minimize total (agency plus user) LCC. The alternative with the lowest total life-cycle cost may well be the one that has the lowest user cost but, at the same time, the highest agency cost. Because there are no readily available mechanisms for highway agencies to transform reductions in user costs to additional highway investment capital, the current system encourages highwav agencies to minimize agency rather than total costs. This tends to result significant sub-optimization of total possible benefits.

This issue is addressed to some extent by requiring full maintenance of traffic on heavily traveled routes. Highway agencies are already paying a premium on certain projects for limiting the contractors hours of operation and/or elaborate traffic detours. Highway agencies need to anticipate this trend and incorporate higher future rehabilitation cost in current life-cycle cost analysis.

Marginal Costs

The LCCA is generally used as a means of determining the most economically efficient (some times the cheapest) project from among a set of alternative that adequately meet the minimum performance requirements. This may well be short sighted. Highway agencies need to look at marginal costs, especially when relatively modest total cost increases make significant differences in performance and or service lives. Premium pavements may be economically justif~~ied~~ in areas with no alternative routes for maintenance, rehabilitation, and/or reconstruction activities.

Discount Rate

As a minimum, model LCCA procedures should incorporate the time value of money and discount future cost and benefits to a common time. As just noted, such procedures must include internal (highway agency), as well as external (user) costs associated with a highway facility over its intended useful life. Such procedures, however, would have to provide guidance on how to deal with the highway agency's inability to capture user cost saving for future reinvestment.

Procedures

To be practical, LCCA must be conducted using procedures that recognize the policy issues that influence the analysis and explicitly document the policy positions taken in the analysis. The FHWA does not currently have LCCA procedural guidelines. If the FHWA intends to use LCCA internally, it needs to establish procedures governing such applications. If, on the other hand, FHWA expects to encourage consideration of LCCA in State and local highway agency decisions affecting Federal-aid highway funds, FHWA will need to establish LCCA procedural guidelines. From a technical aspect, model procedures should identify and evaluate all viable alternatives and relevant cost factors. They should incorporate techniques for developing accurate cost, performance, and service lives of identified alternatives.

Alternate Approaches

While the Task Force has been able to identify areas where LCCA research would be productive, it believes a more comprehensive look at the entire process as applies to highway investment decisionmaking is warranted. The Task Force further believes that integration of the many debatable positions into a cohesive position on the application of LCC and appropriate guidelines on the conduct of LCCA within the FHWA program would be much more positive contribution.

The Task Force also looked at developing an in-house working group to review the literature and identify and conduct the needed research. The Task force believes FHWA does not have sufficient manpower in the appropriate multi-disciplinary fields available to make a significant contribution to advancing LCC within FHWA. LCC embraces many complex issues; some are readily apparent, others are more subtle. Prior to more active FHWA involvement, endorsement, or technical support of LCC, FHWA sponsored research is necessary to:

(1) more clearly define, explore, and resolve identified LCC issues;
(2) identify and explore other important LCC issues not currently identified; and
(3) develop a comprehensive approach to incorporate the research findings into integrated procedures for the various LCC applications.

Policy Recommendations

The Task Force recommends that FHWA policy explicitly promote the long-term cost-effective use of Federal funds, both in its internal operations and in the Federal-aid highway program.

The FHWA should continue to use LCCA and cost-effectiveness considerations in its internal operations to evaluate the condition and performance trend of the Nation's highways, and to determine whether or not we are using resources to the maximum advantage in achieving the national transportation goals. Other internal applications could include developing and analyzing highway investment polic developing and evaluating cost allocation studies, and evaluation of competi... IVHS technologies and other R&D activities.

The FHWA should increase its efforts to encourage, support, and implement State and local use of life-cycle cost analysis principles at all decision levels. It should develop model LCC guidelines, building on extensive existing LCCA knowledge base including that of State and local highway agencies. The FHWA should make these LCCA guidelines available to highway agencies and require consideration of LCC in the Urban and Statewide Planning processes. The FHWA should also require the development of LCC and cost-effectiveness information as part of each ISTEA mandated management system.

In response to specific ISTEA LCC requirements, FHWA should focus on program rather than project specific requirements. The FHWA should provide guidance on conducting LCCA, require that it be conducted, and ensure that the results are explicitly considered in the decisionmaking process. It should not become involved in conducting or reviewing/approving actual LCCA's conducted by State and local highway agencies, even on Federal-aid highway program funded projects.

Research Recommendations:

In order to move forward with LCCA, FHWA should initiate research and training, necessary to foster improved LCC analysis at all decision levels.

Because of the financial/economic focus, the research should be conducted by a multi-disciplinary team that draws on the strengths of economists, financial analysts, and other appropriate disciplines, as well as the highway engineering community.

Because of the enormity and complexity of LCCA and the pervasiveness of potential application opportunities, it will be difficult to formulate a comprehensive research work plan with existing in-house resources.

The Task force recommends that FHWA pursue a two-phase LCCA contract research effort as follows:

Phase I - an innovative exploratory research effort.

Phase II - a traditional, in depth, detailed research effort into specific LCCA issue areas identified in phase I.

Phase I - Exploratory Research

The exploratory research phase would require that selected contractor(s) develop an inter-disciplinary team acceptable to FHWA that would;

1. Explore policy issues and the implications of various FHWA courses of action.

2. Identify specific LCC research needs associated with the courses of action identified.
3. Develop a detailed work plan and cost proposal that addresses the specific research needs identified.

Because of the complexity of LCCA, and the relatively inexpensive cost anticipated for the exploratory research, the Task Force believes it would be extremely beneficial (i.e., cost effective from a LCC perspective) to fund multiple research teams for this early stage research. The Task Force envisions awarding multiple contracts under one primary exploratory research contract. The exact number of exploratory research contracts to be funded would be based on the responses received to the request for proposals (RFP).

Phase II - Detailed Research

The Phase II research component is basically designed to carry out the specific research that will be proposed in the detailed work plans developed by the inter-disciplinary teams under Phase I. Upon completion of the Phase I exploratory research, FHWA would evaluate the research team(s) findings and proposed work plans. At that point, FHWA would decide whether to fund of all or part of the research activities identified by one or all the exploratory research contractors. The Task Force envisions the Phase II component would be an option included in the Phase I research contract.

On completion of this proposed two-phase research effort, FHWA will still need to consolidate the various research teams efforts, produce LCCA guidelines, and where necessary, develop LCCA policy, technical advisories, and possibly regulations. The Task Force recommends that the final component would be to establish appropriate training program(s).

With the concurrence of the Research and Development Executive Board, the Task Force will establish a LCCA working group to develop an RFP consistent with the preceding recommendations. Preliminary estimates are that an RFP could be ready for early FY 93 Funding. Funding for the second phase would not be necessary until FY 94.

2. Identify specific LCC research needs associated with the courses of action identified.
3. Develop a detailed work plan and cost proposal that addresses the specific research needs identified.

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Federal Highway Administration

[FHWA Docket No. 94–15]

Life-Cycle Cost Analysis

AGENCY: Federal Highway Administration (FHWA), DOT.

ACTION: Interim policy statement; request for comments.

SUMMARY: This FHWA policy statement on life-cycle cost analysis (LCCA) helps fulfill Federal management responsibilities for analyzing life-cycle cost aspects of infrastructure investment decisions under Executive Order 12893, "Principles of Federal Infrastructure Investment." The policy statement establishes LCCA principles to be applied by FHWA in infrastructure investment analyses, and in evaluating the adequacy of State highway agency procedures used in conducting required LCCA for investments funded through the Federal-aid highway program. States and local agencies are expected to apply these principles in evaluating program and project level investment decisions involving Federal-aid highway funds as required under applicable FHWA regulations. Comments are solicited on potential problems in implementing provisions of this policy statement and specific needs for training and technical assistance in LCCA.

DATES: This interim policy statement is effective on July 11, 1994. Comments on the interim policy statement must be received on or before October 11, 1994. A final LCCA policy statement will be published that takes into consideration comments received on this interim statement.

ADDRESSES: Submit written, signed comments concerning this interim policy statement to FHWA Docket No. 94–15, Federal Highway Administration, room 4232, HCC–10, Office of the Chief Counsel, 400 Seventh Street, SW., Washington D.C. 20590. In addition to specific comments on this policy statement, comments are requested on training and technical assistance needed to implement LCCA. All comments received will be available for examination at the above address between 8:30 a.m. and 3:30 p.m. e.t. Monday through Friday, except legal Federal holidays.

FOR FURTHER INFORMATION CONTACT: Mr. James W. March, Chief, Systems Analysis Branch, (202) 366–9237, or Mr. Steven M. Rochlis, Legislation and Regulations Division, (202) 366–1395, Federal Highway Administration, 400 Seventh Street SW., Washington D.C. 20590.

SUPPLEMENTARY INFORMATION:

Background

There is an increasing recognition that total life-cycle costs of highway and transportation investments must be given greater consideration in all phases of highway programs. Executive Order 12893, "Principles of Federal Infrastructure Investment," requires that benefits and costs of infrastructure investment be measured and appropriately discounted over the full life cycle of each project. Sections 1024 and 1025 of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) (Pub. L. 102–240, 105 Stat. 1914, 1977) also require consideration of "the use of life-cycle cost in the design and engineering of bridges, tunnels, or pavement." Subpart B of the interim final rule on implementation of ISTEA management systems (23 CFR 500.207) requires use of LCCA for pavement management systems (PMS) and Subpart C (23 CFR 500.307) requires use of LCCA or comparable techniques for bridge management systems (BMS).

Life-cycle cost analysis is an economic evaluation of all current and future costs associated with investment alternatives. It is a valuable economic analysis technique for evaluating highway and other transportation programs and projects that require long-term capital and maintenance expenditures over the extended lives of facilities. Future costs are discounted using an appropriate discount rate to compare costs incurred at different points in time.

Life-cycle cost analysis principles and techniques are used in many types of economic analysis to compare benefits and costs arising at different points in time. Benefit-cost analysis and cost effectiveness analysis, for instance, use life-cycle cost analysis principles to discount future benefits and costs of investment alternatives over the lives of alternatives being evaluated.

Life-cycle cost analysis is used to evaluate programs of pavement and bridge improvements as well as individual projects. It is an important input to estimates of future funding requirements and to the development of improvement programs, especially when there are budget constraints.

The use of value engineering is receiving increased attention as a technique for analyzing the functions of a program, project, system, product, or service to identify opportunities to significantly lower costs while still achieving the essential functions. Life-cycle costs are often analyzed to ensure that unnecessary costs are avoided by considering future operations, maintenance, and reconstruction requirements.

Total life-cycle costs of specific facilities may be many times the initial construction costs when user costs are considered. It is essential that a long term perspective be taken in programming improvements, selecting among alternative maintenance, rehabilitation, and reconstruction strategies, and designing pavements, structures, and other highway elements. Longer design lives may have to be considered, and traditional strategies for programming maintenance and rehabilitation activities may have to be reevaluated to determine whether they adequately consider future costs, including user delay-related costs.

Increasing congestion on important highways in urban areas and some rural areas makes it critical to fully consider life-cycle costs of investment decisions. Safety concerns and auxiliary construction costs to maintain, rehabilitate, or reconstruct congested highways and bridges under traffic are very high. User costs and delays around work zones in congested areas may be even higher and represent significant inefficiencies that may adversely affect economic productivity, especially on the National Highway System (NHS). These delays can erode productivity gains realized by the growing number of industries using just-in-time and other advanced logistics strategies that depend on efficient and predictable transportation.

Regardless of whether user costs are included in a formal LCCA, most States already implicitly consider user costs when they choose to pay premiums to maintain traffic through work zones or design more durable pavements in congested urban areas. Including user costs in LCCA makes these implicit considerations explicit, and may help identify other opportunities to reduce overall agency and user costs.

Recognition of the high future costs to maintain and rehabilitate highways, bridges and tunnels, and their associated traffic control, safety, environmental, and hydraulic components has led to increased interest in the potential for LCCA to improve investment productivity and reduce public and private costs of highway and other transportation programs. The FHWA and the American Association of State Highway and Transportation Officials (AASHTO) jointly sponsored a symposium in December 1993 to learn more about LCCA practices among the States and to identify research, training, technical assistance, and policy-related needs to improve LCCA application. An important input to that symposium was an AASHTO survey of State LCCA practices.

Many specific LCCA issues and research needs were identified at the symposium. Key technical issues included how to establish the appropriate analysis period, how to value and properly consider user costs, and how to choose the appropriate discount rate. Participants also identified important research and data needed to predict pavement and bridge performance and forecast future traffic.

An important policy issue raised at the symposium was the recognition that results of LCCA may favor selection of improvements with higher initial costs in order to achieve significant long term savings in overall investment requirements. It may indicate, for instance, that more projects warrant reconstruction rather than rehabilitation strategies, that early intervention with preventive maintenance is cost effective, or that somewhat higher designs or levels of service may be appropriate for some facilities. The FHWA recognizes that LCCA, thus, may result in proposals for greater expenditures up front. At the same time virtually all transportation agencies will continue to face budgetary limitations at least over the short term. Life-cycle cost analysis will help agencies identify and explain the real costs borne by transportation users of inadequate infrastructure funding. Furthermore, LCCA can assist agencies that face fiscal constraints in making the best use of available funds. Several States already use LCCA in developing network improvement programs as part of their pavement and bridge management systems. Eventually it is desirable for all States to have such capabilities.

The following paragraphs highlight key principles of good LCCA practice. Applying these principles generally will allow States and local agencies to identify investment alternatives that will minimize total life-cycle costs. While their use is not mandatory in all instances, States are strongly encouraged to apply these principles in conducting life-cycle cost analyses unless there are unique characteristics of particular programs or projects that require principles to be modified. Life-cycle cost analysis, of course, is only one consideration in many investment decisions, but it certainly is one of the most important for NHS routes and other high volume roads in light of the costs and lost productivity associated with future maintenance and rehabilitation actions.

In general there are no hard and fast rules concerning the appropriate length of the analysis period. The analysis period will vary depending on the type of improvement (bridge, versus tunnel, versus pavement), the location (urban versus rural), the highway system (NHS versus other), and the design lives of all appropriate alternatives. In general, longer design lives should be considered for improvements on the NHS and other high volume urban roadways because future agency and user costs associated with maintenance and rehabilitation activities may be so high. For pavement improvements on the NHS, design lives of 50 years may be reasonable while bridge and tunnel improvements may have design lives of

100 or more years. The consideration of longer design lives will require longer analysis periods in LCCA. Analysis periods for projects involving other modes generally should be long enough to cover the full life-expectancy of the investment—the time until facilities would have to be reconstructed if initially constructed to an optimum design. These lives would vary according to the modal alternative being examined. Analysis periods for all project alternatives should be the same length.

The inclusion of user costs in LCCA is particularly controversial among some States. Part of the controversy over user costs is the fact that they often are many times higher than agency costs and can critically influence decisions. While all motorists do not value costs of delays as highly as do commercial travelers, the costs and lost productivity to businesses of delays around work zones are simply too high to ignore. In fact, such delays arguably have a greater impact on business than delays associated with inadequate capacity because businesses factor normal congestion costs into their plans; but delays around work zones generally cannot be foreseen and thus are more disruptive. Technical advisories to be developed on estimating user operating and delay costs will address this issue in greater detail.

In addition to increased delay and vehicle operating costs, rehabilitation and maintenance activities may result in increased accident costs around work zones. Technical advisories will be developed to assist in estimating increases in accident rates associated with different types of rehabilitation and maintenance activities. The most comprehensive information on the costs of motor vehicle accidents is contained in the National Highway Traffic Safety Administration's publication, "The Economic Cost of Motor Vehicle Crashes, 1990." A copy of this document is available in the public docket for this notice.

The proper use of the discount rate has been an issue for LCCA, cost-benefit analysis and other types of economic analysis as well. Among the issues are the relationship between the discount rate and inflation, factors that affect the choice of rates, and how to establish rates over a long analysis period. Office of Management and Budget (OMB) Circular A–94, "Guidelines and Discount Rate for Benefit-Cost Analysis of Federal Programs," provides guidance on selecting appropriate discount rates for economic analyses. Since the choice of discount rate can affect relative life-cycle costs, sensitivity analysis may be appropriate if two or more alternatives are close in cost, if streams of costs and benefits among alternatives vary significantly over time, or if the discount rate is outside the range of discount rates recommended by OMB.

The FHWA will develop training and technical assistance materials to address issues in LCCA. These materials should supplement guidance on economic analysis techniques contained in AASHTO's 1977 publication, "A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements,"[1] the "Red Book," in the forthcoming update to that publication which was developed under National Cooperative Highway Research Program Project 7–12, and in other guidance on LCCA issues. While additional materials are being developed, this interim policy statement provides guidance on LCCA principles applicable to highway and structure design.

The FHWA is reviewing its policy on alternative bridge designs (53 FR 21637, June 9, 1988) for consistency with this interim life-cycle cost analysis policy as well as with Executive Order 12893.

Policy

The following is FHWA's LCCA policy for infrastructure investment analyses. It represents good practice that should be followed by States and local transportation agencies in making program and project investment decisions:

1. Life-cycle costs are an important consideration in all highway investment decisions.

2. The level of detail in LCCA should be commensurate with the level of investment involved and the types of alternatives being analyzed. Investments on the NHS generally warrant more detailed analysis than investments on non-NHS routes. Similarly, evaluation of decisions whether to reconstruct or rehabilitate a facility warrants more detailed analysis than consideration of alternative maintenance strategies.

3. Typical life-cycle cost analysis profiles may be developed and used as the basis for evaluating alternatives for general types of improvements, such as, consideration of alternative pavement designs or different types of bridges on various functional class highways. Major programs and projects, however, often will require consideration of a broad range of alternative rehabilitation and reconstruction options and more detailed analysis of potential alternatives. The potential applicability and use of LCCA profiles will be discussed in greater detail in future technical advisories.

4. Other factors, including budgetary, environmental, and safety considerations, legitimately influence highway investment decisions and should be considered along with the results of LCCA in evaluating investment alternatives. Life-cycle cost analysis principles should be used in conjunction with other appropriate economic analysis techniques in pavement and bridge management systems. Systemwide or network objectives as well as project level concerns should be considered in decisionmaking, and both levels of analysis should consider life-cycle costs.

5. Analysis periods should be for the life of the facility or system of facilities being evaluated and should account for costs of foreseeable future actions. Analysis periods should not be less than 75 years for major bridge, tunnel, or hydraulic system investments, and not less than 35 years for pavement investments. Longer design lives may be appropriate for the NHS or other major routes or corridors.

6. All appropriate agency costs anticipated during the analysis period should be considered in the analysis, including traffic control costs during maintenance and rehabilitation, costs of special construction procedures required to maintain traffic, and agency operating costs for such things as tunnel lighting and ventilation. In those cases where the agency required to operate a facility is not the one making the investment decision, it is important for the funding agency to include operating costs borne by other organizations responsible for operating the facilities.

7. User costs including increased vehicle operating costs, accident costs, and delay-related costs incurred throughout the analysis period should be considered in LCCA. Increased costs due to deteriorated riding surfaces, circuitous routings, and accidents and delays around and through maintenance and construction work zones are all important.

8. Future agency and user costs should be discounted to net present value or converted to equivalent uniform annual costs using appropriate discount rates. Discount rates selected should be consistent with guidance provided in OMB Circular A–94.

[1] This document is available for inspection as prescribed at 49 CFR Part 7, Appendix D. It may be purchased from the American Association of State Highway and Transportation Officials, 444 N. Capitol Street, NW., Suite 225, Washington DC 20001. A copy also will be available in the public docket for this notice.

Technical advisories on these and other technical issues in the application of LCCA will be issued by FHWA in the future.

Issued on: June 30, 1994.

Rodney E. Slater,

Federal Highway Administrator.

[FR Doc. 94-16719 Filed 7-8-94; 8:45 am]

BILLING CODE 4910-22-P

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: **INFORMATION:** 1991 Intermodal Surface Transportation Efficiency Act (ISTEA) Implementation Interstate Maintenance Program

Date: MAY 21 1992

From: Associate Administrator for Program Development

Reply to Attn of: HNG-13

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

The purpose of this memorandum is to provide written guidance regarding the provisions in the 1991 ISTEA which created the Interstate maintenance (IM) program.

Authorizations - Section 1003

Section 1003(a)(1) establishes the first annual authorizations for the IM program for FY 1992 through FY 1997, in amounts ranging from $2.431 billion to $2.914 billion.

Apportionments - Section 1009

Section 1009 modified Section 104(b)(5)(B) of Title 23, which previously established the apportionment formula for the I-4R program. The formula remains based on the same factors, lane-mile (55 percent) and vehicular miles of travel (45 percent), for apportioning IM funds, but the formula now includes those Interstate routes designated under Sections 103 and 139(c) of Title 23 plus Interstate routes designated under 23 U.S.C., Section 139(a) before March 9, 1984 (except toll roads not subject to a secretarial agreement as provided in Section 105 of the Federal-Aid Highway Act of 1978). Section 104(b)(5)(B) of Title 23 provides that no State shall receive less than one-half percent of the total IM funds apportioned annually.

The certificate of apportionment of FY 1992 funds was transmitted by the FHWA Notice N 4510.264 dated December 18, 1991.

Availability - Section 1020

Section 1020(a) rewrites 23 U.S.C. 118 and provides that IM funds shall remain available for obligation in a State for a period of 3 years after the last day of the fiscal year for which they are authorized. For example, FY 1992 funds were apportioned on December 18, 1991, and will lapse on September 30, 1995, and FY 1993 funds will be apportioned on October 1, 1992, and will lapse on September 30, 1996.

Federal Share - Section 1021

Section 1021(a) provides that the Federal share on all IM projects shall be 90 percent, except as modified in States with sliding scales.

Eligibility - Section 1009

Section 1009(e)(5) amends 23 U.S.C. 119(a) to permit the Secretary to approve IM funded projects for resurfacing, restoring, and rehabilitating routes on the Interstate System designated under Sections 103 and 139(c) of Title 23, and routes designated prior to March 9, 1984, under Section 139(a) and (b) of Title 23.

Section 1009(e)(3) amends Section 119(c) of Title 23 to establish types of work eligible for IM funding. The section has been interpreted to include as eligible, those work items which provide for 3R work on existing features on the Interstate route and its interchanges and grade separations within normal "touchdown limits." For example, the rehabilitation of existing roadside hardware may include IM funding for work such as bringing old guardrail up to current standards, maintenance of impact attenuators, refurbishing existing traffic control signs, pavement markings, and other devices, etc. However, excluded from eligibility for IM funding are all new work elements, such as new interchanges, new ramps, new rest areas, new noise walls, or other work which does not resurface, restore, or rehabilitate an existing element.

Existing bridges (including over crossing structures) may be replaced with IM funds, provided they meet the structurally deficient criteria of the bridge program. Bridges classified as functionally obsolete may also be replaced with IM funding, except that capacity expansion elements should be subject to the limitations discussed in the following paragraphs.

Section 1009(a) prohibits IM funding for the portion of the cost of any project attributable to the expansion of the capacity of any Interstate highway or bridge, except for the addition of high-occupancy vehicle lanes or auxiliary lanes (such as truck climbing lanes).

In determining what portion of a project is eligible for IM funding and what portion is capacity expansion (and, therefore, not eligible for IM funds), the basic purpose of the project should be considered. If the project is a combination of preservation and capacity expansion, the cost should be split with 3R items eligible for IM funding and capacity expansion items eligible for other funds. In determining the split, it may be helpful to visualize the project without the capacity expansion work (added lanes, bridge widening or extension for example) and allow IM funding for all necessary 3R items.

Section 1009(e)(4) amends 23 U.S.C. 119(e) to allow IM funding for preventative maintenance activities, which a State can demonstrate through its pavement management system, are a cost-effective means of extending Interstate pavement life. Preventative maintenance includes activities such as sealing joints and cracks, patching concrete pavement, shoulder repair, and restoration of drainage systems which are found to be cost-effective projects resulting in extending the service life of pavements.

This provision has been extended administratively to allow IM funding for other preventative maintenance activities. Examples may include structure work such as crack sealing, joint repair, seismic retrofit, scour countermeasures, and painting of steel members which are cost-effective in extending the service life of the structure.

Toll Roads, Bridges and Tunnels - Section 1012

Section 1012(d) provides that existing toll agreements entered into under Section 119(e) or 129 of Title 23 prior to and in effect on the date of enactment of the 1991 ISTEA, shall continue in effect. All new agreements must be executed in accordance with the provisions of the 1991 ISTEA. Guidance on the use of Federal-aid funds on toll roads has been provided by Mr. Kane's memorandum of March 12, 1992.

Discretionary Funds

There is no provision for set aside of funds from the IM program for discretionary purposes. Also there is no provision for reallocation of apportioned IM funds which lapse at the end of the availability period.

Section 1020 does provide for a continuation of the I-4R discretionary fund program that is separate and distinct from the IM program. The source of the I-4R discretionary funds is an annual set aside from National Highway System (NHS) funds. These I-4R discretionary funds may be used for IM-type projects or for other improvements on the Interstate including projects to provide additional Interstate capacity. A memorandum was issued on December 20, 1991, which outlined procedures for applying for FY 1992 I-4R discretionary funds. A similar memorandum will be issued annually.

Transferability - Section 1009

Section 1009(e)(5)(D) and (E) modifies 23 U.S.C. 119(f) to allow a State to unconditionally transfer an amount not to exceed 20 percent of its IM apportionment to its apportionments under 23 U.S.C. 104(b)(1) for the NHS, or 23 U.S.C. 104(b)(3) for the Surface Transportation Program (STP).

Section 1009(b) further amends 23 U.S.C. 119(f) to allow a State to transfer an amount in excess of the 20 percent unconditional IM fund transfer, if the State certifies to the Secretary that (1) the sums to be transferred are in excess of its needs for resurfacing, restoration or rehabilitating its Interstate System routes and (2) the State is adequately maintaining the Interstate System, and if the Secretary accepts the certification.

State requests to transfer IM funds should be submitted to the Division Administrator and may be approved by the Regional Federal Highway Administrator. Funds transferred into the STP will be transferred into the State Flexible Appropriation Code 33D.

Adequate Maintenance of the Interstate System

Requirements for the State to certify that it is adequately maintaining the Interstate System and that the Secretary develop criteria for determining what constitutes "adequate maintenance" were added by Section 1009(c)(2).

We anticipate that formal rulemaking may be necessary to allow input from the States in the development of definitive guidance on what constitutes adequate maintenance. Therefore, for the purpose of evaluating State requests to transfer IM funds, in excess of the 20 percent unconditional amount, and until such time as these criteria are established, the guidance contained in the Federal-Aid Policy Guide, CFR 635E and its supplement (old FHPM 6-4-3-1) should be used for determining whether the State is adequately maintaining the Interstate System.

Headquarters Contacts

This guidance will be updated in the future if further clarifications are found necessary. Questions about what constitutes adequate maintenance of the Interstate System should be directed to the Construction and Maintenance Division (HNG-21). Pavement management systems are coordinated by the Pavement Division (HNG-41). Other questions about the IM program should be directed to the Interstate and Program Support Branch (HNG-13).

Anthony R. Kane

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Subject: INFORMATION: Preventive Maintenance

Date: JUL 27 1992

From: Associate Administrator for Program Development

Reply to Attn of: HNG-10

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Section 119 of Title 23, United States Code, was amended by the Intermodal Surface Transportation Efficiency Act of 1991 to provide specific Federal-aid fund eligibility for preventive maintenance on Interstate highways.

We consider preventive maintenance to include roadway activities such as joint repair, pavement patching, shoulder repair, and restoration of drainage systems, and bridge activities such as crack sealing, joint repair, seismic retrofit, scour countermeasures, and painting. Such work is eligible for Federal-aid participation where the work is determined to be cost-effective for preserving the pavement and bridge structure and extending the pavement and bridge life to at least achieve the design life of the facility.

Due to the nature of preventive maintenance type work, the Division Administrator may approve a request to advance this type of project on Interstate highways without including safety or geometric enhancements, but with the understanding that appropriate safety and geometric enhancements will be an integral part of future 3R/4R projects. This approach may also be applied to minor work the Division Administrator considers eligible for Federal-aid funding on other Federal-aid highways. Preventive maintenance or minor work items shall not degrade any existing safety or geometric aspects of the facility.

Anthony R. Kane

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: INFORMATION: Interstate Maintenance Program

Date: June 14, 1993

From: Executive Director

Reply to Attn of: HNG-21

To: Regional Federal Highway Administrators
Division Administrators
Federal Lands Highway Program Administrator

Over the last decade, the State highway agencies have carried out necessary resurfacing, restoration, rehabilitation and reconstruction (4R) of Interstate highways in accordance with the provisions of 23 U.S.C. 119 using funds apportioned under 23 U.S.C. 104(b)(5)(B). Since there was no differentiation in eligibility or pro rata funding for the various classes of work, there was not a need to develop strict definitions for determining whether the proposed work was resurfacing, restoration, rehabilitation or reconstruction. General definitions for pavement reconstruction and pavement rehabilitation (3R) are included in the "Pavement Policy" (23 CFR 626) which was established in 1988.

Currently, some questions pertaining to the definitions for rehabilitation and reconstruction have been raised since Section 1009(e) of the ISTEA of 1991 generally eliminated reconstruction on the Interstate System from eligibility under 23 U.S.C. 119, Interstate Maintenance (IM) Program. As revised, this section promotes maintenance of the Interstate System through approval of projects for resurfacing, restoration and rehabilitation, and through preventive maintenance activities.

Preventive maintenance includes restoration or rehabilitation of specific elements of a highway facility when it can be demonstrated that such activities are a cost-effective means of extending the pavement life. The list of specific work elements which are generally accepted as extending the service life of pavements and bridges is extensive. In general, any work which provides additional pavement structural capacity (general overlays or replacement of portions of the pavement structure), or prevents the intrusion of water into the pavement or pavement base (seal coats, joint seals, crack seals, overlays), or provides for removal of water that is in the pavement or pavement base (underdrains, restoration of drainage systems), restores pavement rideability (profiling, milling), or prevents the deterioration of bridges (cleaning and painting, seismic retrofit, scour countermeasures, deck rehabilitation or repair, deck drain cleaning) are considered to be work which extends the service life of the highway. These typical preventive maintenance work items are not intended to be all inclusive but are rather a limited list of examples. The changes made by Section 1009(e) of the ISTEA of 1991 allow considerable flexibility in determining, based on good engineering analysis, the most cost-effective method of extending the service life of the existing Interstate pavements and bridges.

Each of the States either have, or are in the process of developing pavement, bridge and other management systems in response to the ISTEA of 1991 and previous FHWA policies. One of the purposes of a pavement management system is to identify cost-effective strategies for proposed pavement work. In some cases, the most cost-effective pavement strategy may be removal and replacement of all or part of a badly deteriorated pavement structure. However, if a removal and replacement strategy is considered ineligible for IM funding, a less cost-effective strategy may be selected by the State based only on the class of available funding. Forcing any particular strategy based primarily on availability of funds would not provide the public with the best use of Federal-aid funds. Therefore, in order to provide the States with necessary flexibility and still meet the intent of the revised 23 U.S.C. 119, pavement work which is identified by the State's pavement management system as being cost-effective, including removal and replacement strategies, where no additional capacity is provided is eligible as an IM Program funded project.

Reconstruction on the Interstate System may still be approved; however, unless the proposed work meets the eligibility requirements of 23 U.S.C. 119(c), such work must use funds other than those apportioned under 23 U.S.C. 104(b)(5)(B).

Mr. Anthony R. Kane's May 21, 1992, memorandum on "1991 Intermodal Surface Transportation Efficiency Act (ISTEA) Implementation Interstate Maintenance Program" listed, as examples, several types of improvements which were not eligible for IM funding. The example concerning "new ramps" has created some confusion. As a result, further clarification is necessary.

After reviewing the legislation, we have determined that the addition of new ramps at existing interchanges is properly a part of "interchange reconstruction" and does not constitute added capacity under 23 U.S.C. 119(g). Eligible new ramps may include those associated with reconstruction of existing interchanges necessitated by traffic growth or operational problems. Examples might include the addition of one or more loops to an existing diamond interchange, the addition of a directional ramp to relieve Interstate traffic congestion, or the addition of a ramp or ramps to provide a missing traffic movement. These examples are also not intended to be all inclusive. In general, new ramps associated with the reconstruction of an existing interchange are eligible for IM funding and conversely, new ramps on an Interstate route where there is presently no existing interchange are not eligible for IM funding.

In addition to these comments and guidance concerning pavement and interchange eligibility, any proposals for IM funded projects should include considerations for safety or geometric enhancements in accordance with Mr. Kane's July 27, 1992, memorandum on "Preventive Maintenance."

E. Dean Carlson

McTrans

CATALOG

Mc*Trans* (Center for Microcomputers in Transportation), is a software distribution and user support center, originally established by the Federal Highway Administration (FHWA), and now supported by the Federal Transit Administration (FTA). The **Mc*Trans*** Center provides support to microcomputer users through technical assistance of the software it distributes.

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N **NEW PRODUCTS** since June 94

U **UPDATES** since June 94

Carson City PMS

The Carson City Pavement Management System was developed under an FHWA Rural Technical Assistance Program (RTAP) project.

Road inventory data include street name, segment limits and location, subgrade strengths, lengths, widths and surrounding land uses. Structural information includes presence of curb and gutter, shoulder width, surface and base type, thickness and deflection. The condition survey includes information on ride quality, alligator cracking, ravelling and longitudinal plus traverse cracking as the recorded forms of distress; and acceptable, tolerable and unacceptable listed as the three degrees of severity. The total quantity of each distress and severity combination is recorded for each street segment and deduct values assigned. Traffic survey information includes volumes and classification.

The type and extent of distress determine the rehabilitation strategy alternative. The ride quality, alligator cracking and status of surface ravelling are checked. Then, depending on the traffic index (a measure of truck volume and weights), a maintenance and rehabilitation treatment is recommended. Priorities are assigned based on a cost-benefit ratio determined as a function of cost-per-vehicle-mile. Cost estimates are then applied and listed with the expected life cycle before new treatments are required.

LOS: 3 (from FHWA)

Operating System: IBM PC/MS-DOS 2.1+ (384K and Hard Disk)

Supporting Software: dBASE III+

Product#	Description	Price
CCPMS	Carson City PMS, 7/89	$50
CCPMS.D	Documentation	$10

ELSYM 5

ELSYM 5 is a computerized procedure which models a three-dimensional idealized elastic layered pavement system. It computes the various component stresses, strains, and displacements along with principal values at locations specified by the user, within the layered pavement. This program was developed for the Federal Highway Administration.

LOS: 3 (from FHWA)

Operating System: IBM PC/MS-DOS 2.1+

Product#	Description	Price
ELSYM	ELSYM5, 12/86	$40
ELSYM.D	Documentation	$5

EXPEAR

EXPEAR (EXpert system for Pavements Evaluation And Rehabilitation) is a comprehensive computerized system to assist engineers in evaluating concrete highway pavements, developing feasible rehabilitation alternatives, and predicting the performance and cost effectiveness of the alternatives. In its current state of development it is considered an excellent training tool. Some modifications would be required to make this program suitable for routine use.

A computer program has been developed for each of the three pavement types: Jointed Plain Concrete Pavement (JPCP), Jointed Reinforced Concrete Pavements (JRCP), and Continuously Reinforced Concrete Pavement (CRCP). The current version is EXPEAR 1.4 which possesses the capability to do life-cycle cost analysis and to delay rehabilitation up to five years.

EXPEAR was developed by the University of Illinois at Urbana-Champaign under FHWA administrative funded or Highway Planning and Research funded contracts. Further work to enhance the capabilities of EXPEAR is proposed. A hard disk is recommended both for speed of execution and storage of data files.

EXPEAR comes from Kathleen T. Hall of the University of Illinois. A supplemental document describing the Concrete Pavement Evaluation and Rehabilitation System is also available.

LOS: 3

Operating System: IBM PC/MS-DOS 3.0+

Product#	Description	Price
EXPEAR	EXPEAR, Ver.1.4	$45
EXPEAR.D	Documentation	$20
EXPEAR.DS	Supplemental Document	$25

HDM-III and HDM-PC

HDM-III and HDM-PC (Highway Design and Maintenance Standards Model) is designed to make comparative cost estimates and economic evaluations of different construction and maintenance options, including different time staging strategies, either for a given road section or an entire network. The concept can simply be outlined as: determining costs, adding the set of costs over time and comparing the total cost streams for various maintenance and construction alternatives.

HD-PC includes the core HDM-III model, a facility to input data, a mechanism to use the outputs with Lotus 1-2-3, and a constrained version of the Expenditure Budgeting Model (EBM). If HDM is used with the EBM, it is capable of comparing options under year-to-year budget constraints.

The basic data requirements are the network description, construction options, maintenance standards and unit costs, vehicle characteristics and unit costs, traffic volumes and projections, exogenous benefits and costs, and analysis period and discount rates. The program is distributed exclusively by McTrans under license from the World Bank in Washington, DC.

The HDM-PC comes in two versions: 1) fully supported, which includes free technical assistance and updates and 2) unsupported, which has no support services. Both include the HDM-PC User's Manual and the EBM. The EBM may also be purchased separately (PC only). The mainframe version is only available as fully supported. The main HDM-III documentation (HDM.DV1 and .DV2 below), which describe the model in detail, must be purchased separately.

A French version of HDM III is available from PENDC of Paris or through McTrans. Call for details.

LOS: 1 (Copyright 1988, the World Bank)

Operating System: IBM PC/MS-DOS 2.2+ (640K and Hard Disk) and Mainframe

Product#	Description	Price
HDM	Fully supported HDM-PC, Ver.2.0 (incl. EBM, User's Manual, Volumes 1, 2 and HDM Manager)	$400
EBM	Fully supported version of EBM (incl. User's Manual)	$60
HDM.UPG	Upgrade to supported	$300
HDM.UN	Unsupported HDM-PC (incl. EBM and User's Manual)	$100
EBM.UN	Unsupported version of EBM (incl. User's Manual)	$30
HDM.D	Extra copies of HDM-PC User's Manual	$15
HDM.DV1	HDM model documentation Vol. 1: Description of HDM-III	$20
HDM.DV2	HDM model documentation Vol. 2: User's Manual for HDM-III	$25

HDM Manager

HDM Manager is a user-friendly shell environment for specific customized applications of HDM-III. It stores the input data in an efficient manner, creates all the required HDM-III input files, runs the HDM-III program, collects the results and presents the results in a practical way. It provides a simple but powerful package for learning and using the major concepts of HDM-III.

HDM Manager is designed to be used with the full HDM-III package and documentation, which must be obtained separately. HDM Manager comes from the World Bank and *is included with the fully-supported HDM-III.*

LOS: 3 (Copyright 1993, The World Bank)

Operating System: IBM PC/MS-DOS 3.1+

Product#	Description	Price
HDM.MGR	HDM Manager, Ver.2.0	$15

ILLI-BACK

ILLI-BACK is a closed-form backcalculation procedure for rigid pavements. It is a computerized adaptation of a rigorous, theoretically sound and efficient backcalculation procedure, applicable to two-layer, rigid pavement systems. This method simplifies considerably the effort required in interpreting nondestructive testing (NDT) data. A unique feature of this approach is that in addition to yielding the required backcalculated parameters, it also allows an evaluation of the degree to which the in situ system behaves as idealized by theory, and provides an indication of possible equipment shortcomings when these arise in the field.

The ILLI-BACK backcalculation procedure considers a two-layer system, consisting of a rigid pavement slab resting on an elastic solid (ES) or a dense liquid (DL) foundation. The backcalculation process requires four sensor deflections and utilizes the concept for determining the Area of the deflecting basin.

When ILLI-BACK is executed on a personal computer, execution time per deflection basin permits the interpretation of a vast amount of NDT data in a very reasonable time. The method makes it feasible for the first time to have a practical backcalculation procedure attached to the testing device in the field, providing instant checks on the accuracy of the deflection results generated, while there is still time and opportunity for remedial action. The program supports English and Metric units and runs interactively or in batch mode and is distributed in Copy-Protected format.

LOS: 7 (Copyright 1988, A.M. Ioannides)

Operating System: IBM PC/MS-DOS 2.1+ and math coprocessor

Product#	Description	Price
ILBACK	ILLI-BACK, Ver.2.0	$225

ILLI-PAVE Algorithms

ILLI-PAVE Algorithms is a program based on a set of algorithms that were assembled from ILLI-PAVE, a very large complex finite element program. The algorithms are contained in the program called ILLIALGR in the form of a series of spreadsheets selected from the menus. ILLI-PAVE Algorithms can be used for preliminary design and analysis of flexible pavements. This program was developed for the Federal Highway Administration.

LOS: 3 (from FHWA)

Operating System: IBM PC/MS-DOS 2.1+

Product#	Description	Price
ILLI	ILLI-PAVE, 12/86	$40
ILLI.D	Documentation	$5

JCP-1

JCP-1 (Jointed Concrete Pavement) determines the serviceability and fatigue data for use in rigid pavement design. The design process is an iterative process in which a designer specifies trial structural designs, determines the required inputs, executes the program, analyzes the resulting fatigue and serviceability data, modifies the design, and repeats the procedure. The program will analyze any number of slab thicknesses and provide outputs for each thickness, while holding all other inputs constant.

LOS: 3 (from FHWA)

Operating System: IBM PC/MS-DOS 2.0+

Product#	Description	Price
JCP	Jointed Concrete Pavement-1, 12/86	$45
JCP.D	Documentation	$5

Long Beach PMS

The Long Beach Pavement Management System was also developed under the FHWA Rural Technical Assistance Program (RTAP) project.

The system uses data files for physical information on the sections to be included in the analysis; pavement survey data detailing the condition of the surface; and information on the scoring, treatment and cost estimates for each road segment. Traffic data are incorporated into the analysis in the form of a Traffic Index based on ESAL's. An evaluation system is utilized which rates the sections from the pavement surveys and applies a decision tree to determine initial proposed treatments and their estimated costs. LBPMS analyzes both flexible (asphalt concrete) and rigid (Portland cement concrete) pavement types and produces several intermediate and final reports.

LOS: 3 (From FHWA)

Operating System: IBM PC/MS-DOS 2.1+ (384K and Hard Disk)

Supporting Software: dBASE III+

Product#	Description	Price
LBPMS	Long Beach PMS, 6/89	$40
LBPMS.D	Documentation	$10

MAPCON

MAPCON (Methods for Analyzing Pavement CONdition data) is a comprehensive, but user friendly package for pavement safety, roughness, structural capacity and surface condition analysis. MAPCON includes ELSYM5 and the California FPMS and RPMS (which also are distributed separately) and others. MAPCON provides "paths" to all the individual programs, enabling the user to better analyze the pavement conditions, which can then be made part of a pavement management system.

MAPCON was developed by Pennsylvania State University and ARE, Inc., under contract to FHWA. A hard disk is highly desirable, but not required.

LOS: 3 (from FHWA)

Operating System: IBM PC/MS-DOS 2.0+ (512K)

Product#	Description	Price
MAPCON	MAPCON, 4/87	$100
MAPCON.D	Documentation	$65

MIX

MIX is a menu driven, BASIC program which calculates the specific gravities of aggregates for the design of the asphalt mix and the proportions of each aggregate in the mix. The program is based on the methodology described in the MS-2 Report published by the Asphalt I
stitute. No formal documentation is availabl

LOS: 5 (from University of Puerto Rico)

Operating System: IBM PC/MS-DOS 2.0+

Supporting Software: BASIC

Product#	Description	P
MIX	MIX, 1/80	

MODULUS and PASELS

MODULUS and PASELS are two programs
assess the current condition of the moduli o
various structural layers of existing asphalt
pavement. The moduli values are often obtained through nondestructive testing with
use of falling weight deflectometers. The h
volume data collection capabilities of mode
nondestructive testing equipment require a
analysis method which is capable of rapid
backcalculation of pavement layer moduli
production mode of data reduction. A laye
elastic method, MODULUS, was develope
microcomputer use which is very fast in op
tion and provides consistently reliable resu
Random errors in the measurements and sy
tematic errors in the backcalculation proce
may be reduced–the former by repeating th
measurements and the latter by using a mi
computer expert system, PASELS, to provi
consistently acceptable layer moduli value

These programs were developed under a
tional Cooperative Highway Research Pro
gram project, the results of which are pub
lished as NCHRP Report 327, "Determini
Asphaltic Concrete Pavement Structural P
erties by Nondestructive Testing." This r
which contains user's manuals for both p
grams, may be obtained through the Tran
tation Research Board, Washington, D.C.

LOS: 3

Operating System: IBM PC/MS-DOS 2.0+

Product#	Description
MODUL	MODULUS, Ver.4.0
PASEL	PASELS, Ver.1.0

NULOAD

NULOAD is a computerized procedure th
evaluates the effect of legal load limit cha
on the (set of 12) life cycle costs of flexibl
rigid, and/or composite pavements. Data
are interactively input through NULDIN,
user-friendly processor for NULOAD. Co
erable input data is required.

LOS: 3 (from FHWA)

Operating System: IBM PC/MS-DOS 2.0+

Product#	Description
NULOAD	NULOAD, 12/86
NULOAD.D	Documentation

PAVECHEK

Pavechek is a software package for designing interlocking concrete pavements. The structural design of flexible interlocking concrete pavements can be accomplished quickly on this menu-driven, PC computer based program. Pavement cross section designs can be generated for both new or overlay interlocking concrete pavements with unbound or bound base materials. Various levels of sophistication can be used in the program depending on the level of detail of input data available. The design rationale is based on the widely used 1986 AASHTO "Guide for the Design of Pavement Structures".

LOS: 7

Operating System: IBM PC/MS-DOS 2.1+ (Graphics)

Product#	Description	Price
PAVECHEK	Pavechek, Ver.1.0	$55

Pavement Management Forecasting Model

Pavement Management Forecasting Model (PMF) is a Lotus 1-2-3 template for use in planning roadway maintenance and strategies. It runs in a Lotus, Release 2 environment and is completely menu driven. Data on road maintenance and construction unit costs, pavement deterioration rates, future funding estimates and current road conditions are required. Based upon three repair strategies, output is generated in tabular summaries and graphic plots. It allows changes at any level to iterate to desired results.

Agencies responsible for roadway maintenance related funding decisions will find it useful to compare various alternatives. The Lotus design is included in the appendix for users who might modify the algorithms to customized applications. PMF was donated by Mr. William Massicott of the Metropolitan Area Planning Council, Boston.

LOS: 3

Operating System: IBM PC/MS-DOS 2.0+

Supporting Software: Lotus 1-2-3

Product #	Description	Price
PMF	PMF, Ver. 1.0	$40
PMF.D	Documentation	$15

Pavement Management System

Pavement Management System (PMS) is a decision support tool used to assist management responsible for allocating pavement maintenance resources. In a simple view, PMS is a process where information about the pavement system is collected, stored, analyzed and reported.

This third generation, Version 3.0, combines a life cycle approach to pavement maintenance with a user-friendly, mouse or keyboard driven graphical user interface. This standard system includes five modules for analyzing inventory, history, pavement condition, cost and budget, and a knowledge-based ranking system. It uses a maintenance priority ranking system based upon the data collected and stored in the other four modules. In addition, the system's modular design allows the integration with other software to provide enhanced graphical reports and system performance feedback.

LOS: 7 (Copyright 1992, Resource International, Inc.)

Operation System:IBM PC/MS-DOS 3.0+

Product#	Description	Price
PMS	PMS	$695
PMS.GIS	PMS GIS version	$2,500

PMSPro

PMSPro is a pavement management program written in the Microsoft Windows environment using FoxPro for Windows. The program allows the user to completely customize the program by defining decision trees, rehabilitation strategies, deterioration curves, deduct curves, and costs for different pavement types, functional classes, and traffic classes. PMSPro also contains other methods of calculating condition scores such as: WADOT PSC, FAA PCI, PAVER PCI.

PMSPro evaluates a street network both at the project level and the network level. At the Project Level, condition scores are used to prioritize streets. Decision trees evaluate the type and amount of distress to select an appropriate rehabilitation strategy. PMSPro can evaluate all street segments or only those that have changed since the last analysis.

A complete cost accounting package allows costs to be adjusted according to the type and amount of distress as well as other costs such as flagging and engineering.

At the Network Level, a simplified decision process uses future calculated condition scores to select an appropriate rehabilitation strategy and cost. The analysis period can range from 5 to 80 years. Evaluate by functional class or traffic class. Carry unspent funds forward. Prioritize by Worst First or Last.

PMSPro also can handle condition surveys or ditches, sidewalks, street signs and other street accessories. A maintenance module allows the tracking of past maintenance and costs.

Compatible with most GIS programs, including MapInfo from MapInfo, Inc. A GIS program can display pavement condition, recommended rehabilitation strategies, pavement types, sign inventory, etc. by connecting the databases to a map.

LOS: 7 (Copyright 1992©1994, Pavement Engineers, Inc.)

Operating Software: IBM PC/MS-DOS 3.0+

Product #	Description	Price
PMSPRO	PMSPRO Pavement Management Program Ver. 5.2	$1,000

Road Manager™

The Road Manager™ is a modular roadway management system. Its unique features are the ability to include ALL roadway features in the evaluation of a road section, a modular design, user defined parameters allowing extensive customization to fit local conditions and policies, and a modern software design using light bar menus, a complete help system and pick lists for easy data entry.

The General Roadway module serves as the "control center" for all other modules, recording road lengths, widths, classifications, etc., as well as overall condition indices for eight different types of roadway features. The General Roadway module can also be used as a stand alone system, suitable for "windshield survey" evaluation of a road network. *The General Roadway module is required for all other modules.*

The Asphalt Pavement, Roadway Drainage and Roadway Utility modules allow the detailed inventory and evaluation of roadway distresses, drainage needs and utility related features. These modules include a user definable decision table that determines recommended repairs or maintenance. All calculations related to determining a condition index, recommended repairs and estimated costs can be modified by the user.

The Improvement Plan module uses information generated in the Asphalt Pavement, Roadway Drainage and Roadway Utility modules to develop lists of recommended improvements as well as required budgets to attain a given network condition level. The computer-generated plan for improvements can be overridden by the user. The estimated deterioration curves used by the system in projecting future pavement and utility patch condition can also be modified.

The Repair History module serves as an electronic file cabinet, recording all work performed on a road section as it is completed. The Street Diagram module graphically displays and prints all Drainage and Utility features that have been inventoried through their respective modules.

LOS: 7 (Copyright 1989, The Info Center, Inc.)

Operating System: IBM PC/MS-DOS 3.0+ (640K and Hard Disk)

Product#	Description	Price
RMRD	General Roadway, Ver. 1.51	$495
RMAS	Asphalt Pavement, Ver. 1.51	$995
RMGR	Gravel Road, Ver. 1.51	$495

CHAPTER 3

RIGID PAVEMENT

3.1 TA 5040.30, Concrete Pavement Joints, November 30, 1990.

3.2 The Benefits of Using Dowel Bars, Technical Paper 89-03, May 17, 1989.

3.3 Preformed Compression Seals, Technical Paper 89-04, September 11, 1989.

3.4 Reinforcing Steel for JRCP (Cores form Kansas I-70), July 25, 1989.

3.5 Dowel Bar Inserters, February 23, 1996.
- Dowel Bar Inserters, March 6, 1990.
- Dowel Bar Placement: Mechanical Insertion versus Basket Assemblies, February 1989.

3.6 TA 5080.14, Continuously Reinforced Concrete Pavement, June 5, 1990.
- Modification to TA 5080.14, August 29, 1990.

3.7 Case Study, CRCP, June 22, 1987.

3.8 Lateral Load Distribution and Use of PCC Extended Pavement Slabs for Reduced Fatigue, June 16, 1989.

3.9 Longitudinal Cracking at Transverse Joints of New Jointed Portland Cement Concrete (PCC) Pavement with PCC Shoulders, November 30, 1988.

3.10 TA 5080.17, Portland Concrete Cement Mix Design and Field Control, July 14, 1994.

3.11 Summary of State Highway Practices on Rigid Pavement Joints and their Performance, May 19, 1987.

3.12 Bondbreakers for Portland Cement Concrete Pavement with Lean Concrete Bases, June 13, 1988.

U.S. Department
of Transportation
Federal Highway
Administration

Technical Advisory

Subject

CONCRETE PAVEMENT JOINTS

Classification Code	Date
T 5040.30	November 30, 1990

1. PURPOSE. To provide guidance and recommendations relating to the design and construction of joints in jointed portland cement concrete pavements.

2. CANCELLATION. Technical Advisory T 5140.18, Rigid Pavement Joints, dated December 15, 1980, is canceled.

3. BACKGROUND

a. The performance of concrete pavements depends to a large extent upon the satisfactory performance of the joints. Most jointed concrete pavement failures can be attributed to failures at the joint, as opposed to inadequate structural capacity. Distresses that may result from joint failure include faulting, pumping, spalling, corner breaks, blow-ups, and mid-panel cracking. Characteristics that contribute to satisfactory joint performance, such as adequate load transfer and proper concrete consolidation, have been identified through research and field experience. The incorporation of these characteristics into the design, construction, and maintenance of concrete pavements should result in joints capable of performing satisfactorily over the life of the pavement. Regardless of the joint sealant material used, periodic resealing will be required to ensure satisfactory joint performance throughout the life of the pavement. Satisfactory joint performance also depends on appropriate pavement design standards, quality construction materials, and good construction and maintenance procedures.

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b. The most common types of pavement joints, which are defined by their function, are as follows:

(1) Transverse Contraction Joint - a sawed, formed, or tooled groove in a concrete slab that creates a weakened vertical plane. It regulates the location of the cracking caused by dimensional changes in the slab, and is by far the most common type of joint in concrete pavements.

(2) Longitudinal Joint - a joint between two slabs which allows slab warping without appreciable separation or cracking of the slabs.

(3) Construction Joint - a joint between slabs that results when concrete is placed at different times. This type of joint can be further broken down into transverse and longitudinal joints.

(4) Expansion Joint - a joint placed at a specific location to allow the pavement to expand without damaging adjacent structures or the pavement itself.

4. TRANSVERSE CONTRACTION JOINTS. The primary purpose of transverse contraction joints is to control the cracking that results from the tensile and bending stresses in concrete slabs caused by the cement hydration process, traffic loadings, and the environment. Because these joints are so numerous, their performance significantly impacts pavement performance. A distressed joint typically exhibits faulting and/or spalling. Poor joint performance frequently leads to further distresses such as corner breaks, blow-ups, and mid-panel cracks. Such cracks may themselves begin to function as joints and develop similar distresses. The performance of transverse contraction joints is related to three major factors:

a. Joint Spacing. Joint spacing varies throughout the country because of considerations of initial costs, type of slab (reinforced or plain), type of load transfer, and local conditions. Design considerations should include: the effect of longitudinal slab movement on sealant and load transfer performance; the maximum slab length which will not develop transverse cracks in a plain concrete pavement; the amount of cracking which can be tolerated in a jointed reinforced concrete pavement; and the use of random joint spacings.

(1) The amount of longitudinal slab movement that a joint experiences is primarily a function of joint spacing and temperature changes. Expansion characteristics of the aggregates used in the concrete and the friction between the bottom of the slab and the base also have an effect on slab movement.

(a) Joint movement can be estimated by the following equation:

$$\Delta L = CL(\alpha \Delta T + \epsilon)$$

where:

ΔL = the expected change in slab length, in inches.
C = the base/slab frictional restraint factor (0.65 for stabilized bases, 0.8 for granular bases).
L = the slab length, in inches.
α = the PCC coefficient of thermal expansion (see Table 1 for typical values).
ΔT = the maximum temperature range (generally the temperature of the concrete at the time of placement minus the average daily minimum temperature in January, in °F).
ϵ = the shrinkage coefficient of concrete (see Table 2 for typical values). This factor should be omitted on rehabilitation projects, as shrinkage is no longer a factor.

TABLE 1. TYPICAL VALUES FOR PCC COEFFICIENT OF THERMAL EXPANSION (α) [1]

Type of Coarse Aggregate	PCC Coeff. of Thermal Expansion (10^{-6}/°F)
Quartz	6.6
Sandstone	6.5
Gravel	6.0
Granite	5.3
Basalt	4.8
Limestone	3.8

(b) While the above equation can be used to estimate anticipated joint movements, it may be worthwhile to physically measure joint movements in existing pavements. These measurements could provide the designer with more realistic design inputs.

TABLE 2. TYPICAL VALUES FOR PCC COEFFICIENT OF SHRINKAGE (ϵ) [1]

Indirect Tensile Strength (psi)	PCC Coeff. of Shrinkage (in./in.)
300 (or less)	0.0008
400	0.0006
500	0.00045
600	0.0003
700 (or greater)	0.0002

(2) For plain concrete slabs, a maximum joint spacing of 15 feet is recommended. Longer slabs frequently develop transverse cracks. It is recognized that in certain areas, joint spacings greater than 15 feet have performed satisfactorily. The importance of taking local experience into account when selecting joint spacing (and designing pavements in general) cannot be overstated. Studies have shown that pavement thickness, base stiffness, and climate also affect the maximum anticipated joint spacing beyond which transverse cracking can be expected. Research indicates that there is a general relationship between the ratio of slab length (L) to the radius of relative stiffness (ℓ) and the amount of transverse cracking [2]. This research shows that there is an increase in transverse cracking when the ratio L/ℓ exceeds 5.0. Further discussion is provided in Attachment 1.

(3) For reinforced concrete slabs, a maximum joint spacing of 30 feet is recommended. Longer slab lengths have a greater tendency to develop working mid-panel cracks caused by the rupture of the steel reinforcement. Studies have also shown that, as the joint spacing increases above 30 feet, the rate of faulting increases and joint sealant performance decreases [4].

(4) Random joint spacings have been successfully used in plain undoweled pavements to minimize resonant vehicle responses. When using random joint spacings, the longest slab should be no greater than 15 feet, to reduce the potential for transverse cracking. Some States are successfully using a spacing of 12'-15'-13'-14'. Large differences in slab lengths should be avoided.

(5) While they do not affect joint spacing, skewed joints have been used in plain pavements to provide a smoother ride. A skew of 2 feet in 12 feet is recommended, with the skew placed so that the inside wheel crosses the joint ahead of the outside wheel. Only one wheel crosses the joint at a time, which minimizes vehicle response and decreases stresses within the slab. Skewed joints are most commonly used when load transfer devices are not present. While skewed joints may be used in conjunction with load transfer devices, studies have not substantiated that skewing doweled joints improves pavement performance and are not recommended. Dowels in skewed joints must be placed parallel to the roadway and not perpendicular to the joints.

b. Load Transfer across the joint. Loads applied by traffic must be effectively transferred from one slab to the next in order to minimize vertical deflections at the joint. Reduced deflections decrease the potential for pumping of the base/subbase material and faulting. The two principal methods used to develop load transfer across a joint are: aggregate interlock; and load transfer devices, such as dowel bars. It is recommended that dowel bars be used.

(1) Aggregate Interlock. Aggregate interlock is achieved through shearing friction at the irregular faces of the crack that forms beneath the saw cut. Climate, and aggregate hardness have an impact on load transfer efficiency. It can be improved by using aggregate that is large, angular, and durable. Stabilized bases have also been shown to improve load transfer efficiency [14]. However, the efficiency of aggregate interlock decreases rapidly with increased crack width and the frequent application of heavy loads to the point that pavement performance may be effected. Therefore, it is recommended that aggregate interlock for load transfer be considered only on local roads and streets which carry a low volume of heavy trucks.

(2) Dowel Bars. Dowel bars should be used on all routes carrying more than a low volume of heavy trucks. The purpose of dowels is to transfer loads across a joint without restricting joint movement due to thermal contraction and expansion of the concrete. Studies have shown that larger dowels are more effective in transferring

loads and in reducing faulting. It is recommended that the minimum dowel diameter be D/8, where D is the thickness of the pavement. However, the dowel diameter should not be less than 1¼ inches. It is also recommended that 18-inch long dowels be used at 12-inch spacings. Dowels should be placed mid-depth in the slab. Dowels should be corrosion-resistant to prevent dowel seizure, which causes the joint to lock up. Epoxy-coated and stainless steel dowels have been shown to adequately prevent corrosion.

c. Joint Shape and Sealant Properties

(1) The purpose of a joint sealant is to deter the entry of water and incompressible material into the joint and the pavement structure. It is recognized that it is not possible to construct and maintain a watertight joint. However, the sealant should be capable of minimizing the amount of water that enters the pavement structure, thus reducing moisture-related distresses such as pumping and faulting. Incompressibles should be kept out of the joint. These incompressibles prevent the joint from closing normally during slab expansion and lead to spalling and blow-ups.

(2) Sealant behavior has a significant influence on joint performance. High-type sealant materials, such as silicone and preformed compression seals, are recommended for sealing all contraction, longitudinal, and construction joints. While these materials are more expensive, they provide a better seal and a longer service life. Careful attention should be given to the manufacturer's recommended installation procedures. Joint preparation and sealant installation are very important to the successful performance of the joint. It is therefore strongly recommended that particular attention be given to both the construction of the joint and installation of the sealant material.

(3) When using silicone sealants, a minimum shape factor (ratio of sealant depth to width) of 1:2 is recommended. The maximum shape factor should not exceed 1:1. For best results, the minimum width of the sealant should be 3/8-inch. The surface of the sealant should be recessed 1/4- to 3/8-inch below the pavement surface to prevent abrasion caused by traffic. The use of a backer rod is necessary to provide the proper shape factor and to prevent the sealant from bonding to the bottom of the joint reservoir. This backer rod should be a closed-cell polyurethane foam rod having a diameter approximately 25 percent greater than the width of the joint to ensure a tight fit.

(4) When using preformed compression seals, the joint should be designed so that the seal will be in 20 to 50 percent compression at all times. The surface of the seal should be recessed 1/8- to 3/8-inch to protect it from traffic. Additional information can be obtained from FHWA Technical Paper 89-04, "Preformed Compression Seals" [5] for PCC pavement joints."

5. LONGITUDINAL JOINTS

a. Longitudinal joints are used to relieve warping stresses and are generally needed when slab widths exceed 15 feet. Widths up to and including 15 feet have performed satisfactorily without a longitudinal joint, although there is the possibility of some longitudinal cracking. Longitudinal joints should coincide with pavement lane lines whenever possible, to improve traffic operations. The paint stripe on widened lanes should be at 12 feet and the use of a rumble strip on the widened section is recommended.

b. Load transfer at longitudinal joints is achieved through aggregate interlock. Longitudinal joints should be tied with tiebars to prevent lane separation and/or faulting. The tiebars should be mechanically inserted and placed at mid-depth. When using Grade 40 steel, 5/8-inch by 30-inch or 1/2-inch by 24-inch tiebars should be used. When using Grade 60 steel, 5/8-inch by 40-inch or 1/2-inch by 32-inch tiebars should be used. These lengths are necessary to develop the allowable working strength of the tiebar. Tiebar spacing will vary with the thickness of the pavement and the distance from the joint to the nearest free edge. Recommended tiebar spacings are provided in Table 3.

c. Tiebars should not be placed within 15 inches of transverse joints. When using tiebars longer than 32 inches with skewed joints, tiebars should not be placed within 18 inches of the transverse joints.

d. The use of corrosion-resistant tiebars is recommended, as corrosion can reduce the structural adequacy of tiebars.

e. It is recommended that longitudinal joints be sawed and sealed to deter the infiltration of surface water into the pavement structure. A 3/8-inch wide by 1-inch deep sealant reservoir should be sufficient.

TABLE 3. MAXIMUM RECOMMENDED TIEBAR SPACINGS (In.)

Note : 48" maximum spacing recommended.

BAR SIZE		# 4 BAR										# 5 BAR									
GRADE STEEL		GRADE 40					GRADE 60					GRADE 40					GRADE 60				
DIST TO FREE EDGE (ft.)		10	12	16	22	24	10	12	16	22	24	10	12	16	22	24	10	12	16	22	24
PVMT THICKNESS	TYPE OF JOINT																				
9"	Warp	37	31	23	17	16	48	47	35	25	23	48	48	36	26	24	48	48	48	40	36
	Butt	26	22	16	12	11	40	34	25	18	16	42	35	26	19	17	48	48	39	29	26
10"	Warp	34	28	22	16	14	48	42	32	23	20	48	44	33	24	22	48	48	48	36	32
	Butt	24	20	16	11	10	36	30	23	16	14	38	31	24	17	16	48	47	35	26	23
11"	Warp	31	25	20	15	13	47	38	29	21	19	48	40	30	22	20	48	48	44	32	30
	Butt	22	18	14	11	9	34	27	21	15	14	34	29	21	16	14	48	43	31	23	21
12"	Warp	28	23	18	13	12	42	35	27	19	18	44	36	28	20	18	48	48	41	30	28
	Butt	20	16	13	9	9	30	25	19	14	13	31	26	20	14	13	47	39	29	21	20

Warp joint: a sawed or construction joint with a keyway

Butt joint: a construction joint with no keyway

6. CONSTRUCTION JOINTS

a. Transverse Construction Joints

(1) Transverse construction joints should normally replace a planned contraction joint. However, they should not be skewed, as satisfactory concrete placement and consolidation are difficult to obtain. Transverse construction joints should be doweled as described in paragraph 4b(2) and butted, as opposed to keyed. Keyed transverse joints tend to spall and are not recommended.

(2) It is recommended that transverse construction joints be sawed and sealed. The reservoir dimensions should be the same as those used for the transverse contraction joints.

b. Longitudinal Construction Joints

(1) The decision to use keyed longitudinal construction joints should be given careful consideration. The top of the slab above the keyway frequently fails in shear. For this reason, it is recommended that keyways not be used when the pavement thickness is less than 10 inches. In these cases, the tiebars should be designed to carry the load transfer.

(2) When the pavement thickness is 10 inches or more, a keyway may be used to provide the necessary load transfer. If a keyway is to be used, the recommended dimensions are shown in Figure 1. Keyways larger than the one shown may reduce the concrete shear strength at the joint and result in joint failures. The keyway should be located at mid-depth of the slab to ensure maximum strength. Tiebars are necessary when using keyways. Consideration should be given to deleting the keyway and increasing the size and/or number of tiebars. The additional steel cost may be more than offset by the potential savings in initial labor and future maintenance costs.

(3) Tiebars should not be placed within 15 inches of transverse joints. When using tiebars longer than 32 inches with skewed joints, tiebars should not be placed within 18 inches of the transverse joints.

(4) It is essential that the tiebars be firmly anchored in the concrete. Tiebars should be either mechanically inserted into the plastic concrete or installed as a two-part threaded tiebar and splice coupler system. It is recommended that periodic pull-out tests be conducted to ensure the tiebars are securely anchored in the concrete. Attachment 2 describes a recommended testing procedure for tiebars.

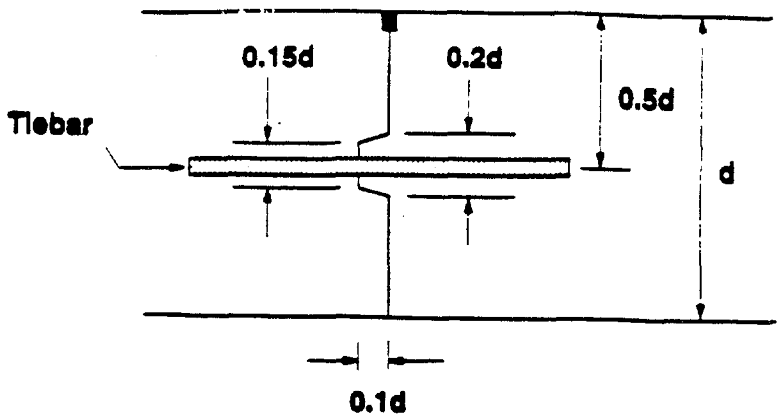


Figure 1. Recommended Keyway Dimensions

(5) Bending of tiebars is not encouraged. Where bending of the tiebars would be necessary, it is recommended that a two-part threaded tiebar and splice coupler system be used in lieu of tiebars. If tiebars must be bent and later straightened during construction, Grade 40 steel should be used, as it better tolerates the bending. It may be necessary to reapply a corrosion-resistant coating to the tiebars after they have been straightened. When pull-out tests are performed, they should be conducted after the tiebars have been straightened.

(6) It is recommended that longitudinal construction joints be sawed and sealed. The reservoir dimensions should be the same as those used for the longitudinal joints.

7. <u>EXPANSION JOINTS</u>

a. Good design and maintenance of contraction joints have virtually eliminated the need for expansion joints, except at fixed objects such as structures. When expansion joints are used, the pavement moves to close the unrestrained expansion joint over a period of a few years. As this happens, several of the adjoining contraction joints may open, effectively destroying their seals and aggregate interlock.

b. The width of an expansion joint is typically 3/4-inch or more. Filler material is commonly placed 3/4- to 1-inch below the slab surface to allow space for sealing material. Smooth dowels are the most widely used method of transferring load across expansion joints. Expansion joint dowels are specially fabricated with a cap on one end of each dowel that creates a void in the slab to accommodate the dowel as the adjacent slab closes the expansion joint, as shown in Figure 2.

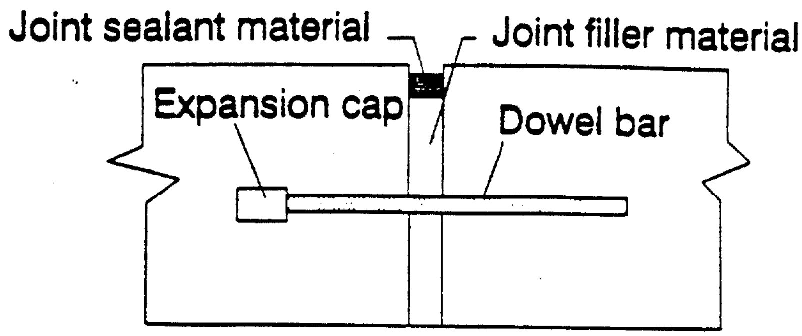


Figure 2. Expansion Joint Detail

c. Pressure relief joints are intended to serve the same purpose as expansion joints, except that they are installed after initial construction to relieve pressure against structures and to alleviate potential pavement blowups. Pressure relief joints are not recommended for routine installations. However, they may be appropriate to relieve imminent structure damage or under conditions where excessive compressive stresses exist. Additional information can be obtained from the FHWA Pavement Rehabilitation Manual, Chapter 9.

8. JOINT CONSTRUCTION

a. Concrete Placement

(1) A prepaving conference should be considered on all major paving projects. This conference should include the project engineer and the paving contractor and should discuss methods for accomplishing all phases of the paving operation. The need for attention to detail cannot be overstated.

(2) When using dowel baskets, the baskets should be checked prior to placing the concrete to ensure that the dowels are properly aligned and that the dowel basket is securely anchored in the base. It is recommended that dowel baskets be secured to the base with steel stakes having a minimum diameter of 0.3-inch. These stakes should be embedded into the base a minimum depth of 4 inches for stabilized dense bases, 6 inches for treated permeable bases, and 10 inches for untreated permeable bases, aggregate bases, or natural subgrade. A minimum of 8 stakes per basket is recommended. All temporary spacer wires extending across the joint should be removed from the basket. Securing the steel stakes to

the top of the dowel basket, as opposed to the bottom, should stabilize the dowel basket once these spacer wires are removed.

(3) Dowels should be lightly coated with grease or other substance over their entire length to prevent bonding of the dowel to the concrete. This coating may be eliminated in the vicinity of the welded end if the dowel is to be coated prior to being welded to the basket. The traditional practice of coating only one-half of the dowel has frequently resulted in problems, primarily caused by insufficient greasing and/or dowel misalignment. The dowel must be free to slide in the concrete so that the two pavement slabs move independently, thus preventing excessive pavement stresses. Only a thin coating should be used, as a thick coating may result in large voids in the concrete around the dowels.

(4) The placement of concrete at construction joints is particularly critical. Therefore, care must be taken to ensure that only quality concrete is used in their construction; i.e., do not use the first concrete down the chute, nor the "roll" from the screed to construct this type of joint. The concrete used to construct these joints should be the same as for the remainder of the slab. The practice of modifying the mix at the joints is not recommended.

(5) Careful and sufficient consolidation of the concrete in the area of the joints is essential to good joint performance. Load transfer across a doweled joint is greatly affected by the quality of concrete consolidation around the dowels. Consolidation also has a direct relationship to concrete strength and durability. Concrete strength, in turn, has a significant effect on the amount of spalling that occurs at the joint.

(6) The placement of dowels should be carefully verified soon after paving begins. If specified tolerances are not being achieved, then an evaluation of the dowel installation, concrete mix design, and placement techniques must be made. Appropriate corrections should be made to the paving process to ensure proper alignment of the load transfer devices.

(7) When paving full-depth full-width, a mechanical prespreader and finishing machine in the paving train can be used to reduce drag and shear forces on the dowels.

(8) In cases where separate concrete placement is made adjacent to previously placed concrete, i.e., truck climbing lanes or concrete shoulders being placed after mainline pavement, it

is important that incompressibles do not enter the previously sawed transverse joint reservoir or crack that typically forms below the transverse joint reservoir. It is recommended that backer rod, tape, or other material be placed on the vertical face of the transverse joint at the edge of the pavement to prevent mortar from intruding into the existing joint. Failure to keep incompressibles out could prevent the joint from closing normally during slab expansion and may lead to delaminations near the edge of the previously placed concrete.

b. Sawing

(1) It is recommended that all joints be sawed. The sawing of transverse contraction and longitudinal joints should be a two-phase operation. The initial sawing is intended to cause the pavement to crack at the intended joint. It should be made to the required depth, as described later, with a 1/8-inch wide blade. The second sawing provides the necessary shape factor for the sealant material. This second sawcut can be made any time prior to the sealant installation. However, the later the sealant reservoir is made, the better the condition of the joint face. Both sawcuts should be periodically checked to ensure proper depth, as saw blades tend to wear, as well as ride up when hard aggregate is encountered. Periodic measurement of blade diameter is an excellent method to monitor random blade wear, particularly when using gang saws.

(2) Time of initial sawing, both in the transverse and longitudinal directions, is critical in preventing uncontrolled shrinkage cracking. It is very important that sawing begin as soon as the concrete is strong enough to both support the sawing equipment and to prevent raveling during the sawing operation. All joints should be sawed within 12 hours of concrete placement. The sawing of concrete constructed on stabilized base must be sawed earlier. This is particularly critical during hot weather. Once sawing begins, it should be a continuous operation and should only be stopped if raveling begins to occur.

(3) For transverse contraction joints, an initial sawcut of D/3 is recommended, particularly for pavements with a thickness greater than 10 inches. In no case should the sawcut depth be less than D/4. Transverse contraction joints should be initially sawed in succession. Skip sawing is not recommended, as this practice results in a wide range of crack widths that form beneath the sawed joints. These varied crack widths affect the shape factors and may cause excessive sealant stresses in those joints initially sawed.

The dimensions of the final sawing should be dependent upon the sealant type and the anticipated longitudinal slab movement.

(4) For longitudinal joints, a minimum initial sawcut depth of D/3 is recommended to ensure cracking at the joint. The maximum sawcut depth should be such that the tiebars are not damaged. A final sawing that provides a 3/8-inch wide by 1-inch deep sealant reservoir should be sufficient.

(5) When a lengthy period is anticipated between the initial sawing of the joint and the final sawing and sealing, consideration should be given to filling the joint with a temporary filler. This filler material should keep incompressibles out of the joint and reduce the potential for spalling.

(6) The use of plastic inserts is not recommended. Although a few States have had success with these inserts, most States no longer allow their use. Improper placement of plastic inserts has been identified as a cause of random longitudinal cracking [2]. It is also very difficult to seal the joint formed by these inserts.

Anthony R. Kane
Associate Administrator for Program Development

Attachments

DESIGN OF LENGTH

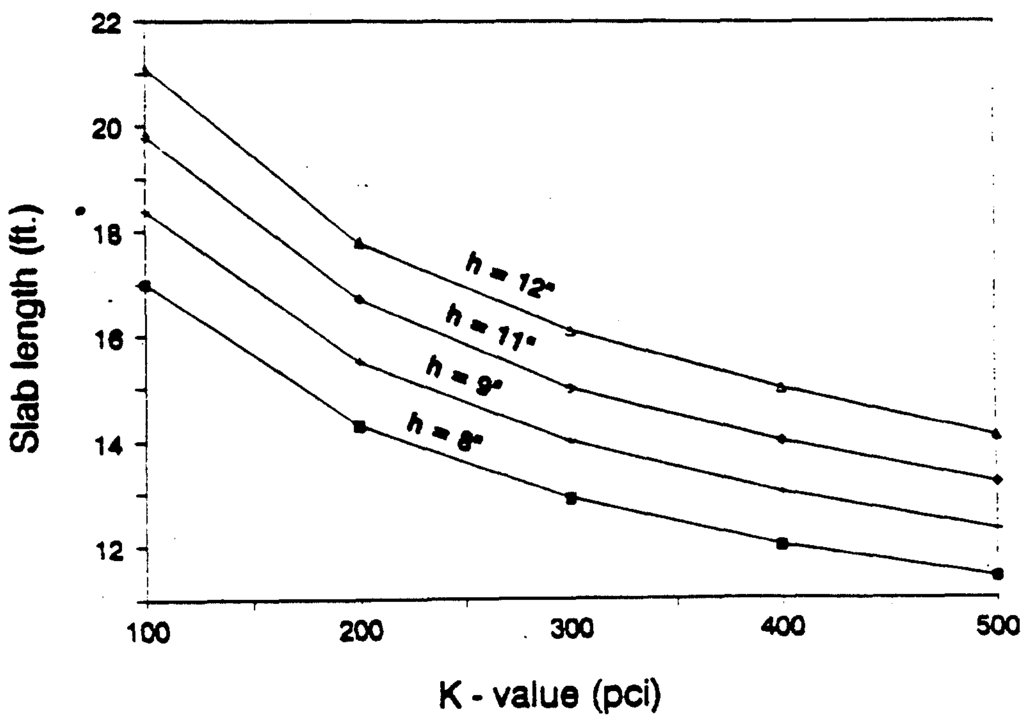
Studies have shown that pavement thickness, base stiffness, and climate affect the maximum anticipated joint spacing beyond which transverse cracking can be expected [2]. Research indicates that there is a general relationship between the ratio of slab length (L) to the radius of relative stiffness (ℓ) and transverse cracking. The radius of relative stiffness is a term defined by Westergaard to quantify the relationship between the stiffness of the foundation and the flexural stiffness of the slab. The radius of relative stiffness has a lineal dimension and is determined by the following equation:

$$\ell = [Eh^3/12k(1-\mu^2)]^{0.25}$$

where

- ℓ = radius of relative stiffness (in.)
- E = concrete modulus of elasticity (psi.)
- h = pavement thickness (in.)
- μ = Poisson's ratio of the pavement
- k = modulus of subgrade reaction (pci.)

Research data indicates that there is an increase in transverse cracking when the ratio L/ℓ exceeds 5.0. Using the criteria of a maximum L/ℓ ratio of 5.0, the allowable joint spacing would increase with increased slab thickness, but decrease with increased (stiffer) foundation support conditions. The relationship between slab length, slab thickness, and foundation support for a L/ℓ ratio of 5.0 is shown below.



TIEBAR PULL-OUT TESTS

Proper consolidation of the concrete around the tiebars is essential to the performance of longitudinal construction joints. Adjacent lanes should not be constructed until the project engineer has had opportunity to test the pull-out resistance of the tiebars. Acceptance of the tiebars should be based on the results of the tests for resistance to pull-out. The project engineer will select 15 tiebars from the first day's placement, after the concrete has attained a flexural strength of 550 psi. The tiebars will be tested to 12,000 lbs. or to a slippage of 1/32-inch, whichever occurs first. The average of the results of these pull-out tests, divided by the spacing of the tiebars, will be used to determine the pull-out resistance in lbs. per linear foot.

If the test results on the first day's placement are well within the test requirements shown below, additional testing will be at the discretion of the project engineer and will be based on comparison of the installation methods and spacings of the first day's placement with subsequent placements.

If the results of the pull-out tests are less than the minimum requirements specified for the width of concrete being tied, the contractor shall install additional tiebars to provide the minimum average pull-out resistance required, as directed by the project engineer. Testing of the supplemental tiebars will be at the discretion of the Engineer.

Tiebars shall be installed by methods and procedures such that the tiebars will develop the minimum average pull-out resistance specified without any slippage exceeding 1/32-inch in accordance with the following table:

Tied Width of Pavement (Distance from Joint Being Constructed to Nearest Free Edge).	Average Pull-out Resistance of Tiebars, lbs./L.F. of joint, minimum.
12 feet or less	2200
Over 12 feet to 17 feet	3200
Over 17 feet to 24 feet	4500
Over 24 feet to 28 feet	5200
Over 28 feet to 36 feet	6800
Over 36 feet	9000

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U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: Technical Paper - The Benefits of Using Dowel Bars

Date: MAY 17 1989

From: Director, Office of Highway Operations
Washington, D.C. 20590

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Direct Federal Program Administrator

Attached for your use are two copies of a technical paper on the benefits of using dowel bars in PCC pavements. This paper suggests that dowel bars should be used on all pavements, except possibly those with low truck volumes. The paper also points out the need for proper installation if the benefits of the dowels are to be realized. Many States are experiencing premature deterioration of their undoweled PCC pavements. In these cases, we encourage the field offices to work with the States in evaluating the merits of dowel bars.

We suggest that a copy of this paper be forwarded to each division office. We appreciate the efforts of the regional offices in reviewing the draft of this paper. If you have any questions concerning this paper, or wish to offer information relating to recent field experience with the installation of dowel bars, please contact Mr. David Law at FTS 366-1341.

N. Van Ness

Norman J. Van Ness

TECHNICAL PAPER 89-03 -- Benefits of Using Dowel Bars

Over the past few years, Pavement Division personnel have reviewed several sections of undoweled PCC pavements. In general, these pavements have experienced a level of deterioration due to faulting that is significantly greater than that found in comparable sections of doweled PCC pavements. This finding has led to a concern over the design and construction of undoweled pavement sections. The purpose of this brief paper is to illustrate the benefits of using dowels on jointed PCC pavements, particularly on those routes carrying a large number of trucks.

For jointed PCC pavements to perform satisfactorily, traffic loads must be effectively transferred from one slab to the next. Without adequate load transfer, the pavement is subjected to a variety of distresses, such as pumping, faulting, and corner breaks. There is considerable disagreement on how load transfer should be obtained. One school of thought is to rely on aggregate interlock in combination with short joint spacings, skewed joints, and stabilized subbases. The other school of thought is to rely on load transfer devices, such as dowel bars.

Aggregate interlock is ineffective at crack widths greater than 0.035 inch. A smaller crack width, generally 0.025 inch, is considered necessary for satisfactory long-term performance of undoweled pavements. An Iowa DOT study(1) of undoweled pavements concluded that "from measurements of joint openings it appears doubtful that aggregate interlock is maintained even by joints spaced at 20 ft." When measured beneath the sawed portion of the joint, over 90% of the joints had crack widths in excess of 0.06 inch. In order to limit crack widths to 0.035 inch over a temperature range of 60-80 Fahrenheit degrees, joint spacings in the range of 6 to 11 feet are needed. Such a spacing is not considered practical. Properly sized dowels, on the other hand, provide effective load transfer at reasonable joint spacings. Maximum joint spacings of 15-20 ft. and 30-40 ft. are recommended for plain and reinforced pavements respectively.

The use of dowels has been shown to reduce faulting. A Florida DOT study(2) concluded that "doweled contraction joints fault less than non-doweled contraction joints." A Georgia DOT study(3) of a project on I-85 found that "dowel bars were effective in reducing the faulting at the contraction joints." The Wisconsin DOT conducted a condition survey of their Interstate system. One of the findings of this study(4) was that "building nonreinforced concrete pavements with additional thickness (2-3 inches) in lieu of using positive load transfer devices (dowel bars) at transverse contraction joints is not successful in preventing or reducing joint faulting to an acceptable level during a pavement's life." Faulting at the joints was notably absent during the AASHO Road Test. One transverse joint faulted seriously, but investigation showed that the joint had been accidently sawed at some distance beyond the end of the dowel. Over the two-year test period, there were no other cases of measurable faulting at the joints, all of which were doweled. Based on road tests performed at the NARDO track in Italy, the XVIII World Road Congress reported that dowels significantly increased the pavement service life[5]

Dowels reduce deflections at the joint, which in turn reduce the magnitude of concrete flexural stresses. These deflections and stresses are reduced due to the load being more effectively shared with the adjoining slab through shear and bending stresses in the dowel itself. Reduced concrete flexural stresses increase the fatigue life of the pavement and thus extend its service life. A theoretical analysis indicates that a 10" doweled slab with 80% load transfer will have the same deflection as a 12" undoweled slab with only 40% load transfer. Dowels can also reduce the potential for premature failure due to corner breaking caused by loss of subgrade support through pumping.

When dowels are properly designed and installed, they can reduce faulting and increase the pavement's service life. When they are not, dowels can cause premature failure of the pavement in the vicinity of the joint. Dowels too small in diameter to handle the necessary stresses have resulted in premature joint failures. Excessive concrete bearing stresses have crushed the concrete around the dowels and allowed faulting to occur. Considerable research has been performed recently which supports the use of larger bars (1-1/4 to 1-1/2 inch). The AASHTO **Guide for Design of Pavement Structures** recommends a dowel diameter of 1/8th the pavement thickness, with the dowels placed near the center of the slab to minimize bending stresses. Most States are currently using, as a minimum, 1-1/4 inch diameter bars, with good results. Most States are using dowels 18 inches long spaced at 12 inches and are reporting no problems.

Dowels should also be corrosion-resistant. The use of epoxy-coated or stainless-steel dowels has been shown to provide the necessary resistance to corrosion. It is important that a bond-breaker be applied to the dowels to allow the slabs to freely and independently expand and contract without developing restraint forces. This bond-breaker should be applied to provide a thin but uniform coating.

Many of the past performance problems associated with doweled joints were the result of excessive joint spacings, ranging from 60 to 100 ft. The trend to plain doweled slabs with joint spacings of 15 to 20 ft. eliminates many of these problems. Shorter joint spacings result in smaller crack widths, which reduce the stresses acting on the dowels. The shorter spacing also reduces slab movement, which makes dowel alignment less critical, as the restraint forces due to misalignment are directly proportional to the amount of slab movement. In addition, the effect of two slabs acting as one as the result of a locked or "frozen" joint is not as severe for the shorter slab lengths.

In order to perform satisfactorily, dowels must also be reasonably aligned. The prevailing practice is to specify dowel alignment tolerances on the order of 1 percent, or roughly 1/8-inch per foot. This frequently results in a high percentage of the dowels being "out of specs" and gives the impression that obtaining proper dowel alignment is very difficult. Studies[6][7] suggest that the alignment tolerances can be relaxed. FHWA now recommends an alignment tolerance of 1/4 inch per foot and will be evaluating the possibility that these tolerances can be further relaxed. It is recognized that the problems with misaligned dowels are generally the result of gross misalignment occurring during concrete placement. Equipment to precisely measure compliance with alignment tolerances after concrete placement are not readily available. However, it is recommended that the completed pavement joints be inspected using a metal detector to verify that no significant dowel misalignment has occurred.

The use of mechanical dowel bar inserters holds promise for the improved installation of dowel bars. Two manufacturers, Guntert-Zimmerman and Gomaco, have developed and are marketing new automatic inserters. It is their claim that these inserters are capable of placing dowels more efficiently and at less cost than basket assemblies without sacrificing placement accuracy.

Construction Technology Laboratories was recently retained to monitor the results of the placement of dowels using these new machines. The Guntert-Zimmerman inserter was evaluated on projects in Texas and Wisconsin and the Gomaco inserter was evaluated on an Idaho project. The placement of dowels using basket assemblies was also monitored in Texas and Wisconsin. Preliminary findings indicate that the inserters placed the dowels with approximately the same accuracy as dowels placed using basket assemblies. Cost figures from the Wisconsin study indicate that a savings of approximately $ 0.35 per sq. yd. of concrete pavement was obtained by using the dowel implanter in lieu of dowel baskets.

The use of dowels is strongly encouraged on all pavements except possibly those with very low truck volumes. Dowels can provide a higher serviceability level over a longer period of time than pavements relying only on aggregate interlock for load transfer. Dowels can minimize pavement distress caused by overloads or heavier loads travelling by permit.

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TECHNICAL PAPER 89-04 -- PREFORMED COMPRESSION SEALS FOR PCC PAVEMENT JOINTS

Joint sealants for jointed concrete pavements are intended to prevent, or at least deter, the intrusion of water and incompressibles into the joint and pavement structure. Water entering a joint can lead to pumping and faulting, while incompressibles in a joint can cause spalling and blow-ups. A joint sealant must be capable of remaining in firm contact with the concrete at the faces of the joint while withstanding repeated expansion and contraction of the pavement slabs due to thermal variations. There are two types of joint sealants which are currently recognized as having the potential for satisfactory long-term performance. These are the preformed compression seals and the low-modulus silicones. The purpose of this technical paper is to discuss key factors in the design and installation of the preformed compression seals.

DESIGN: Preformed compression seals should be designed so that the sealant will be in compression at all times. These seals are typically manufactured from a neoprene compound and factory molded into a web design. The seal is compressed and inserted into the pavement joint. These compressed webs exert an outward force which keeps the seal tightly pressed against the joint faces, thus effectively sealing the joint. As long as these seals are in compression, they will generally be effective. If compression is lost, they will fail. It is essential to maintain a good uniform seal between the joint faces and the compression seal.

Generally, compression seals function best when compressed between 20 percent and 50 percent of their nominal width. This range will vary slightly with manufacturer and the seal dimensions. Compressive forces less than 20 percent may not be sufficient to hold the sealant in place. If the seal is exposed to compressive forces greater than 50 percent for an extended period of time, it may undergo a compression set. Compression set occurs when the seal doesn't recover to its initial position. Once it undergoes compression set, the seal will not expand as the joint opens, resulting in a total loss of compression and joint sealant failure.

When designing joints using compression seals, the anticipated joint movement, the uncompressed width of the compression seal, and the joint width must all be determined. The first step is to determine the anticipated joint movement, using the following equation:

$$\Delta L = CL * (\alpha \Delta T + \epsilon)$$

where:

ΔL is the anticipated amount of joint movement.

C is the slab/subbase frictional restraint factor (0.65 for stabilized subbases, 0.8 for granular subbases).
L is the joint spacing, in inches.
α is the PCC coefficient of thermal expansion (4.5-6.5 $\times 10^{-6}$). This coefficient is primarily affected by aggregate type. It generally ranges from $5\times10^{-6}/^{\circ}F$ for the carbonaceous aggregate mixes to $6\times10^{-6}/^{\circ}F$ for the siliceous aggregate mixes. For further information, see Reference #1, pages 151-152.
ΔT is the maximum anticipated temperature range, generally the mean maximum daily temperature for the hottest month minus the mean minimum daily temperature for the coldest month.
ϵ is the shrinkage coefficient of concrete (0.5-2.5 $\times 10^{-4}$). This factor is ignored on rehabilitation projects, as drying shrinkage has already taken place.

The second step is to select the uncompressed width of the compression seal. The sealant manufacturers have information available which should be used in this selection process. If ΔL is the amount of joint movement and W is the width of the uncompressed seal, then $\Delta L \div W$ should be less than or equal to the allowable movement of the compression seal. This range of allowable movement should be obtained from the sealant manufacturer and typically varies from 50 percent maximum compression to 20 percent minimum compression. If $\Delta L \div W$ is too large, then either the amount of joint movement should be reduced by decreasing the joint spacing or the width of the compression seal should be increased. The width of the uncompressed seal can be determined from the following equation:

$$W \geq \Delta L \div (C_{max} - C_{min})$$

where:

W is the width of the uncompressed seal.
ΔL is the anticipated amount of joint movement.
C_{max} is the maximum recommended compression of the seal, as a decimal (typically 0.5).
C_{min} is the minimum recommended compression of the seal, as a decimal (typically 0.2).

The final step is to select the joint width, based on the width of the compression seal and the anticipated temperature of the pavement at the time of sealant installation. (This need only be a rough estimate.) An approximate installation temperature is necessary so that the compression seal can be installed at the proper compression. Warmer installation temperatures necessitate greater initial compression, as the pavement is closer to its maximum expansion. This will allow the seal to remain sufficiently compressed during cold weather. Conversely, cooler temperatures require a lower initial compression, as the pavement is nearer its maximum contraction. This will prevent the seal from undergoing excessive compressive forces during hot weather. The width of the joint sawcut can be determined from the following equation:

$$Sc = (1 - Pc) * W$$

where:

Sc is the width of the joint sawcut.
Pc is the percent compression of seal at installation, expressed as a decimal.
W is the width of the uncompressed seal.

$$Pc = C_{min} + \left[\frac{\text{Install temp - Min temp}}{\text{Maximum temp - Min temp}}\right] * (C_{max} - C_{min})$$

It should be pointed out that this procedure is approximate; saw blades and compression seals are only available in a limited number of widths. This design procedure is not dependent upon precise temperature predictions and minute variations in joint widths.

Since the pavement temperature at the time of seal installation is not known at the design phase, it is recommended that the design be flexible enough to allow for installation of compression seals over a wide range of temperatures. This can best be achieved by reducing the joint spacing, preferably to 30 feet or less, as shorter joint spacings significantly reduce the amount of joint movement. Selecting a compression seal one or two sizes larger than the minimum required by $\Delta L+W$ will reduce the sensitivity of the design to the installation temperature. Regardless, it may still be necessary to either vary the joint width to account for the pavement temperature at the time of seal installation or to prohibit the installation of compression seals during certain temperatures (i.e., less than 45°F).

Differential vertical movements at the joint also affect seal performance. The greater the vertical movement, the greater the potential that the seal will "walk" up and out of the joint. Doweled joints will reduce vertical movements and are recommended when using compression seals.

Compression seals should not be used within 100 feet of expansion joints. Joints near expansion joints can be expected to expand sufficiently to allow these seals to loosen and pop out.

INSTALLATION: Proper construction techniques must be followed to ensure that the compression seals will perform as intended. Improper installation procedures are a primary cause of premature failures of these seals. Close attention must also be paid to the manufacturer's recommendations.

The joint faces must be vertical, so that the seal does not work itself up and out of the joint. Any spalls at the joint should be patched prior to installation of the compression seal. (Spalls less than 1/4-inch may remain; however, the seal should be recessed sufficiently to avoid the spalled area.) Irregularities in the joint width could reduce the pressure on the seal to the point that it would no longer remain in compression.

It is recommended that the concrete surfaces at the joint be dry prior to installation of the compression seal. The joint should be air-blasted to remove any debris. Both the air temperature and the temperature of the pavement should be above freezing. Prior to installing the compression seal, a lubricant-adhesive should be applied to either the joint faces or the seal. This material primarily serves as a lubricant to facilitate the installation process. This material also cures to form a weak adhesive, which helps keep the seal at the proper height. However, it does not provide any tensile strength.

The compression seal should be adequately recessed, so that it won't be damaged by traffic. The joint edge may be beveled to reduce spalling. A 1/4-inch radius bevel or a 1/8-inch straight bevel is sufficient. The compression seal should be recessed approximately 1/8-inch below the bottom of the bevel. When the joint edge is not beveled, the seal should be recessed from 1/8-inch to 3/8-inch beneath the top of the slab. The seal may be recessed up to 1/2-inch if grinding of the concrete pavement is anticipated in the future. While this additional depth should prevent the seal from being damaged by the grinding operation, it may allow incompressibles to accumulate and cause spalling. The joint reservoir should be deep enough to allow the seal to be compressed without extruding to an elevation where it will be exposed to traffic.

Care should be taken to not stretch the seal during the installation process. A stretched seal will not perform as well or as long as a properly installed seal. The seal should also not be twisted, as intimate contact must be maintained between the seal and joint faces over the full length of the seal. Most compression seal manufacturers have developed installation equipment which do not stretch, twist or damage the seals. This type of equipment should be used.

When sealing a width of two lanes or less, splices should not be permitted. When sealing more than two lanes, one splice may be permitted; however, the contractor should closely follow the manufacturer's recommendations for splicing.

Close inspection of the installation procedure is necessary to ensure that the seals will perform as intended. The inspector should verify that the pavement joint is sawed to the proper width and depth, and that the compression seal is the correct width prior to commencing sealing. The tolerance for the joint width should be ± 1/16-inch. During the installation process, the inspector should verify that the seal is not being stretched. This can be done by comparing the distance between two marks on the surface of the seal measured before and after installation. The inspector should also visually inspect the compression seal to ensure that it has not been twisted or damaged, and is adequately recessed.

SUMMARY: If properly designed and installed, preformed compression seals have the potential to provide excellent performance over an extended period. It is not uncommon to find compression seals more than 10 years old still performing as well as newly installed seals. To ensure this type of performance, both the width of the pavement joint and the pavement temperature at the time of installation need to be coordinated with the width of the compression seal. The joint should be designed and constructed so that the compression seal will function entirely within the manufacturer's recommended operating range, generally 20 percent to 50 percent compression of the uncompressed seal width. This may necessitate that seal installation be prohibited during certain extremes in pavement temperature. Satisfactory joint sealant performance is dependent upon good construction procedures and proper inspection.

SAMPLE DESIGN CALCULATIONS: The design of compression seals is a simple procedure. The following examples show how a design process could be used.

EXAMPLE A:

A State wants to use preformed compression seals on all new PCC pavement projects. Their standard design calls for a 60-foot joint spacing on a granular base. The expected temperature range is from 14°F to 90°F.

Step 1 - Determine the anticipated joint movement.

$$\Delta L = CL\ (\alpha \Delta T + \epsilon)$$
$$= (0.8)(60 * 12)[(5.5*10^{-6})(76) + (1.0*10^{-4})]$$
$$= 576\ (4.18*10^{-4} + 1.0*10^{-4})$$
$$= 0.3 \text{ inch}$$

Step 2 - Select width of uncompressed seal.

(This particular sealant manufacturer recommends an operating range of 55 percent to 20 percent)

$$W \geq \Delta L \div (C_{max} - C_{min})$$
$$\geq 0.3 \div (0.55 - 0.20)$$
$$\geq 0.3 + 0.35$$
$$\geq 0.86 \text{ inch}$$
use $W = 1$ inch

Step 3 - Select width of sawcut.

$$Pc = C_{min} + \left[\frac{\text{Install temp - Min temp}}{\text{Maximum temp - Min temp}}\right] * (C_{max} - C_{min})$$

$$Sc = (1 - Pc) * W$$

Case 1: Installation temperature = 80°F

$$Pc = 0.2 + [(66 \div 76) * (0.55 - 0.2)]$$
$$= 0.2 + (0.87) * (0.35)$$
$$= 0.50$$

$$Sc = (1 - 0.5) * 1$$
$$= 0.5 \text{ inch}$$

Case 2: Installation temperature = 40°F

$$Pc = 0.2 + [(26 \div 76) * (0.55 - 0.2)]$$
$$= 0.2 + (0.34) * (0.35)$$
$$= 0.32$$

$$Sc = (1 - 0.32) * 1$$
$$= 0.68 \text{ inch}$$

EXAMPLE B:

The State elects to change their design by reducing the joint spacing to 30 feet.

Step 1 - Determine the anticipated joint movement.

ΔL is directly proportional to L; decreasing L by 50 percent decreases ΔL by 50 percent.
ΔL = 0.15 inch

Step 2 - Select width of uncompressed seal.

$$
\begin{aligned}
W &\geq \Delta L \div (C_{max} - C_{min}) \\
&\geq 0.15 \div (0.55 - 0.20) \\
&\geq 0.15 \div 0.35 \\
&\geq 0.43 \text{ inch}
\end{aligned}
$$

use W = 0.688 inch (a larger size than necessary)

Step 3 - Select width of sawcut.

$$Pc = C_{min} + \left[\frac{\text{Install temp - Min temp}}{\text{Maximum temp - Min temp}}\right] * (C_{max} - C_{min})$$

$$Sc = (1 - Pc) * W$$

Case 1: Installation temperature = 80°F

$$
\begin{aligned}
Pc &= 0.2 + [(66 \div 76) * (0.55 - 0.2)] \\
&= 0.2 + (0.87) * (0.35) \\
&= 0.50
\end{aligned}
$$

$$
\begin{aligned}
Sc &= (1 - 0.5) * 0.688 \\
&= 0.344 \text{ inch}
\end{aligned}
$$

Case 2: Installation temperature = 40°F

$$
\begin{aligned}
Pc &= 0.2 + [(26 \div 76) * (0.55 - 0.2)] \\
&= 0.2 + (0.34) * (0.35) \\
&= 0.32
\end{aligned}
$$

$$
\begin{aligned}
Sc &= (1 - 0.32) * 0.688 \\
&= 0.47 \text{ inch}
\end{aligned}
$$

Because a larger seal was used, would a sawcut width of 0.375 inch work regardless of the temperature at installation?

Case 1: Insufficient compression

(Seal width - Max. joint opening (JO_{max})) ÷ Seal width = C_{min}

$(0.688 - JO_{max}) \div 0.688 = 0.2$
$JO_{max} = 0.688 - (0.2) (0.688)$
$JO_{max} = 0.55$ inch

$JO_{max} - Sc$ = allowable movement
0.55 - 0.375 = allowable movement
0.175 inch = allowable movement

0.175 inch $\geq$ 0.15 inch (anticipated joint movement)
The seal will not be undercompressed.

Case 2: Compression set

(Seal width - Min. joint opening (JO_{min})) ÷ Seal width = C_{max}

$(0.688 - JO_{min}) \div 0.688 = 0.55$
$JO_{min} = 0.688 - (0.55) (0.688)$
$JO_{min} = 0.31$ inch

$Sc - JO_{min}$ = allowable movement
0.375 - 0.31 = allowable movement
0.065 inch = allowable movement

0.065 inch $\leq$ 0.15 inch (anticipated joint movement)
The seal may undergo compression set.

The acceptable installation temperature range
= [allowable movement ÷ total movement] * temp range
= (0.065 ÷ 0.15) * 76°
= 33°
90° - 33° = 57°

The seal will not undergo compression set so long as the joint is sawed and sealed when the pavement temperature is greater than 57°F. It is recommended that the specifications be revised to limit the installation operation to temperatures above 57°F or the design revised to provide a shorter joint spacing.

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Memorandum

U.S. Department of Transportation

Federal Highway Administration

Subject: Examination of Cores from Kansas I-70

Date: JUL 25 1989

From: Chief, Pavement Division
Washington, D.C. 20590

Reply to Attn of: HHO-12

To: Mr. Thomas J. Ptak,
Deputy Regional Federal Highway Administrator HEO-07
Kansas City, Missouri

Attached is Dr. Stephen Forster's report on the examination of the concrete cores from Kansas I-70 east of Abilene (Kansas Project No. 70-21 K-2588-01). Dr. Forster did not find any evidence of "D" cracking of the aggregate or alkali-aggregate reactivity. The crack faces appear rough enough to provide load transfer if the cracks remain tight. However, the cracks in these cores have opened to the point where load transfer has been lost and the cracks are working.

We are observing a significant number of jointed reinforced concrete pavements (JRCP) with working cracks. The two factors believed to be the primary cause of working cracks in JRCP are corroded and locked up dowel bars and inadequate reinforcement. The introduction of epoxy coated dowels has reduced the risk of dowel bar corrosion. However, the procedures used to determine the amount of reinforcement in JRCP are not adequate.

Reinforcement for JRCP is designed using the subgrade drag theory. The procedure does not consider the crack aggregate interlock capability or the repeated shear loads from traffic. Also, the subgrade drag theory does not directly consider climatic effects. In the absence of a good design procedure for the reinforcement in JRCP, we believe the following conclusion from an ongoing research study "Performance/Rehabilitation of Rigid Pavements" provides good guidance:

> "The amount of reinforcement appeared to have an effect in controlling the amount of deteriorated transverse cracking. Although often confounded by the presence of corrosion-resistance dowel bars, pavement sections that contained more than 0.1 percent reinforcing steel exhibited less deteriorated transverse cracking; sections with less than that amount often displayed a significant amount of transverse cracking, particularly in cold climates. A minimum of 0.1 percent reinforcing steel is therefore recommended, with larger amounts required for harsher climates and longer slabs."

Based on Kansas Standard Plan 707.2, it appears there was approximately 0.07 percent reinforcing steel in the Kansas I-70 pavement. We recommend that the State consider increasing the amount of reinforcing steel on future projects.

Correction of working cracks is a very costly process. The available alternatives include installation of retrofit dowels and full-depth patching. If the retrofit dowel technique is used, the work should be performed before the cracks start to deteriorate. A minimum of three dowels is required in each wheelpath. The cost per dowel should be in the range of $30 to $70, depending on the quantities, labor costs, and hardness of the aggregate. When the full-depth repair option is selected, work should be performed after the distress begins to have a serious impact on pavement serviceability. The use of full-depth patching after the distress has occurred is generally the preferred alternative for several reasons: (1) It is difficult to predict whether the working cracks will result in a significant reduction in pavement serviceability. (2) If the rate of serviceability loss is low, the full-depth patching can be performed at the same time future rehabilitation needs are addressed.

Attached is a copy of the latest draft of the report for the research project "Performance/Rehabilitation of Rigid Pavements." Also attached is information on retrofit dowel bar installation.

Please contact Mr. John Hallin at 366-1323, if you have any questions concerning these comments.

Louis M. Papet

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Subject: Dowel Bar Inserters

Date: February 23, 1996

From: Chief, Pavement Division

Reply to Attn. of: HNG-40

To: Regional Administrators
Federal Lands Highway Program Administrator
Attention: Regional Pavement Engineers

By a March 6, 1990, memorandum, Mr. Louis Papet provided a copy of a Wisconsin Department of Transportation report on "Dowel Bar Placement: Mechanical Insertion Versus Basket Assemblies." Since that time, there appears to have been poor acceptance of the use of dowel bar inserters. A recent draft NCHRP report noted that 8 States allow the use of inserters, 13 States allow it as an acceptable option, and 20 States do not allow their use.

This technique has been used exclusively in some European countries for over 20 years with satisfactory dowel placement results. We believe all States should be encouraged to make this an allowable option in their specifications. We continue to encourage checking of dowel tolerances by probing through the fresh concrete early during the project and periodically as the work progresses. We also continue to recommend that when either baskets or inserters are used, the location of the dowels in the completed pavement be verified using metal detectors, pachometers, and cores.

If you have any comments or questions please contact Mr. John Hallin at (202) 366-1323 or Mr. Roger Larson at (202) 366-1326

T. Paul Teng

T. Paul Teng, P.E.

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Subject: Dowel Bar Inserters

Date: MAR 6 1990

From: Chief, Pavement Division
Washington, D.C. 20590-0001

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Attention: Pavement Engineers

Attached for your information is a copy of a report prepared by the Wisconsin Department of Transportation (WDOT) entitled "Dowel Bar Placement: Mechanical Insertion Versus Basket Assemblies." This study found that dowel placement accuracy achieved with the mechanical inserters equaled or surpassed the accuracy achieved with basket assemblies. As a result, the WDOT now permits the use of mechanical dowel bar inserters on construction projects.

Wisconsin's evaluation of dowel placement accuracy was based on their specification, which permits an alignment tolerance of 1/2-inch per dowel. This is slightly greater than the 1/4-inch per foot (3/8-inch per dowel) recommended in our May 17, 1989, Technical Paper 89-03, Benefits of Using Dowel Bars. Wisconsin is using a joint spacing of 12-13-19-18 feet and has not reported any distress which would indicate dowel alignment problems. As pointed out in Technical Paper 89-03, we are continuing to evaluate the specification tolerances for dowel alignment.

We concur with the WDOT's conclusion that: "The initial set-up of the dowel bar inserter with respect to depth of dowel placement is critical at the start of each project, and dowel depths should be verified by probing through the fresh concrete." We also recommend that when either baskets or inserters are used, the location of the dowels in the completed pavement be verified using metal detectors, pachometers, and cores.

If you have any questions or comments, please contact Mr. John Hallin at 366-1323.

Louis M. Papet

DOWEL BAR PLACEMENT:
MECHANICAL INSERTION VERSUS BASKET ASSEMBLIES

FINAL REPORT

WISCONSIN DEPARTMENT OF TRANSPORTATION

MARCH 1989

3.5.2

Dowel Bar Placement: Mechanical Insertion Versus Basket Assemblies

Final Report
Project I.D. 0624-32-08
Study No. 88-10

Workplan and Project Coordination by:
Ashwani K. Sharma
Research Project Engineer

Analysis and Report by:
James M. Parry, P.E.
Pavement Monitoring Engineer

February 1989

Wisconsin Department of Transportation
Division of Highways and Transportation Services
Central Office Materials
Applied Research and Pavement Management Sections

ABSTRACT

A mechanical dowel bar inserter was used on three highway construction projects in Wisconsin in 1987 and 1988. Coring was performed on these projects, and on three projects where dowel basket assemblies were used, to determine the dowel placement accuracy of both techniques. Study results indicate that the dowel placement accuracy achieved with the mechanical inserter equaled or surpassed the accuracy achieved with basket assemblies. Based on the results of this study, the mechanical dowel bar inserter will be allowed as an equal alternate to basket assemblies on 1989 construction projects in Wisconsin.

There were some problems on the initial projects where the mechanical inserter was used, including occasional missing dowels, improper location of sawed joints with respect to the location of the dowels, and voids in the concrete above the ends of the dowel bars. However, with continued refinement of construction techniques by the contractors and careful inspection by WisDOT construction personnel, it is believed that these problems associated with the new technology can be reduced or eliminated on future projects.

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INTRODUCTION

Traditionally, the dowel bars at Portland Cement Concrete (PCC, transverse pavement joints have been placed using wire basket assemblies, which are staked to the base prior to paving. Dowel basket assemblies are expensive, and their placement is labor intensive. For at least three decades, contractors and equipment manufacturers have been trying to develop a piece of paving equipment which can accurately place dowels in the plastic concrete at transverse joints, to eliminate the need for dowel baskets.

In the days of side form paving, a machine was developed and used in several states that was doing an acceptable job of vibrating dowels in ahead of the final finishing machine, but with the advent of slipform paving this machine proved unsatisfactory. If the dowels were placed between the spreader and the slipform, the dowels tended to settle or move horizontally when the slipform passed over them. If the dowels were vibrated in behind the slipform paver, there was a depression at the joint location which could not be removed by hand finishing with a straightedge.(1)

In recent years many states, including Wisconsin, are specifying dowels in all transverse joints on heavily trafficked PCC pavements, due to excessive faulting which has occurred on existing pavements without dowels. Shorter joint spacings are now also commonly used, which require many more dowel baskets p mile of pavement. These changes in design policy have stimulated even greater interest in developing a mechanical dowel bar inserter that will work with a slipform paver.

For several years a mechanical dowel bar inserter made by Guntert & Zimmerman has been used in Europe with reported success. A Wisconsin paving contractor purchased one of these machines, and was granted an opportunity to use it on an experimental basis on a project on I-90 at Janesville in 1987. Construction Technology Laboratories, Inc., of Skokie, IL (CTL) was retained to conduct a study of the dowel placement accuracy of the inserter versus baskets on the Janesville project, using a ground penetrating radar system.

The report from this study was reviewed by WisDOT staff, and the study results were found to be inconclusive. This was principally due to shortcomings of the ground penetrating radar technology used in the study. The problems included lack of precision of the measurements for some of the dowel placement parameters, and marginal correlation between the radar data and coring results. Consensus opinion of the WisDOT staff was that additional investigation was needed before the dowel inserter could be approved for general use.

Numbers in parentheses denote references given at end of repor

In July of 1988 this need for additional information was addressed when the workplan for the current study was developed. Coring was chosen as the method to be used for evaluation of dowel location despite the destructive nature of the testing, because of the precise and accurate results which could be obtained. Two additional pavements were constructed using the dowel inserter in 1988, so a more broad-scaled investigation was now possible. The purpose of the current study is to determine whether the dowel bar inserter is acceptable as an equal alternate to dowel baskets for future WisDOT paving projects.

The report by Construction Technology Laboratories, entitled "Field Evaluation of Dowel Placement along a Section of I-90 Near Janesville, Wisconsin", which was the final report for the radar study, will be hereafter refered to as the CTL report.(2)

SAMPLING AND CORING PROCEDURES

Number of Projects: Since three dowel inserter projects were available for evaluation, three additional independent projects using dowel baskets were also selected for this study. The comparison section with dowel baskets on the I-90 Janesville dowel inserter project was not included in this study due to reports from field personnel that unusual care was taken in placing the baskets on that project. The six projects that were selected for this study are described in Table 1.

Number of Dowels per Project: To achieve good statistical reliability while keeping costs and coring damage down to a reasonable level, a nominal sample size of 90 dowels per project was selected.

Location of Test Joints: The test joint locations on each project were selected in such a manner so that a representative sample would be collected. The optimum procedure would have been to select joints at random intervals throughout the project, but traffic control cost constraints dictated the need to concentrate the testing in limited test sections. Three test sections were designated for each project. The test sections were located a mile apart, and were located in the central portion of the project. Spreading the test sections out in this manner assured that all test joints would not fall in a single isolated problem area of the project. Central location of the test sections on the project assured that samples would not be taken near the ends of the project, where "start-up" paving problems typically occur. The location of the starting points for each of the test sections on each of the projects are given in Table 2.

A total of 15 test joints were designated in each test section. To assure equal rotation through the 4-joint repeating random skewed joint spacing pattern, every third joint in the test section was designated as a test joint.

Location of Test Dowels in Test Joints: At each test joint, one dowel was tested in the passing lane, and one dowel was tested in the driving lane. This was done to assess the relative placement error as it varied across the test joint. On the South Madison Beltline project, which had three lanes per direction, testing was confined to the two adjacent lanes (closest to the median) which were paved simultaneously with the dowel bar inserter.

The lateral position of each of the dowels across the pavement was identified by numbering the dowels for each lane in ascending order from the shoulder to the longitudinal joint between the two lanes, as shown in Figure 1. The lateral position of the test dowels for all successive test joints on a project was determined by rotating through the random number sequence shown in Table 3. This assured even representation in the sample for all dowel positions across the lanes.

Coring Procedure: A metal detector was used to locate the designated test dowel at the test joint. Partial depth core holes, centered over the ends of the dowel, were then drilled down to the depth of the dowel. The concrete core was then snapped off at the depth of the dowel, exposing the upper portion of the ends of the dowel bar for location measurements.

Measurements Collected for Test Dowels: The following measurements were collected for each end of each test dowel.

Vertical position: The distance from the top of the dowel end to the pavement surface was measured directly with a tape measure, either from the core or the core hole. These measurements were used to determine the average depth and vertical rotation of the dowels.

Lateral position: The distance from the center of the dowel end to the shoulder edge of the lane was measured using the following procedure. A two-foot carpenter's level was fitted with a custom-built tripod with leveling screws. A small black mark was made at the center of the dowel end. To project a vertical line above the dowel center, the level was plumbed vertically and the edge of the level was sited in to line up with the mark on the dowel. An eight-foot straightedge was laid longitudinally at the shoulder edge of the pavement. A tape measure was stretched transversely along the pavement surface from the siting edge of the level to the straightedge at the shoulder, and the resulting measurement was defined as the lateral position. These measurements were used to determine the horizontal rotation of the dowels.

Longitudinal position: The distance from the end of the dowel to the sawn joint was measured using the following procedure. To project a vertical line above the end of the dowel, the same mounted carpenter's level was plumbed and sited in to line up with the end of the dowel. A tape measure was laid on the pavement surface directly over and parallel to the dowel from the siting edge of the level to the center of the sawn joint. The resulting measurement was defined as the longitudinal position. These measurements were used to determine the longitudinal offset from the center of the dowel to the sawn joint.

Dowel Placement Parameters Evaluated: The four dowel placement parameters evaluated in this study are defined below.

Note that lateral spacing of the dowels was not evaluated in this study, because consecutive dowels across a single joint were not cored. Lateral spacing is not an especially sensitive parameter, and previous WisDOT coring on basket projects and the CTL report on the Janesville inserter project both showed no problem with meeting the contract specification (12" plus or minus 1") for lateral spacing.

The dowel placement parameters are illustrated in Figure 2.

<u>Vertical translation</u>: This is defined as the average depth of the dowel, measured from the top of the dowel to the pavement surface.

<u>Vertical rotation</u>: This is defined as the difference in depth (vertical position) between the opposite ends of the dowel.

<u>Horizontal rotation</u>: This is defined as the difference in lateral position between opposite ends of the dowel.

<u>Longitudinal translation</u>: This is defined as the longitudinal offset between the midpoint of the dowel and the sawn joint.

PLACEMENT TOLERANCE SPECIFICATIONS

The exact placement tolerance specifications used for this analysis are not critical, because the principal objective of this study is to compare the relative performance of the two dowel placement techniques. However, tolerance specifications do provide a useful frame of reference for this performance comparison, so the following specifications were used.

Vertical Translation: Specifications for this parameter were included in the construction contracts for all six projects in this study. The specifications consisted of a target depth for the dowels, and an allowable range of deviation. For the two 1987 construction projects, the I-90 Janesville project and the West Madison Beltline project, the target depth specified called for the dowels to be centered 1/2 inch above the mid-depth of the slab. For the four 1988 construction projects, the target depth specified called for the dowels to be centered at the mid-depth of the slab. For the I-90 project, the allowable range of tolerance for dowel depth was plus or minus 1 inch from the target depth. The same tolerance ranges were used in this analysis for the other five projects in the study.

Vertical Rotation: The tolerance specified in the CTL report for the I-90 project allowed 1/2 inch of vertical deviation from the true longitudinal axis of the pavement. This same tolerance was used for analysis of all of the projects.

Horizontal Rotation: The tolerance specified in the CTL report for the I-90 project allowed 1/2 inch of horizontal deviation from the true longitudinal axis of the pavement. This same tolerance was used for analysis for all of the projects.

Longitudinal translation: No specification was established for this placement parameter in the CTL report for the I-90 project. However, it was cited in a recent FHWA publication that it was necessary to have 6 inches of dowel on each side of the joint for effective load transfer and joint life.(3) Dowels longer than 12 inches are used in practice to allow leeway for joint sawing errors. Thus, for 18-inch dowels, a longitudinal offset of 3 inches in either direction is tolerable, and adequate load transfer is still provided. This tolerance was used for all projects in this analysis.

ANALYSIS OF PERFORMANCE

The coring results and statistics are summarized in Table 4 fo all six projects. A detailed listing of all dowel locations a measurements is provided in Appendix A.

The types of analysis that were performed for each of the dowel placement parameters included the following areas. A basic analysis of averages and distribution densities was conducted using means and standard deviations, and these statistics are included in Table 4.

Additionally, analysis was performed on the direction of deviation from the optimum dowel position (e.g. rotated left or right, rotated up or down, longitudinally offset forward or backward; with respect to the direction of paving). For all dowel placement parameters on all projects, the data had approximately balanced normal distributions which were centered at or near the optimum dowel position. The lack of skewed distributions and lack of distributions centered well away from the optimum position indicate that the variation is due to normal random fluctuation of the two dowel placement methods. It indicates that no pervasive systematic problems exist in either of the dowel placement processes which would caused skewed data.

Analysis of directional deviation of the dowel placement parameters versus the lateral position of the dowels across the roadway was also conducted. Again, for all placement paramete on all projects, the lateral position of the dowels across the roadway had no significant effect on the placement parameters of the dowels, indicating no pervasive systematic problems in either of the dowel placement processes.

<u>Vertical Translation</u>: First, the distribution of dowel depths was examined. The standard deviations for the inserter projects (0.20" to 0.46") were comparable or smaller than those for the basket projects (0.35" to 0.57"), indicating that the inserter is capable of consistent depth placement of the dowels. This does not necessarily mean that the inserter is better in this respect than baskets, because the frame of reference is different for the two placement methods. The depth of the dowels was measured from the pavement surface down to the top of the dowels. Dowel baskets are staked to the base course, thus referenced to the bottom of the slab. As the thickness of the pavement varies along the length of the project (typically 1" or more fluctuation), the depth of the dowels, as measured from the surface, would vary directly with the fluctuation of the slab thickness. The placement of dowels by the inserter is referenced to the paver frame, and is thus more closely correlated with the pavement surface. Thus, fluctuation in pavement thickness should have little or no effect on dowel depths for the inserter projects. In conclusion, the dowel bar inserter is capable of

placing dowels in a satisfactory close distribution around a target depth.

Second, the average placement depth of the dowels was examined. The mean depth of placement for all of the inserter and basket projects was below the target depth specification. However, all of the projects except two had had at least 90% of the dowels placed within the allowable range of the depth specification. On one inserter project and one basket project, a significant percentage of the dowels were placed too deep to meet the maximum depth specification.

On the South Madison Beltline project, where the dowel bar inserter was used, 39% of the sampled dowels were placed too deep to meet the maximum depth specification. Since the dowel bar inserter is attached to the frame of the paver, dowel placement is therefore referenced to the top of the slab, as discussed previously. Thus, placement depth, as measured from the top of the slab, should not be influenced by fluctuation in overall pavement thickness. It is then very probable that the inserter was set up incorrectly at the start of the project, and that the dowels were consistently placed too deep throughout the project. The target depth for the dowels was 0.8 inches higher than the mean depth measured. In conclusion, great care must be taken to set up the inserter for proper depth placement at the start of each project, and the setup should be verified by checking dowel depths in the fresh concrete during the early stages of paving.

On the West Madison Beltline project, where dowel baskets were used, 39% of the sampled dowels were placed too deep to meet the maximum depth specification. Since the dowel baskets are staked to the base course, dowel placement is therefore referenced to the bottom of the slab, as discussed previously. Thus, placement depth, as measured from the top of the slab, would directly reflect fluctuations in overall slab thickness. The nominal slab thickness for this project, which was used for the analysis, was 10 inches. Based upon the results of the pavement thickness quality control coring which was performed on the project, the average actual slab thickness was 10.6 inches in the southbound lanes, where the dowel test sections were located. The target depth for the dowels was 0.9 inches higher than the mean depth measured. This difference is only slightly greater than the additional slab thickness measured. Also, if this project had been constructed under the 1988 specifications, the target dowel depth would have been 1/2 inch deeper, and only 6% of the dowels would have been too deep, based on nominal slab thickness. If the analysis were based on actual slab thickness, the out of spec figure would have been reduced even further. In conclusion, while the depth of concrete cover over the dowels fluctuates with pavement thickness on basket projects, the depth of concrete cover below the dowels remains predominantly consistent and adequate.

Vertical Rotation: At least 90% of the dowels were placed within the specification for vertical rotation on all inserter and basket projects. The mean rotation varied from 0.14 to 0.25 inches on the individual projects. In conclusion, the dowel bar inserter performed satisfactorily with respect to the vertical rotation parameter.

Horizontal Rotation: At least 90% of the dowels were placed within the specification for horizontal rotation on all of the inserter projects. Somewhat surprisingly, 17% to 22% of the dowels on the three basket projects did not meet the specified limit of 1/2 inch of horizontal rotation. The mean horizontal rotation for the inserter projects ranged from 0.21 to 0.26 inches, while the mean for the basket projects ranged ranged from 0.32 to 0.40 inches. In conclusion, the dowel bar inserter performed satisfactorily with respect to the horizontal rotation parameter, and was superior to basket performance.

Longitudinal Translation: This parameter is more closely associated with the marking and sawing of joints than with the actual performance of the dowel bar inserter itself. However, whether placement is by inserter or baskets, it is imperative that the sawed joint is aligned properly with the midpoint of the dowels. The number of joints which were improperly aligned with the dowels ranged from 1% to 15% on the inserter projects, and from 1% to 22% on the basket projects. Hence, there is definitely room for improvement of the joint locating techniques used both on inserter and basket projects. An economical means by which to improve performance in this area would be to have available a magnetic rebar locator on all doweled PCC construction projects. This could be used to verify the location of the dowels relative to the pre-established locating marks, especially in the early stage of paving on a project, until the joint marking and sawing procedure is refined to an acceptable level. It was the experience of the field crew doing the coring for this study, that the position of the dowel ends could be accurately established (plus or minus 1 inch) using a magnetic rebar locator. In conclusion, performance with respect to longitudinal translation needs improvement on both inserter and basket projects.

Ride Quality: Concern has been expressed over adverse effects on the ride quality of pavements where dowel bar inserters are used. Pavement serviceability index (PSI) is measured on all newly constructed pavements in Wisconsin to assess the ride quality of these projects. These ride quality measurements are collected after diamond grinding has been completed to meet the California Profilograph based WisDOT smoothness specifications which are part of the construction contract. The total amount of original roughness on a project can then be qualitatively assessed as a combination of the final PSI, and the amount of grinding which was done to achieve that level of ride quality. It is important to remember that some or most of the roughness on a project may originate from paving problems which are independent of the dowel bar inserter.

On the first inserter project, I-90 at Janesville, an extensive amount of grinding was performed, and the final project PSI was only 3.6. Field inspection of this project revealed that, especially in the southern portion of the project, some of the existing roughness and grinding was related to the dowel bar inserter. In some unground stretches, cyclical distortion of the longitudinal profile was visually evident in synchronization with the pavement joints. In some ground stretches, cyclical variation of the depth of grinding was visually evident in synchronization with the pavement joints. However, the overall majority of roughness and grinding on the project appeared to be related to longer wavelength profile distortion, which is typically associated with other independent paving problems.

On the second inserter project, USH 18/151 from Dodgeville to Mt. Horeb, a moderate amount of grinding was performed, and the final project PSI was 4.2. Field inspection of this project revealed that no significant amount of roughness or grinding was synchronized with the pavement joints. The roughness and grinding on this project appeared to be all related to longer wavelength profile distortion, associated with other independent paving problems. Some roughness on this project may have been caused by construction problems with the experimental open-graded base course which was used on portions of the project.

On the third inserter project, the South Madison Beltline, a relatively light amount of grinding was performed, and the final project PSI was 4.6. With a PSI that high and the small amount of grinding, there was not much initial roughness built into this project. Field inspection revealed no significant profile distortion associated with the joints. The limited grinding which was done appeared to be related to longer wavelength profile distortion, independent of the joints.

All three of these projects were paved by the same contractor. It is evident that the contractor's ability to produce a smooth riding pavement with this paver/dowel inserter combination has improved dramatically. In conclusion, by the third project, satisfactory performance was achieved with the dowel inserter with respect to ride quality.

<u>Voids</u>: Another concern about the dowel inserter has been the quality of consolidation of the concrete around the dowels when they are inserted. Significant voids (dime-size or larger - small bugholes were not counted) were found immediately above the ends of 22% to 34% of the dowels on the inserter projects. The voids were always very close to the ends of the dowel, within about the last inch of the bar. On two of the basket projects, no voids were found above the dowels, but on the STH 29 Vinton project voids were found above 40% of the dowels.

With the inserter, the concrete flows upward past the dowel as the dowel is vibrated downward, and the logical location for voids is above the dowel, so it is likely that all voids were detected on the inserter projects. However, on the basket projects the concrete flows downward past the dowel, and the logical location for voids is underneath the dowel. Since on_ the top of the dowel was inspected for this study, it is possible that the void problem may be understated for the basket projects due to undetected voids beneath the dowels. Further coring needs to be performed on basket projects to assess the extent of this problem.

Voids of this magnitude could affect the load transfer capacity of the dowels, so improvement in this area is needed. The amount of vibration used on the inserter needs to be increased slightly, but caution must be exercised to avoid using excessive vibration which may damage the concrete. In conclusion, quality control coring should be performed on inserter and basket projects constructed in the near future to assess the progress in solving this problem.

Missing Dowels: Another problem unique to the dowel bar inserter is that sometimes dowel bars are missing completely. This is difficult to inspect for, when the dowels are immediately buried in the concrete, instead of being laid out on the grade ahead of the paver. At one test joint on the USH 18/151 project, all of the dowels for the joint were missing. Using the magnetic rebar locator, it was determined that all of the dowels were present in the two adjacent joints, but the 24 missing dowels were not found anywhere between the adjacent joints. On the South Madison Beltline project, three dowels were missing from one test joi and one dowel was missing from another test joint. The three missing dowels were located at the edge of the pavement on the opposite side of the road from where the inserter distribution carriage is loaded. In that instance, it is likely that an insufficient number of dowel bars were loaded into the distribution carriage for that joint. The single missing dowel was located in the first position on the same side of the road where the distribution carriage is loaded. In that instance, it is possible that the missing dowel resulted from a misfeed or jam of the distribution system.

If every joint on the three inserter projects in this study was to be checked with a rebar locator, it is doubtless that additional occurrences of missing dowels would be identified, but the extent of the problem is currently not known. However, if the frequency of missing dowels noted at the test joints is an accurate indicator, then the incidence of missing dowels is probably relatively rare and isolated. This issue presents another good justification for having a magnetic rebar locator available on future inserter projects. The paving inspector cannot possibly observe the performance of the inserter on every joint. It would be good practice to make random checks with the rebar locator to verify dowel presence, and to make more extensive searches if a problem is suspected. If a significant number of bars was determined to be missing, payment penalties could be assessed by this type of survey. In conclusion, missing dowels do not appear to represent a widespread problem on the inserter projects in this study, but should still be monitored on future projects.

Other Brands of Dowel Bar Inserters: All of the data and conclusions in this study are valid only for the Guntert & Zimmerman dowel bar inserter used on the projects in this study. If a different brand of dowel bar inserter is used which differs greatly in design and operation from the Guntert & Zimmerman model, a thorough performance evaluation of the new machine will be essential. Performance with respect to any or all of the placement parameters discussed in this analysis could be widely different for a different machine.

SUMMARY AND CONCLUSIONS

With any new form of technology, there will always be some problems that need to be resolved during the initial learning period. The dowel bar inserter is no exception to this rule, and several problems have been identified with its performance on the projects in this study. However, none of these problems appear to be insurmountable. Through the continued cooperative efforts of WisDOT construction personnel and the contractors, it should be possible to improve construction procedures to obtain consistent satisfactory results with the dowel bar inserter.

The primary general recommendation of this study is to accept the dowel bar inserter as an equal alternate to dowel baskets for future WisDOT doweled PCC construction projects. The following list of specific conclusions and recommendations are based on the results of this study.

1. The dowel bar inserter is capable of consistent satisfactory placement of dowel bars with respect to vertical translation (average depth), vertical rotation (difference in depth between two ends of dowel), and horizontal rotation (difference in transverse position between two ends of dowel).

2. The initial set-up of the dowel bar inserter with respect to depth of dowel placement is critical at the start of each project, and dowel depths should be verified by probing through the fresh concrete.

3. The construction procedures currently used for marking and sawing joints need improvement both for inserter and basket projects, to consistently and accurately align the sawn joints with the midpoints of the dowel bars.

4. Having a magnetic rebar locator available on all doweled PCC construction projects would be useful in aligning sawn joints with the dowel bars and in identifying missing dowels.

5. Ride quality has improved on each successive inserter project, and on the latest project, the South Madison Beltline, a project PSI of 4.6 was achieved with minimal diamond grinding.

6. Improved concrete consolidation around the dowels is needed both on inserter and basket projects, and quality control coring is needed to assess future progress in solving the problem of voids around the dowel bars.

7. Problems with missing dowel bars on existing inserter projects appear to be infrequent and isolated, but this problem should be monitored on future projects.

8. All of the data and conclusions in this study are valid only for the Guntert & Zimmerman dowel bar inserter used on the projects in this study. If a different brand of dowel bar inserter is used which differs widely in design and operation from the Guntert & Zimmerman model, a thorough performance evaluation will be essential.

REFERENCES

1. Ray, Gordon K., "Dowel Inserters", Roads and Bridges, April 1988, page 19.

2. Okamoto, Paul A., "Field Evaluation of Dowel Placement Along a Section of I-90 Near Janesville, Wisconsin - Final Report", Construction Technology Laboratories, Inc., for Wisconsin Department of Transportation, August 1988.

3. Cashell, Harry D., "Performance of Doweled Joints Under Repetitive Loading", Public Roads, Volume 30, Number 1, April 1958.

TABLE 1. DESCRIPTION OF PROJECTS INCLUDED IN DOWEL BAR PLACEMENT STUDY

PROJECT LOCATION DESCRIPTION	DISTRICT	STATE PROJECT NUMBER	CONSTRUCTION YEAR	PAVING CONTRACTOR	DOWEL PLACEMENT TECHNIQUE	PAVEMENT THICKNESS	BASE COURSE
I-90 in Rock County (Westbound Lanes Only) Madison - Illinois State Line Road (Manogue Road - USH 14 at Janesville)	1	1001-01-75	1987	James Cape & Sons, Inc.	Inserter	10"	8" Thick Dense Graded
USH 18/151 in Dane County (Eastbound Lanes Only) Dodgeville - Mt. Horeb Road (West County Line - CTH "PD")	1	1204-04-72	1988	James Cape & Sons, Inc.	Inserter	9"	Varied
USH 12/18 in Dane County South Madison Beltline (I-90 - South Towne Drive)	1	1206-02-79	1988	James Cape & Sons, Inc.	Inserter	10"	6" Thick Dense Graded
USH 12/14 in Dane County West Madison Beltline (Old Sauk Interchange)	1	5303-00-71	1987	Trierweiler Construction and Supply, Inc.	Baskets	10"	6" Thick Dense Graded
STH 29/32 in Brown, Shawano, & Outagamie Counties Shawano - Green Bay Road (STH 156 - CTH "U")	3	9202-02-76	1988	Streu Construction Co.	Baskets	10"	4" Thick Open Graded Over 4" Dense
STH 29/32 in Brown County Shawano - Green Bay Road (CTH "U" - USH 41)	3	9202-02-77	1988	Vinton Construction Co.	Baskets	10"	4" Thick Open Graded Over 4" Dense

TABLE 2. LOCATION OF TEST SECTIONS ON PROJECTS INCLUDED IN DOWEL BAR PLACEMENT STUDY

PROJECT LOCATION DESCRIPTION	DISTRICT	STATE PROJECT NUMBER	DIRECTION OF TESTING	TEST SECTION NUMBER	STATIONS
I-90 in Rock County (Westbound Lanes Only) Madison - Illinois State Line Road (Manogue Road - USH 14 at Janesville)	1	1001-01-75	Westbound	1 2 3	559+88 to 553+38 509+27 to 503+36 464+87 to 458+28
USH 18/151 in Dane County (Eastbound Lanes Only) Dodgeville - Mt. Horeb Road (West County Line - CTH "PD")	1	1204-04-72	Eastbound	1 2 3 4	961+37 to 968+01 1014+13 to 1020+70 1066+17 to 1072+73 1232+17 to 1238+73
USH 12/18 in Dane County South Madison Beltline (I-90 - South Towne Drive)	1	1206-02-79	Westbound	1 2 3	191+87 to 185+20 154+79 to 148+27 64+93 to 58+21
USH 12/14 in Dane County West Madison Beltline (Old Sauk Interchange)	1	5303-00-71	Eastbound	1 2 3	100+17 to 106+84 125+07 to 131+72 138+00 to 144+72
STH 29/32 in Brown, Shawano, & Outagamie Counties Shawano - Green Bay Road (STH 156 - CTH "U")	3	9202-02-76	Westbound	1 2 3	484+91 to 478+45 419+80 to 413+17 369+90 to 363+30
STH 29/32 in Brown County Shawano - Green Bay Road (CTH "U" - USH 41)	3	9202-02-77	Eastbound	1 2 3	675+19 to 682+64 725+20 to 731+76 760+20 to 766+72

TABLE 3. RANDOM SAMPLING SEQUENCE FOR LATERAL POSITION OF DOWELS FOR DOWEL BAR PLACEMENT STUDY

SEQUENCE FOR 12-FOOT LANE	SEQUENCE FOR 14-FOOT LANE
DOWEL BAR NUMBER	DOWEL BAR NUMBER
2	2
1	1
8	8
7	7
9	9
12	12
6	6
10	10
3	3
5	5
4	4
11	13
	14
	11

TABLE 4. SUMMARY OF CORING DATA FOR DOWEL BAR PLACEMENT STUDY

TEST PARAMETERS		PROJECT DESCRIPTIONS AND DATA					
		DOWEL BAR INSERTER			DOWEL BASKETS		
(Note: Dimensions for all measurements are in inches.)		1001-01-75 I-90 ROCK CO. PAVED 1987	1206-02-79 USH 12/18 DANE CO. PAVED 1988	1204-04-72 USH 18/151 DANE CO. PAVED 1988	5303-00-71 USH 12/14 DANE CO. PAVED 1987	9202-02-76 STH 29/32 BROWN CO. PAVED 1988	9202-02-7 STH 29/32 BROWN CO. PAVED 198
NUMBER OF DOWELS TESTED		84	90	120	90	90	9
VERTICAL TRANSLATION (Average depth of dowel)	MEAN	4.31	5.22	4.49	4.79	4.59	4.1
	STANDARD DEVIATION	0.46	0.38	0.20	0.42	0.57	0.3
	MINIMUM VALUE OBSERVED	3.34	3.97	3.97	3.50	2.81	2.8
	MAXIMUM VALUE OBSERVED	5.56	6.06	4.88	5.81	5.84	5.0
	RANGE	2.22	2.09	0.91	2.31	3.03	2.1
	DEPTH SPECIFICATIONS	3.875+-1in.	4.375+-1in.	3.938+-1in.	3.875+-1in.	4.375+-1in.	4.375+-1i
	PERCENT OF OBSERVATIONS EXCEEDING SPECIFICATION						
	TOO SHALLOW	0%	0%	2%	0%	3%	[illegible]
	TOO DEEP	10%	39%	0%	39%	7%	[illegible]
VERTICAL ROTATION (Difference in depth between opposite ends of dowel)	MEAN	0.25	0.19	0.15	0.16	0.17	0.1
	STANDARD DEVIATION	0.21	0.14	0.13	0.33	0.18	0.1
	MINIMUM VALUE OBSERVED	0.00	0.00	0.00	0.00	0.00	0.0
	MAXIMUM VALUE OBSERVED	1.13	0.75	0.63	3.00	1.00	0.6
	PERCENT OF OBSERVATIONS EXCEEDING SPECIFICATION (diff. depth > 0.5 in.)	10%	3%	5%	6%	4%	
HORIZONTAL ROTATION (Difference in transverse position between opposite ends of dowel)	MEAN	0.26	0.25	0.21	0.36	0.40	0.3
	STANDARD DEVIATION	0.23	0.31	0.21	0.31	0.32	0.3
	MINIMUM VALUE OBSERVED	0.00	0.00	0.00	0.00	0.00	0.0
	MAXIMUM VALUE OBSERVED	1.31	2.00	1.00	1.44	1.81	1.6
	PERCENT OF OBSERVATIONS EXCEEDING SPECIFICATION (diff. trans.> 0.5 in.)	10%	8%	8%	22%	26%	17
LONGITUDINAL TRANSLATION (Longitudinal offset between sawed joint and midpoint of dowel)	MEAN	1.62	0.87	1.86	2.12	1.66	0.88
	STANDARD DEVIATION	1.31	0.65	2.29	1.91	1.50	0.74
	MINIMUM VALUE OBSERVED	0.00	0.00	0.00	0.00	0.00	0.00
	MAXIMUM VALUE OBSERVED	6.88	3.31	17.31	9.00	7.75	3.81
	PERCENT OF OBSERVATIONS EXCEEDING SPECIFICATION (offset > 3.0 in.)	8%	1%	15%	22%	20%	1
PERCENT OF DOWELS WHICH HAD A SIGNIFICANT VOID OVER AT LEAST ONE END OF THE DOWEL		26%	34%	22%	0%	0%	40

Note: Depth was measured from pavement surface to top of dowel end.
Transverse position was measured from edge of lane to center of dowel end.
Longitudinal position was measured from center of sawed joint to dowel end.

FIGURE 1. CONFIGURATION AND NUMBERING OF DOWELS FOR A TYPICAL ROADWAY SECTION

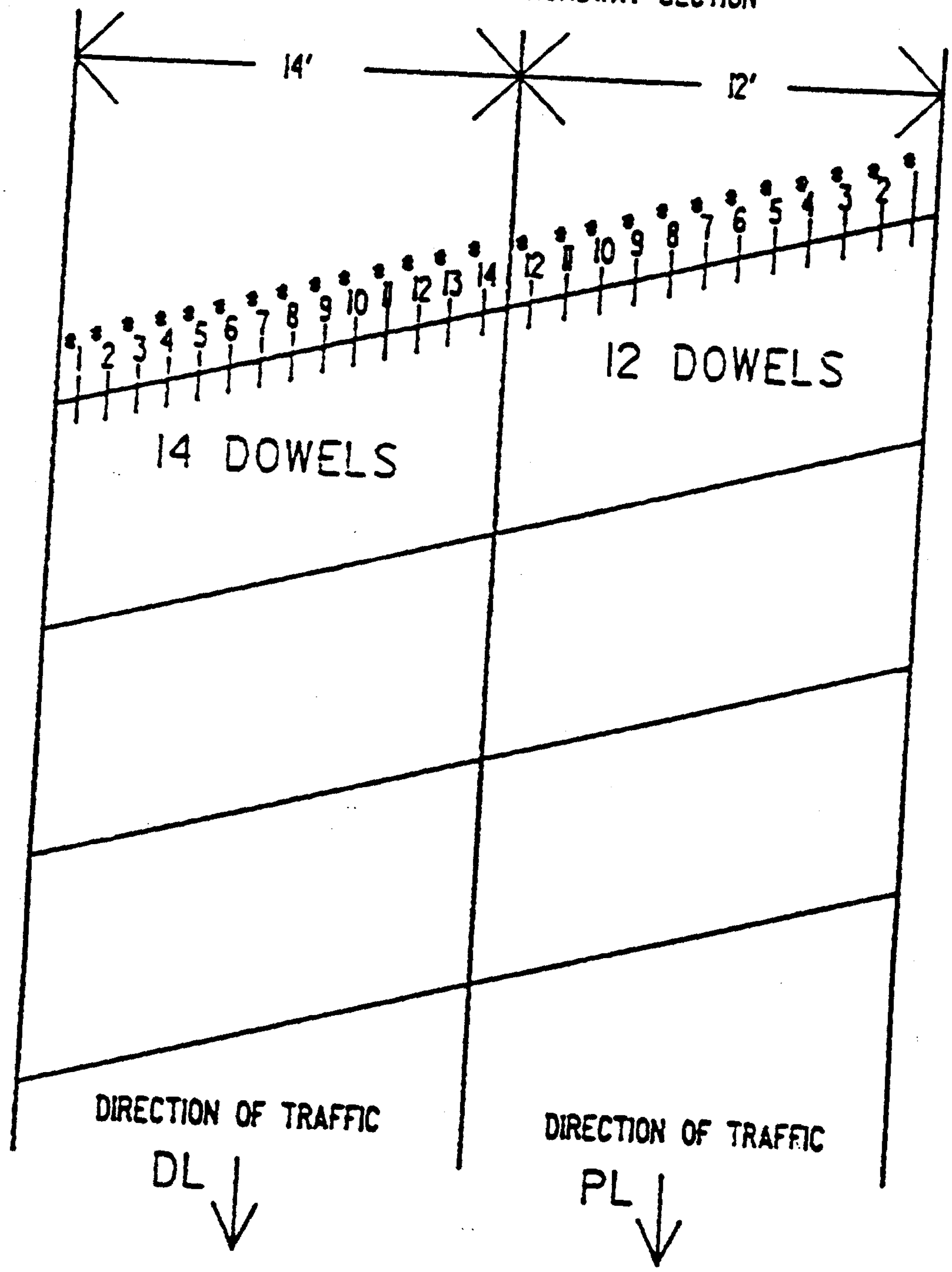
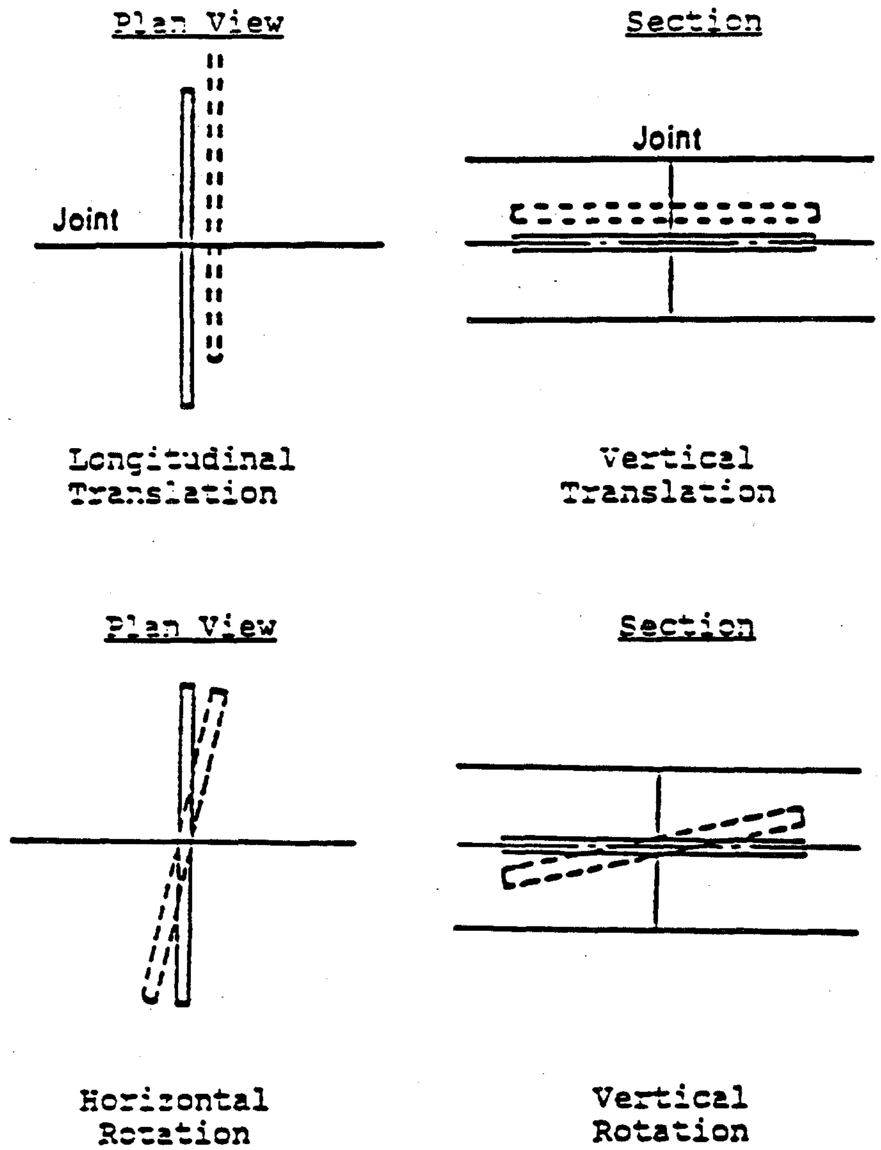


FIGURE 2: Illustration of Dowel Placement Parameters



APPENDIX A. DETAILED DATA FOR DOWEL PLACEMENT STUDY

Key for Appendix A

Variable Name	Description
BARNUMBR	= Lateral position of dowel in lane
DIRECTON	= Direction of traffic
LANE	= Driving lane (1) or passing lane(2)
STATION	= Station of test joint
VOID	= Severity rating for voids at ends of dowel N = none S = small (less than 1/4") M = medium (1/4" to 1/2") L = large (greater than 1/2")
DEPTH1	= Vertical position of upstream end of dowel (upstream with respect to traffic direction)
DEPTH2	= Vertical position of downstream end of dowel
TRANS1	= Transverse position of upstream end of dowel
TRANS2	= Transverse position of downstream end of dowel
LONG1	= Longitudinal position of upstream end of dowel
LONG2	= Longitudinal position of downstream end of dowel
AVGDEPTH	= Average depth of dowel
DIFDEPTH	= Difference in depth between two ends of dowel
DIFTRANS	= Difference in transverse position between two ends of dowel
LOFFSET	= Longitudinal offset from midpoint of dowel to sawn joint

(Counter variables: yes = 1 and no = 0)

AVGDEPSM	= Average depth of dowel too shallow for spec
AVGDEPLG	= Average depth of dowel too deep for spec
DIFDEPLG	= Difference in depth too large for spec
DIFTRNLG	= Difference in transverse position too large for spec
OFFSETLG	= Longitudinal offset too large for spec
VOIDNONE	= Void severity rating is "none"
VOIDSMAL	= Void severity rating is "small"
VOIDMEDM	= Void severity rating is "medium"
VOIDLARG	= Void severity rating is "large"

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

SUBJECT	FHWA TECHNICAL ADVISORY
CONTINUOUSLY REINFORCED CONCRETE PAVEMENT	T 5080.14 June 5, 1990

Par. 1. Purpose
2. Cancellation
3. Background
4. Design Recommendations
5. Construction Considerations

1. PURPOSE. To outline recommended practices for the design, construction, and repair of continuously reinforced concrete pavement (CRCP).

2. CANCELLATION. Technical Advisory T 5080.5, Continuously Reinforced Pavement, dated October 14, 1981, is cancelled.

3. BACKGROUND

a. Continuously Reinforced Concrete Pavement is a portland cement concrete (PCC) pavement that has continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints. The pavement is allowed to crack in a random transverse cracking pattern and the cracks are held tightly together by the continuous steel reinforcement.

b. During the 1970's and early 1980's, CRCP design thickness was approximately 80 percent of the thickness of conventional jointed concrete pavement. A substantial number of the thinner pavements developed distress sooner than anticipated.

c. Attention to design and construction quality control of CRCP is critical. A lack of attention to design and construction details has caused premature failures in some CRCPs. The causes of early distress have usually been traced to; (1) construction practices which resulted in pavements which did not meet design requirements; (2) designs which resulted in excessive deflections under heavy loads; (3) bases of inferior quality, or; (4) combinations of these or other undesirable factors.

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Level 3: Divisions(SH)

OPI: HHO-12

4. DESIGN RECOMMENDATIONS

a. Concrete Thickness. Generally the slab thickness is the same as the thickness of a jointed concrete pavement unless local performance has shown thinner pavements designed with an accepted design process to be satisfactory.

b. Reinforcing Steel

(1) Longitudinal Steel

(a) A minimum of 0.6 percent (based on the pavement cross sectional area) is recommended to aid transverse crack development in the range of 8 feet, maximum, and 3.5 feet, minimum, between cracks. Exceptions should be made only where experience has shown that a lower percentage of steel has performed satisfactorily. In areas where periods of extreme low temperature (average minimum monthly temperatures of 10° F or less) occur, the use of a minimum of 0.7 percent steel is recommended.

(b) Deformed steel bars that meet the requirements set out in AASHTO Specifications, Part I, AASHTO M31, M42, or M53 are recommended. The tensile requirements should conform to the American Society for Testing and Materials (ASTM) Grade 60. Recommended spacing of the longitudinal steel is not less than 4 inches or 2 1/2 times the maximum sized aggregate, whichever is greater, and not greater than 9 inches. A minimum ratio of 0.03 square inches of steel bond area per cubic inch of concrete is recommended. See Attachment 1 for an example problem for determining the minimum longitudinal steel spacing and the minimum bond ratio. Table 1 shows the minimum and maximum bar sizes for given pavement thicknesses and reinforcement percentages. These bar sizes meet the minimum bond ratio and the minimum bar spacing criteria stated above.

(c) The recommended position of the longitudinal steel is between 1/3 and 1/2 of the depth of the pavement as measured from the surface. The minimum concrete cover should be 2-1/2 inches with 3 inches preferable. For pavements thicker than 11 inches, several States have begun to experiment with the use of two layers of longitudinal steel. Pavements constructed with two layers of steel have not been in service long enough to evaluate performance; therefore, this technique should be considered experimental.

Table 1 - Recommended Longitudinal Reinforcement Sizes

	Minimum and Maximum Bar Size					
	Pavement Thickness					
% Steel	8"	9"	10"	11"	12"	13"
0.60	4,5	5,6	5,6	5,6	5,6	6
0.62	5,6	5,6	5,6	5,6	5,6	6
0.64	5,6	5,6	5,7	5,7	6,7	6,7
0.66	5,6	5,7	5,7	5,7	6,7	6,7
0.68	5,6	5,7	5,7	6,7	6,7	6,7

Note: Bars are uncoated deformed bars.

(d) The use of epoxy coated reinforcing steel is generally not necessary for CRCP. However, in areas where corrosion is a problem because of heavy applications of deicing salts or severe salt exposure, epoxy coating of the steel may be warranted. The bond area should be increased 15 percent to increase the bond strength between the concrete and reinforcement if epoxy-coated steel reinforcement is used.

(e) When splicing longitudinal steel, the recommended minimum lap is 25 bar diameters with the splice pattern being either staggered or skewed. If a staggered splice pattern is used, not more than one-third of the bars should terminate in the same transverse plane and the minimum distance between staggers should be 4 feet. If a skewed splice pattern is used, the skew should be at least 30 degrees from perpendicular to the centerline. When using epoxy-coated steel, the lap should be increased a minimum of 15 percent to ensure sufficient bond strength.

(f) Plan details or specifications are needed to insure sufficient reinforcing at points of discontinuity as described in paragraphs 4e(3) and 4f(1).

(2) Transverse Reinforcing and Tiebars

(a) If transverse reinforcement is included, it should be #4, #5, or #6 grade 60 deformed bars meeting the same specifications as mentioned for the longitudinal reinforcement.

(b) Although it can be omitted, transverse reinforcing reduces the risk of random longitudinal cracks opening up and thus reduces the potential of punch-outs. If transverse reinforcement is included, the following equation can be used to determine the amount of reinforcement required (see number 5 of Attachment 2):

$$P_t = \frac{W_s F}{2f_s} \times 100$$

Where:
P_t = transverse steel, %
W_s = total pavement width, (ft)
F = subbase friction factor
f_s = allowable working stress in steel, psi, (0.75 yield strength)

(c) The spacing between transverse reinforcing bars can be calculated using the following equation (see numbers 1 and 5 of Attachment 2):

$$Y = \frac{A_s}{P_t D} \times 100$$

Where:
Y = transverse steel spacing (in)
A_s = cross-sectional area of steel, (in^2) per bar (#4, #5, or #6 bar)
P_t = percent transverse steel
D = slab thickness (in)

Note: The transverse bar spacing should be no closer than 36 inches and no further than 60 inches.

(d) In cases where transverse steel is omitted, tiebars should be placed in longitudinal joints in accordance with the FHWA Technical Advisory, Concrete Pavement Joints.

c. Bases

(1) The base design should provide a stable foundation, which is critical for CRCP construction operations and should not trap free moisture beneath the pavement. Positive drainage

is recommended. Free moisture in a base or subgrade can lead to slab edge-pumping, which has been identified as one of the major contributors to causing or accelerating pavement distress. Bases that will resist erosion from high water pressures induced from pavement deflections under traffic loads, or that are free draining to prevent free moisture beneath the pavement will act to prevent pumping. Stabilized permeable bases should be considered for heavily traveled routes. Pavements constructed over stabilized or crushed stone bases have generally resulted in better performing pavements than those constructed on unstabilized gravel.

(2) The friction between the pavement and base plays a role in the development of crack spacing in CRCP. Most design methods for CRCP assume a moderate level of pavement/base friction. Polyethylene sheeting should not be used as a bond breaker unless the low pavement/base friction is considered in design. Also, States have reported rideability and construction problems when PCC was constructed on polyethylene sheeting.

d. Subgrades. Continuously Reinforced Concrete Pavement is not recommended in areas where subgrade distortion is expected because of known expansive soils, frost heave, or settlement areas. Emphasis should be placed on obtaining uniform and adequately compacted subgrades. Subgrade treatment may be warranted for poor soil conditions.

e. Joints

(1) Longitudinal Joints. Longitudinal joints are necessary to relieve stresses caused by concrete shrinkage and temperature differentials in a controlled manner and should be included when pavement widths are greater than 14 feet. Pavements greater than 14 feet wide are susceptible to longitudinal cracking. The joint should be constructed by sawing to a depth of one-third the pavement thickness. Adjacent slabs should be tied together by tiebars or transverse steel to prevent lane separation. Tiebar design is discussed in the FHWA Technical Advisory entitled "Concrete Pavement Joints".

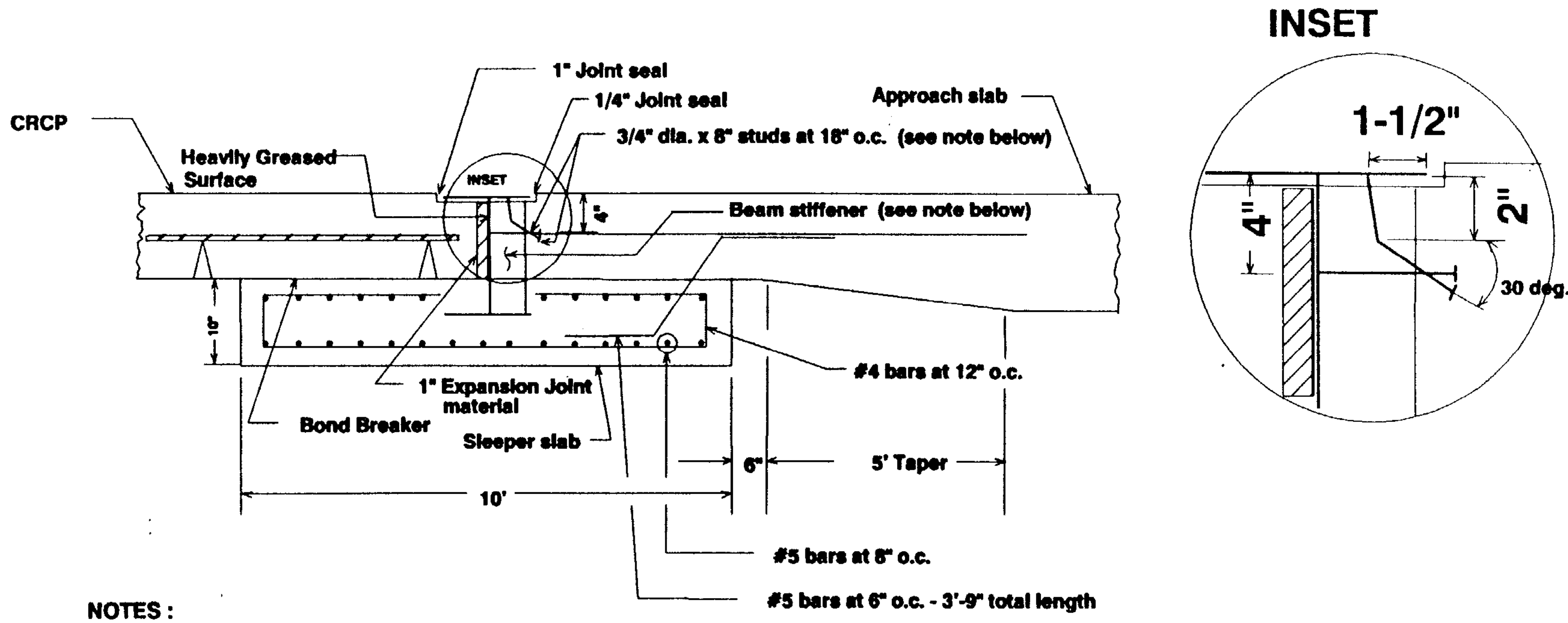
(2) Terminal Joints. The most commonly used terminal treatments are the wide-flange (WF) steel beam which accommodates movement, and the lug anchor which restricts movement.

(a) The WF beam joint consists of a WF beam partially set into a reinforced concrete sleeper slab approximately 10 feet long and 10 inches thick. The top flange of the beam is flush with the pavement surface. Expansion material, sized to accommodate end movements, is placed on one side of the beam along with a bond-breaker between the pavement and the sleeper slab. In highly corrosive areas the beam should be treated with a corrosion inhibitor. Several States have reported premature failures of WF beams where the top flange separated from the beam web. Stud connectors should be welded to the top flange, as shown in Figure 1, to prevent this type of failure. Table 2 and Figure 1 contain recommended design features.

TABLE 2 - Recommended WF Beam Dimensions

WF Beam (weight and dimensions)					
CRCP Thickness (in.)	Embedment in "Sleeper" slab - in.	WF Beam Size	Flange		Web Thickness (in.)
			Width (in.)	Thickness	
8 9	6 5	14 x 61	10	5/8	3/8
10 11	6 5	16 x 58	8-1/2	5/8	7/16

(b) The lug anchor terminal treatment generally consists of three to five heavily reinforced rectangularly shaped transverse concrete lugs placed in the subgrade to a depth below frost penetration prior to the placement of the pavement. They are tied to the pavement with reinforcing steel. Since lug anchors restrict approximately 50 percent of the end movement of the pavement an expansion joint is usually needed at a bridge approach. A slight undulation of the pavement surface is sometimes induced by the torsional forces at the lug. Since this treatment relies on the passive resistance of the soil, it is not effective where cohesionless soils are encountered. Figure 2 shows a typical lug anchor terminal treatment.



NOTES :
Weld beam stiffener to ends of beams
Weld shear connectors to flange and web of beam

FIGURE 1 - Recommended WF Steel Beam Terminal Joint Design

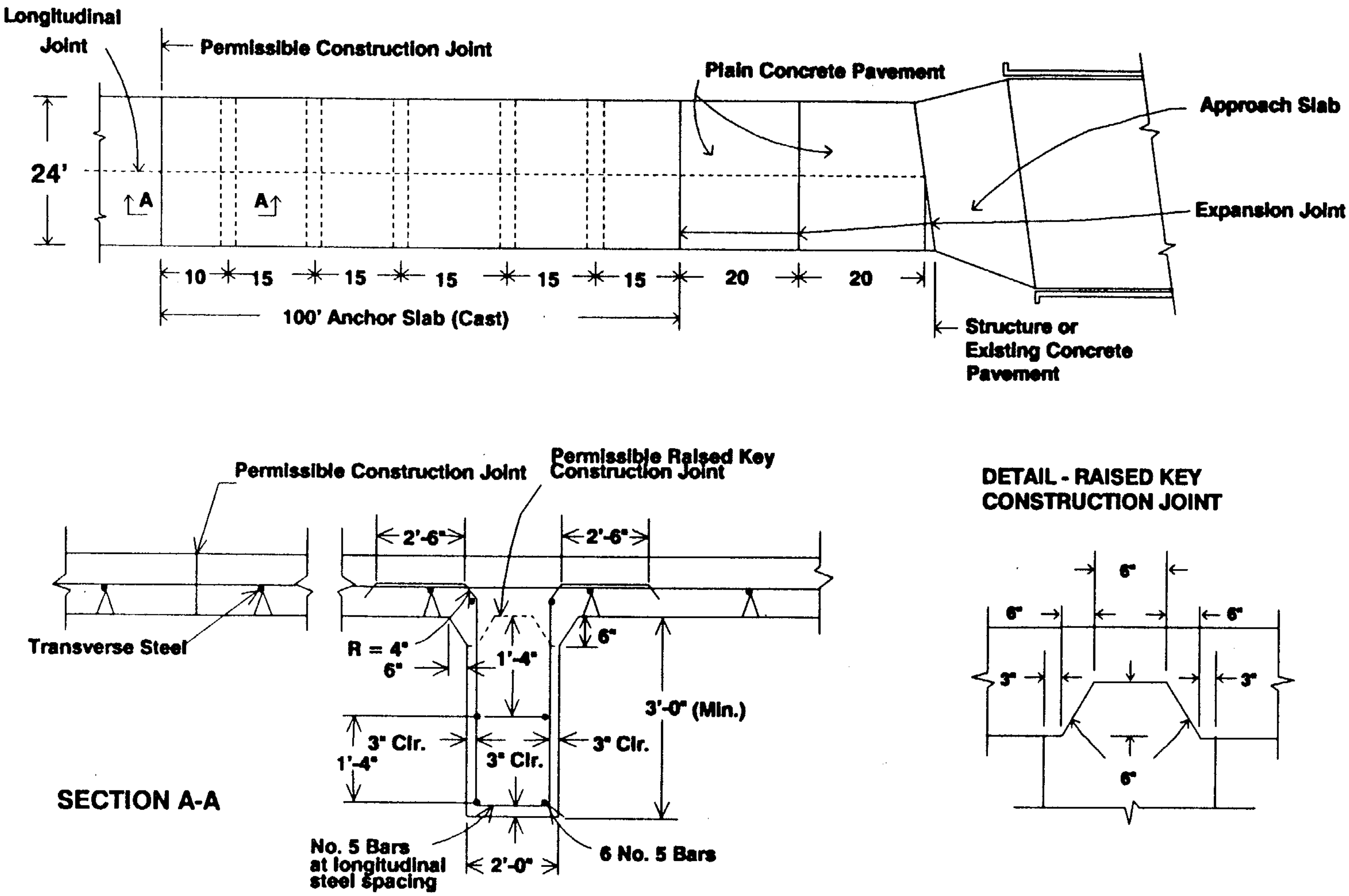


FIGURE 2 - Lug Anchor Treatment

(3) Transverse Construction Joints

(a) A construction joint is formed by placing a slotted headerboard across the pavement to allow the longitudinal steel to pass through the joint. The longitudinal steel through the construction joint is increased a minimum of one-third by placing 3-foot long shear bars of the same nominal size between every other pair of longitudinal bars. No longitudinal steel splice should fall within 3 feet of the stopping side nor closer than 8 feet from the starting side of a construction joint. Refer to paragraph 4b(1)(e) for recommended splicing patterns. If it becomes necessary to splice within the above limits, each splice should be reinforced with a 6-foot bar of equal size. Extra care is needed to ensure both concrete quality and consolidation at these joints. If more than 5 days elapse between concrete pours, the adjacent pavement temperature should be stabilized by placing insulation material on it for a distance of 200 feet from the free end at least 72 hours prior to placing new concrete. This procedure should reduce potentially high tensile stresses in the longitudinal steel.

(b) Special provisions for the protection of the headerboard and adjacent rebar during construction may be necessary.

f. Leave-Outs Temporary gaps in CRCPs should be avoided. The necessity for leave-outs is minimized by giving proper consideration to the paving schedule during project design. The following precautions can be specified to reduce distress in the leave-out portion of the slab in the event a leave-out does become necessary.

(1) Leave-outs require 50 percent more longitudinal deformed bars of the same nominal size as the regular reinforcement. The additional reinforcement should be spaced evenly between every other normal pavement reinforcing bar and should be bonded at least 3 feet into the pavement ends adjacent to the leave-outs. All regular longitudinal reinforcement should extend into the leave-out a minimum of 8 feet. Required slices should be made the same as those in normal construction.

(2) Leave-outs should be paved during stable weather conditions when the daily temperature cycle is small. Because of the closeness of the steel extreme care should be exercised in placing and consolidating the concrete to prevent honeycombing or voids under the reinforcement

(3) If it becomes necessary to pave a leave-out in hot weather, the temperature of the concrete in the free ends should be stabilized by placing an adequate layer of insulating material on the surface of the pavement as described in paragraph 4e(3)(a). The curing compound should be applied to the new concrete in a timely manner. The insulation material should remain on the adjacent pavement until the design modulus of rupture of the leave out concrete is attained.

g. Ramps, Auxiliary Lanes and Shoulders. PCC pavement for ramps, auxiliary lanes, and shoulders adjacent to CRCP is recommended because of the possible reduction in pavement edge deflections and the tighter longitudinal joints adjacent to the mainline pavement. Ramps should be constructed using jointed concrete pavement. The use of jointed pavement in the ramps will accommodate movement and reduce the potential for distress in the CRCP at the ramp terminal. When PCC pavement is used for ramps, auxiliary lanes, or shoulders, the joint should be designed as any other longitudinal joint. Refer to the FHWA Technical Advisory T 5040.29, Paved Shoulders, for further information on proper joint design.

h. Widened Lanes. Widened right lane slabs should be considered to reduce or eliminate pavement edge loadings. This is discussed in the FHWA Technical Advisory T5040.29, "Paved Shoulders".

5. CONSTRUCTION CONSIDERATIONS

a. Many CRCP performance problems have been traced to construction practices which resulted in a pavement that did not meet the previously described design recommendations. Because CRCP is less forgiving and more difficult to rehabilitate than jointed pavements, greater care during construction is extremely important. Both the contractor and the inspectors should be made aware of this need and the supervision of CRCP construction should be more stringent.

b. Steel placement has a direct effect on the performance of CRCP. A number of States have found longitudinal steel placement deviations of ±3 inches in the vertical plane when tube feeders were used to position the steel. The use of chairs is recommended to hold the steel in its proper location. The chairs should be spaced such that the steel will not permanently deflect or displace to a depth of more than 1/2 the slab thickness. An example chair device is shown in Figure 3.

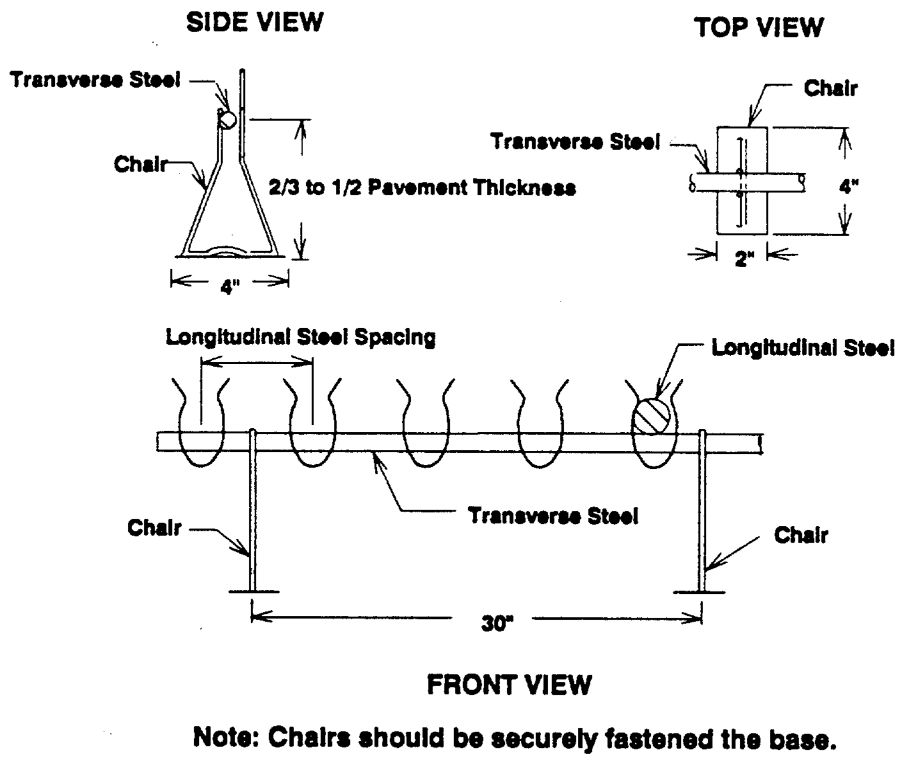


FIGURE 3 - Combination Chair and Transverse Steel Detail

c. Procedures should be implemented to ensure a uniform base and subgrade. Soft spots or gradeline variations should be repaired and corrected prior to concrete placement. Emphasis should be placed on batching, mixing, and placing concrete to obtain uniformity and quality. Strict inspection of batching and mixing procedures is extremely important and may require rejection of batches because of deviations that may have been considered minor under previously existing practices. When placing concrete, adequate vibration and consolidation must be achieved. This is especially critical in areas of pavement discontinuity such as construction or terminal joints. Automatic vibrators should be checked regularly to ensure operation at the specified frequency and amplitude and at the proper location in the plastic concrete. Hand-held vibrators should be used adjacent to transverse joints. Any concrete which exhibits signs of aggregate segregation should be replaced immediately.

d. Inspection procedures are needed to ensure that final reinforcing splice lengths and patterns, as well as bar placement, are consistent with the design requirements. Special precautions should be taken to prevent rebar bending and displacement at construction joints. When leave-outs are necessary, they should be constructed in absolute conformity to the design requirements. Longitudinal joints should be sawed as early as possible to prevent random cracking. This is especially true in multi-lane construction. Sawing should not begin until the concrete is strong enough to prevent raveling.

e. Asphalt concrete patches are not recommended as a temporary or a permanent repair technique because they break the continuity of the CRCP and provide no load transfer across the joint.

Anthony R. Kane
Associate Administrator for Engineering
and Program Development

Attachments

EXAMPLE PROBLEM

The design engineer should perform the following calculations to ensure that the bond between the reinforcing steel and the concrete and the longitudinal steel spacing meet the criteria in paragraph 4c. The equation to determine the ratio of bond area to cubic inches of concrete is as follows and the equation to determine the minimum longitudinal steel spacing follows it:

$$R_b = \frac{(n) \times P_s \times (L)}{(W) \times (t) \times (L)}$$

Where:
- P_s = Perimeter of Bar (in.)
- L = Length of slab = 1"
- W = Width of slab (in.)
- t = Slab thickness (in.)
- n = Number of Longitudinal Bars

Given : #6 reinforcing bars, therefore P_s = 2.356" and Bar Area = 0.44 in.^{2}
W = 12'
t = 10"

Assume: 0.6% steel

Determine: The required minimum area of steel and the required minimum number of bars

Area of Conc. = 10 x 144 = 1440 in.^{2}

Required steel = 0.006 x 1440 = 8.64 in.^{2}

Minimum number if bars required (n) = 8.64 / 0.44 = 19.6 bars, say 20 bars

Determine: The minimum ratio of bond area to cubic inches of concrete.

$$R_b = \frac{(20) \times (2.356) \times (1")}{(1440) \times (1")} = 0.0327,$$

the minimum ratio of bond area to cubic inches of concrete is met so the minimum spacing should be checked.

Determine: Longitudinal steel spacing should be checked as follows:

$$S_b = \frac{(W)}{(n)} = \frac{144}{20} = 7.2 \text{ in., say 7 in.,}$$

therefore the minimum bar spacing is also met.

REFERENCES (CRCP)

1. "AASHTO GUIDE FOR DESIGN OF PAVEMENT STRUCTURES", 1986.

2. "FHWA Pavement Rehabilitation Manual", FHWA-ED-88-025, September 1985 as supplemented.

3. Mooncheol Won, B. Frank McCullough, W. R. Hudson, Evaluation of Proposed Design Standards for CRCP, Research Report 472-1, April 1988.

4. "Techniques For Pavement Rehabilitation - A Training Course", FHWA, October 1987.

5. "Design of Continuously Reinforced Concrete for Highways", Associated Reinforcing Bar Producers - CRSI, 1981.

6. "CRCP - Design and Construction Practices of Various States", 1981, Associated Reinforcing Bar Producers - CRSI.

7. "Design, Performance, and Rehabilitation of Wide Flange Beam Terminal Joints," FHWA, Pavement Branch, February 1986.

8. Darter, Michael I., Barnett, Terry L., Morrill, David J., "Repair and Preventative Maintenance Procedures for Continuously Reinforced Concrete Pavement", FHWA/IL/UI-191, June 1981.

9. "Failure and Repair of CRCP", NCHRP, Synthesis 60, 1979.

10. Snyder, M.B., Reiter, M.J., Hall, K.T., Darter, M.I., "Rehabilitation of Concrete Pavements, Volume I - Repair Rehabilitation Techniques, Volume III - Concrete Pavement Evaluation and Rehabilitation System," FHWA-RD-88-071, July 1989.

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Washington, D.C. 20590

Subject Headquarter's Pavement Rehabilitation and Design Team - Case Study - Continuously Reinforced Concrete Pavement

Date JUN 22 1987

From Chief, Pavement Division

Reply to Attn of HHO-13

To Regional Federal Highway Administrators
Regions 1-10

We have had several requests from the field for information regarding the performance of various pavements. We have elected to use a case study approach as one method of meeting this desire. The case studies will be based on the Headquarter's Pavement Rehabilitation and Design Team field trip reports. Attached is a copy of our initial effort. It describes a distress problem on a continuously reinforced concrete pavement located in the eastern part of the country. The report provides an insight to the types of details that are commonly examined. The process that was followed can be applied to any pavement rehabilitation project.

As you know, the Team at the request of State highway agencies and our field offices has conducted numerous reviews of pavement distress problems. Typically, the Team has been asked to provide assistance when a pavement has experienced premature distress. The Team's role is to determine the cause of the early distress, recommend alternative strategies for rehabilitation/reconstruction and provide suggestions on how to prevent the distress on this or similar pavement.

The Team has been headed by the Pavement Division and has included members from other Headquarter's offices depending on the technical expertise needed to examine a particular problem. From our viewpoint, there have been tremendous benefits gained by the States and FHWA using this review team concept. The Team has always been willing to provide technical assistance when requested and we reaffirm our commitment to continuing these efforts.

We expect to furnish additional case studies that will be based on upcoming Team field trip reports. If you have any comments on this case study approach or the specifics of the attached report, please do not hesitate to contact us.

Sufficient copies of the report are being provided to your office to permit direct distribution to your division offices.

Norman J. Van Ness

CASE STUDY - CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

BASED ON A

FIELD TRIP REPORT

OF THE

PAVEMENT REHABILITATION AND DESIGN TEAM

BY

PAUL TENG

JOHN HALLIN

DON VOELKER

I. Purpose of Trip

To meet with Region, Division, and State engineers to review a Continuously Reinforced Concrete Pavement (CRCP) distress problem on Route XX and discuss its rehabilitation alternatives.

II. Scope of Review

A field review was conducted on (date). During the review, the State provided the Team with available data of the original design. A closeout meeting with the State engineers was held in the afternoon of (date). Subsequently, the State provided additional information concerning traffic data, paving schedules, corrosion surveys, core logs, chlorides studies, delamination surveys, a Pachometer survey and detailed crack surveys. On (date), the Team met with the State engineers to inspect the cores and discuss our recommendations and conclusions. This report summarizes the Team's comments and recommendations.

III. Contacts

State

XXX

FHWA Division Office

XXX

FHWA Region Office

XXX

FHWA Washington Office

XXX

IV. Background Information

Route XX is a six-lane, divided Interstate facility. The project is 13 miles long and involves the overlay of two Jointed Reinforced Concrete Pavement (JRCP) 24-foot roadways with an unbonded 6-inch Continuously Reinforced Concrete Pavement (CRCP) and the construction of an additional 12-foot right lane of a 9-inch CRCP in each direction. The original dual lane was constructed in 1952 as U.S. Route XX. The overlay and outer lane construction projects were completed in 1974-76.

The original pavement consisted of a 9-inch jointed reinforced concrete pavement on a 6-inch Type II (crushed aggregate) subbase constructed on an A-5 silt soil. Transverse joints were sawed at 40-foot spacings. A 1-inch bituminous concrete layer was placed on the original pavement to serve as a leveling course, to correct superelevation, and to serve as a bondbreaker to the 6-inch CRCP overlay. The 6-inch CRCP overlay uses #5 deformed bars spaced at 8.5 inches (0.6 percent steel of the cross sectional area). Transverse steel (#4 bars) was spaced at 34 inches and tied underneath the longitudinal steel. The steel reinforcement was supported on metal chairs and the plans specified a cover tolerance of 2 1/2 - 2 11/16 inches.

The added 12-foot wide CRCP lane was placed on the outside of the overlay portion. The new lane is a 9-inch CRCP on a 4-inch crushed granular base. It uses #5 deformed bars spaced at 5.5 inches (0.6 percent of the cross sectional area). Transverse steel (#4 bars) was spaced at 30 inches and tied underneath the longitudinal steel. The steel reinforcement was supported by metal chairs and the plans specified a cover tolerance of 3 3/8 - 3 7/8 inches.

The plans provided the option of constructing the added right lane with the overlay or separately with a keyway. State highway personnel, however, report the keyway construction joint was likely used. The plans indicate there are tie bars in the keyway but we are unable to determine their length or spacing.

Traffic survey data gathered during the overlay design stage (1970) indicate ADT of 10,600-14,050; 12-13 percent trucks; projected 1991 ADT of 40,650-42,450; 12-13 percent trucks.

An extensive soils investigation was conducted during the overlay design stage. A brief review of the soil boring tabulation indicated the presence of silt, and rock fragments occurring within the top 5 feet of roadbed in the location of the widened outer lane. Little information is available for the roadbed beneath the original pavement. Also, there is little information available concerning the condition of the original pavement during design of the overlay.

Three cores of the existing CRCP pavement were recently taken and a petrographic examination was conducted. An analysis of the cores is found in Section VI.

A French drain was installed where the original roadway was in sag alignment and then daylighted to the reconstructed 6:1 side slopes. State personnel have stated these are not presently functioning.

The inside shoulder width is 4.0 feet. The outside shoulder is 10.0 feet. Each is a 3-inch thick bituminous concrete shoulder on a 7-inch dense-graded stabilized aggregate subbase. The shoulder joint was designed to be sealed with hot-poured, rubber-asphalt joint sealing compound.

Personnel who were present during the overlay construction projects reported that the project was shut down several times because of fines and clay balls in the aggregate. Also, there were difficulties in finishing the slab. The westbound overlay (middle and high speed) lanes were placed in May-July period. The eastbound overlay lanes were placed in October and

November when there were reported temperature variations as high as 40 degrees.

The pavement opened to traffic in late 1975. Pavement failures in the widened (right) lane of the eastbound roadway were noted in February 1976. The number of failures increased substantially during March and April. The State acted quickly in formulating a plan for investigating the causes of the failures.

In June 1976, a report was issued that included data from an extensive field study. However, the study only examined project data related to the widened lane and does not discuss cracks, subbase, soils, concrete quality, etc. of the overlay section of the project since, at that time, there was little distress in the overlay section. Briefly, the report concluded the failures were design associated and included the following: a) inadequate pavement support and the inability of the granular base to drain water away from under the pavement could have resulted in lower stability, b) adverse climatic conditions had reduced the concrete maturity at an early age and resulted in formation of closely spaced transverse cracks, c) the nature of the chairs and poor workability of the concrete could have contributed to the voids and weaknesses in the concrete cross section.

A second report was prepared in June 1982 and attempted to expand on what was learned from the 1976 study. At the time, the westbound lanes of Route XX had performed satisfactorily whereas the eastbound lanes had exhibited distress. The study concluded: a) pavement failures were primarily in these outside (widened) lanes, b) percent of steel reinforcement met the specifications, c) the pavement thicknesses were within the specification tolerance, d) numerous voids were reported in the lower half of the slabs but have not contributed significantly to the failures, e) the CRC overlays were performing satisfactorily and no visible signs of distress were noted, f) a significant portion of the CR-6 subbase material had a high content of fines that led to poor drainage characteristics, g) percent of air entrainment for eastbound and westbound lanes was within specification limits, h) unfavorable curing temperatures were present for the eastbound lanes, i) the crack spacing for the eastbound lanes was in the range of 2 feet apart while the range in the westbound lanes was generally 4-16 feet, and j) the eastbound shoulder lane exhibited high deflection characteristics.

By 1986, it was noted that a large number of areas of pavement distresses were beginning to occur in the middle lane and a lesser number were showing up in the high speed lane. The FHWA Division Office on (date), sent to the State a special report on the Route XX pavement distress and requested a detailed investigation of the pavement to determine the most cost-effective type of repair to be undertaken and determine what lessons could be learned and applied toward other proposed CRCP projects.

V. Details of Field Review

It was noted there were a significant number of punchouts in the middle lanes in both the eastbound and westbound roadways. Of particular interest was the radial cracking around the punchouts. Also, these areas were primarily concentrated in the wheel paths and appeared to be clustered. There were often long sections of pavement that appeared sound followed by sections of distressed pavement. The transverse distress consisted of fatigue cracks resulting in delaminated sections of pavement. These appeared to be located in the vicinity of the transverse bars.

The Team noted several areas of fine longitudinal cracks that appeared to be spaced at approximately the spacing of the longitudinal reinforcing steel; however, there was no evidence of staining of the pavement from possible corrosion of the reinforcing steel.

The full depth patching previously performed by State contractors appeared to be satisfactory. Discussions with State personnel indicate the patching details are in accordance with the state-of-the-practice procedures. Another patching project was initiated in late 1986 but shut down before repairs were completed. Operations are not expected to resume because of limited available funds to complete the necessary work; therefore, the Team was unable to observe the patching operations or the condition of the reinforcing steel.

There are several locations in all lanes where asphalt has been placed over distressed areas as a temporary measure. It is apparent that water is infiltrating the repaired areas.

The outside shoulders in both eastbound and westbound roadways were extremely distressed. In the westbound lanes, alligator cracking was noted to be particularly severe on the section between US XX to west of Route XX. There is a truck weighing station located on WB Route 99 near Route XX. Several trucks were parked on the shoulder, and the weigh station was open. State personnel informed us this was common practice. Also, the top of the shoulder had settled below the top of the mainline pavement.

It appeared the joint had been properly sealed; however, since the shoulder pavement was severely distressed, water is likely infiltrating the pavement structure from below the shoulder surface.

In the eastbound lane, there were large areas of full-depth patching that had been performed under previous maintenance contracts. The quality of the patches appeared to be good. The two outside lanes, but particularly the middle lane, appeared to be showing signs of severe distress at some locations. The previously mentioned punchouts, with their radical cracking patterns, were numerous. The punchouts were centered in the wheel paths. Where the high speed lane exhibited punchouts, it was noted that distressed areas of the center lane were nearly adjacent but staggered from these punchouts.

In the westbound roadway, the most distressed areas were found in the outer and middle lanes. The crack spacing pattern appeared to be acceptable. There were a few hundred feet of a longitudinal crack in the middle lane that was observed to be located about 2 feet inside the joint with the outer lane. There was some concern by project personnel that these cracks were related to the overall pavement distress; however, the team believes while these are undesirable, they are not affecting overall pavement performance.

VI. Discussion Items

We reviewed information on traffic data and a paving schedule for the roadway overlay and outer lane widening work.

Comparing the traffic data for 1970 and 1991 on the cover plan sheet for Project No. XX traffic counts, it appears the forecasted ADT's are within generally accepted margins of error. However, it appears the percent of truck traffic and number of equivalent 18-kip single axle loads being placed on the pavement has increased significantly over the projected loadings. Based on the 1970 traffic survey data, the percent of trucks was 12 percent and projected to remain at 12 percent in 1991. Recent loadometer data from a weigh-in-motion station on the eastern end of the project indicate the percent of trucks may be as high as 21 percent and have average 18-kip equivalent truck load factors as high as 3.76. The current lane truck distribution information indicates there is slightly higher than usual percentage of trucks in the middle lanes. This combination causes heavier than expected loadings on the 6-inch CRCP overlay.

There had been a concern that the weather conditions during placement of the overlay projects affected pavement performance. Two of the three overlay projects had substantial sections of concrete placed in the Fall when there were reported large temperature variations. The State supplied the Team with a paving schedule and reported daily temperatures from the projects' records. We compared this information with identified distress condition survey data. However, we were unable to correlate the two because of the inability to conclusively identify project station numbers with the mileposts shown in the condition survey data. Due to time constraints, we did not pursue this analysis.

The State performed an in-depth evaluation of three 200-foot sections of Route XX. The selected sections were believed to have been low, medium, and high distress areas. The evaluation consisted of a corrosion survey, core logs, longitudinal delamination survey, transverse delamination survey, chloride test results, and steel and chemical tests.

The distress information we received indicates there is extensive corrosion occurring in the three 200-foot test sections. The average chloride content amounts at the depth of the reinforcing steel were 3.1 lbs./cu. yd. for the low distress area, 3.3 lbs./cu. yd. for the medium distress area and 4.5 lbs./cu. yd. for the high distress area. According to accepted practice, chloride contents above 1.5-2.0 lbs./cu. yd. indicate a high potential for corrosion.

The corrosion surveys conducted on these sections show there is a high probability that steel corrosion is widespread throughout the three sections, and a high potential exists for cracking of the pavement due to corrosion. The current ASTM C876-80 specification indicates the following regarding the significance of the numerical value of the potentials measured:

1) Less than 0.2 volts, there is a greater than 90 percent probability that no reinforcing steel corrosion is occurring.
2) Between 0.2 and 0.35 volts, corrosion activity is uncertain.
3) Over 0.35 volts, there is a greater than 90 percent probability that corrosion is occurring.

Also, in laboratory tests where potentials were greater than 0.5 volts, approximately half of the specimens cracked due to corrosion activity.

The delamination survey data for the three sections indicates substantial areas of delaminated concrete in the medium and high distress areas.

The Pachometer survey data for these three sections indicates a range of concrete cover from 1.5 inches to 3.75 inches in the overlay areas. The plans specified a tolerance of 2 1/2 - 2 11/16 inches in the overlay area.

Based on our field observations of the cracks and distresses in the pavement and the above distress survey information, we believe there is a significant amount of corrosion of the reinforcing steel in each of the three test sections. There may be a somewhat lower level of corrosion activity in the low distress area as compared to the medium and high areas, nevertheless, extensive corrosion is likely occurring.

In reviewing the cores from the three test sections, it is apparent that the corrosion activity is predominately in the transverse bars. Nearly without exception, vertical and horizontal cracks were present where transverse bars were experiencing even minor corrosion. See Attachments 1 and 2 for illustrations. The vertical crack above the bars likely contributed to creating an environment that allowed corrosion to begin.

The Team also observed there had been a significant amount of full-depth patching within the project. We understand patching operations have been ongoing the last several years. As stated above, the outer lanes, particularly the EB outer lanes, experienced early distress, and extensive patching was already done in these lanes. Generally, the patches appeared to be performing satisfactorily. Data showing the rate of deterioration on this project was not available.

State personnel informed us they believe there has been a significant increase in the number of distress areas in the overlay areas (middle and fast lanes) in the last couple of years.

The results of the petrographic eximination are given in Attachment 3. Briefly, none of the cores contained reinforcing steel. They all appeared to contain sound concrete. One core, however, appeared to have a shale-type, limestone aggregate while the others had a marble-limestone aggregate. The Team did not have any data that showed whether one or more sources of aggregate were used on the project. State personnel who were present during construction indicated there was only one aggregate source. They did believe the project was shut down several times due to mudballs in the aggregate; however, there was no evidence of this problem in the cores we received.

VII. Recommendations and Conclusion

Information on this project was provided by the State. The information consisted of condition data only for the 6-inch CRCP overlay section (middle and high speed lanes). It must be recognized that we based our proposed alternatives on this limited information.

There are strong indications this pavement is rapidly deteriorating. The relatively thin (6-inch) CRCP overlay specification called for 2 1/2 - 2 11/16-inch cover over the reinforcing steel. The pachometer survey showed there were some areas with as little cover as 1.5 inches and many areas where the cover was in the range of 2 to 2.5 inches. The chloride studies, delamination surveys, and corrosion surveys show there is widespread corrosion in the test sections. The cores confirmed that there was active corrosion in the transverse bars.

If the corrosion occurring in the test sections is representative of the entire project, there is probably very little which can be done to prevent the disintegration of the pavement. Assuming this is the case, the ultimate solution to this problem is a reconstruction alternative.

A detailed distress and corrosion survey of the entire project needs to be made to quantify the amount of pavement that needs repair. An economic analysis should then be made to determine if it is more cost effective to continue heavy maintenance by patching the currently distressed sections or immediately reconstructing this section. Alternatives that need to be considered:

1. Continue heavy maintenance by patching identified distressed sections.

2. Remove the existing PCC and construct a new pavement section. The existing pavement may be suitable for recycling into a new PCC or AC pavement. If it is recycled, we believe the State will need to remove the existing pavement structure (including the original roadway), and construct an adequate drainage system. If a new PCC

pavement is constructed, we also recommend using full-depth, tied concrete shoulders.

3. Construct a minimum 4-inch asphalt overlay. The AC overlay must be thick enough to bridge the distressed areas. Also, we recommend the State perform repairs of currently distressed areas before placing an overlay. Depending on the thickness of the overlay, this may be only a short range improvement given the amount of corrosion activity occurring in the test sections. Experience in another State with overlays of 2 inches on corrosion-distressed CRCP has shown good performance for only 3-4 years. They are presently placing 4-to 6-inch overlays on CRCP and expect to get 10 years of service life.

There is also a need to improve the shoulders on the project, particularly the WB outside shoulder east of the truck weigh station. It is evident the trucks are stopping on the shoulder, presumably to avoid passing through open scales.

From our discussions with the State engineers, there was a recognition that the thin CRCP overlay, corrosion of the transverse bars and heavier than expected traffic loadings are major contributors to the distress. The State may wish to consider using epoxy-coated reinforcing steel on an experimental basis in future CRCP pavements. Another State has placed some CRCP having epoxy-coated reinforcing steel. We are not aware they are experiencing any problems, however, the pavements have not been in place long enough to judge long-term performance.

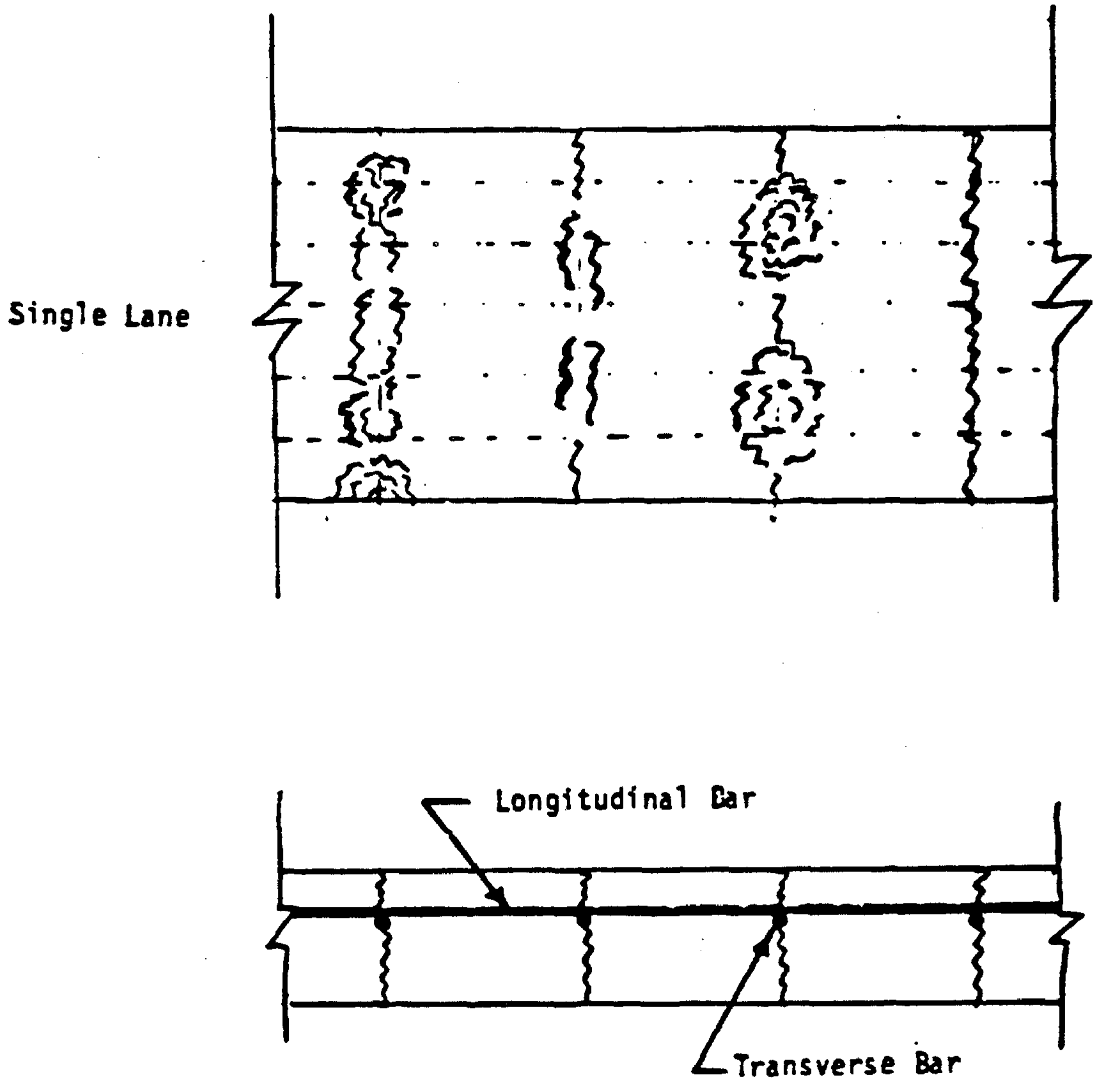
VII. Closing Remarks

It is highly desirable to determine the cause of the distress prior to developing feasible rehabilitation alternatives to ensure that the selected strategy corrects the cause of distress. For any pavement rehabilitation project, the States are encouraged to follow the approach for an engineering and economic analysis as outlined in Mr. Barnhart's November 15, 1983, memorandum. Briefly, this includes the following steps:

(a) Establish existing condition of pavement.
(b) Identify distress.
(c) Determine cause of distress.
(d) Develop feasible alternatives.
(e) Conduct economic (life cycle cost) and engineering analysis of each alternative.
(f) Select most appropriate alternative.
(g) Design alternative.
(h) Provide feedback on performance.

We believe this a logical, practical approach to addressing pavement rehabilitation projects. Our observations recommendations and con lusion are based upon a limited review. We do not feel that we can briefly examine a pavement in a short time period and conclusively give the State the ultimate solution for a problem their engineers have been investigating for many months. We appreciate the opportunity to provide an outside opinion and to provide items for consideration. We hope our visit was as beneficial to the State as it was to us.

During the field trip, we observed a close working relationship among our Regional Office, Division Office and the State. We think this spirit of cooperation is excellent, and we look forward to continuing to work with the State and our field offices whenever we can provide assistance.



CRCP DISTRESS PATTERN

Not to Scale

US Department of Transportation

Federal Highway Administration

Memorandum

Subject: Lateral Load Distribution and Use of PCC Extended Pavement Slabs for Reduced Fatigue

Date

From: Chief, Pavement Division
Washington, D.C. 20590-0001

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Attached are two copies of a report entitled "Lateral Load Distribution and Use of PCC Extended Pavement Slabs for Reduced Fatigue." The report was written by Mr. Mark Sehr, Highway Engineer Trainee, and the final editing was done by ERES Consultants.

The paper summarizes data and findings from several studies on Lateral Load Distribution and Load Stress at Pavement Edge. It includes discussion of the advantages of extended (or widened) lanes for PCC pavements and their effect on stress, strain, deflection, and PCC pavement deterioration.

We believe Mr. Sehr has prepared an excellent report and that it should be distributed to the division offices and shared with the States. We don't, however, have a sufficient number of copies to accomplish the desired distribution. Feel free to make copies or contact Mr. Donald Petersen at FTS 366-2226 to arrange for the printing of additional copies, or if you have any questions concerning the report.

Louis M. Papet

Attachment

LATERAL LOAD DISTRIBUTION

AND

THE USE OF PCC EXTENDED PAVEMENT SLABS

FOR REDUCED FATIGUE

by

Mark Sehr
Assistant Regional Pavement Engineer

June 16, 1989

FEDERAL HIGHWAY ADMINISTRATION
REGION 10

A paper prepared as a project during an Assistant Regional Pavement Engineer Training assignment.

NOTICE

PURPOSE

This paper summarizes data concerning lateral wheel distributions and presents conclusions based on that data. It also examines the advantages of extended (or widened) portland cement concrete (PCC) pavement slabs in terms of their effect on stress, strain, deflection, and PCC pavement deterioration.

BACKGROUND

Early road widths were only 15 ft (4.6 m), wide enough to handle the demands of horse-drawn vehicles. Following the discovery of the internal combustion engine and the development of motorized vehicles, traffic steadily increased. The width of roadways increased to 16 ft (4.9 m), and then to 18 ft (5.5 m). By the late 1920's, primary paved roadways were needed and the construction of 10 ft (3.0 m) lanes (20 ft [6.1 m] roadway) were standard practice. Today, conventional designs use 12 ft (3.7 m) lanes as standard practice.

PAST STUDIES AND COMMENTS ON THE DATA

The lateral location of traffic in the travel lane is the criteria to determine that a 12-ft (3.7 m) channelized lane is wide enough to withstand the repetitive loads of heavy truck traffic, and there have been several studies to determine the lateral truck wheel distribution in the pavement lane. These studies were generally initiated for design and safety concerns. There recently has been consideration of PCC pavement stresses and deflections and their connection with lateral wheel loads and shoulder encroachments. The studies attempt to determine the lateral wheel distribution and evaluate the damage done by differing transverse loads to help designers in building an adequate pavement structure. To summarize the information on lateral wheel distribution and the probability of pavement edge and shoulder encroachment, the results from a number of studies on lateral wheel path traffic distribution are highlighted in the following text.

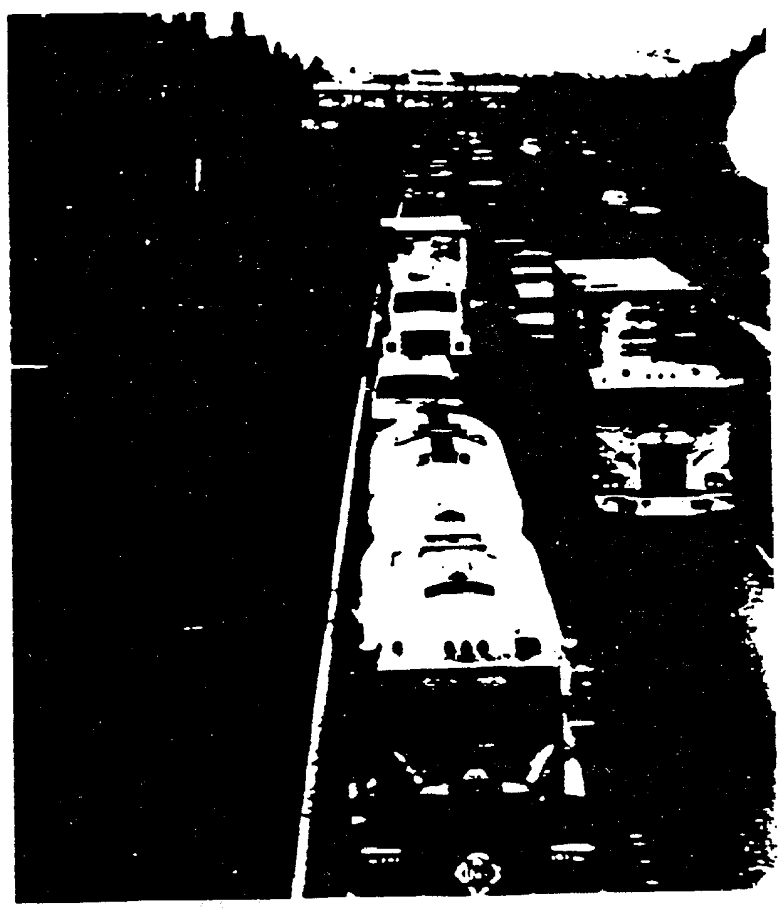
The first study on lateral wheel distribution was completed by Taragin of the Federal Highway Administration in 1958.[1] This data, which is still used in both current PCC and asphalt concrete (AC) pavement design, showed that the highest frequency of travel and mean travel path distance occurred at little more than 2 ft (0.61 m) from the right pavement edge. The findings stated that an average of 2.5 percent of the mainline truck traffic encroached up to 12 in (305 mm) on the outside shoulder of the test section. The findings also stated that about 4 percent of the overall traffic drove closer to the edge than 12 in (305 mm). Taragin's study was completed on 12-ft (3.7 m) pavement lanes with unpaved shoulders.

The applicability of these results for current conditions can be questioned. There is some thought that the unpaved shoulders in the study may have been an artificial deterrent to the trucks encroaching on the shoulder and thus biased these results. Another consideration is that the size, speed, and number of trucks on the road in the 1950's is much less than today (1989). The 2.5 percent truck traffic encroaching on the shoulder is considered low based on current traffic characteristics shown in studies discussed later in the text.

The second study was done by Emery of the Georgia Department of Transportation in 1974.[2] This study found that 53 percent of the overall traffic traveled within 12 in (305 mm) of the pavement edge. It was also observed that at least 2.4 percent of the truck traffic encroached on the pavement edge. Nine percent of the traffic was driving in a 15-in (381 mm) wide wheel path that started 3 in (76 mm) inward from the right pavement edge and extended outward to include 12 in (305 mm) of the shoulder. This data was obtained from PCC pavements with an asphalt concrete shoulder; therefore it was concluded that a visible delineation existed between the pavement and the shoulder.

The data from this study showed that the motorist will drive near the edge of the pavement whenever possible in order to reduce or eliminate the uneasiness of close parallel travel to other vehicles in adjacent lanes.

Photo 1 - Illustration of where vehicles tend to travel. Many vehicles are travelling within 18 inches of the edge of pavement.



A third study was performed by the National Cooperative Highway Research Program (NCHRP 14-3).[3] This project, conducted in Georgia during 1975, used a 10-mi (16.1 km) test section to follow and observe randomly selected trucks. Shoulder encroachment and the longitudinal length of the encroachment were recorded for each selected truck within the test section. A total of 205 trucks were followed in the 10 mi (16.1 km) section in Georgia. The results appear in tables 1, 2 and 3.[3] The length and location of the encroachments is shown in figure 1.[3]

Table 1. Summary of outside shoulder encroachments by type of shoulder pavement (reference 3).

Item	Asphalt Concrete	Bituminous Surface Treatment	Total
Number of Samples	129.0	76.0	205.0
Number of Trucks Encroaching	83.0	50.0	133.0
Percent of Trucks Encroaching	65.3	65.8	64.9
Number of Encroachments	398.0	279.0	677.0
Avg. Encroachments Per Truck Encroaching	4.8	5.6	5.1
Avg. Encroachments Per Truck	3.1	3.7	3.3
Avg. Vehicle Speed, km/h	---	---	103.0

Note: 1 km/h = 0.621 mph

Table 1 is a summary of outside shoulder encroachments by type of shoulder material. Of the 205 trucks observed, 65 percent encroached on the shoulder at least once within the test section. Approximately the same percentage encroached on the different types of shoulders studied (AC and Bituminous Surface Treatment). This seems to indicate that the delineation between a PCC mainline pavement and an asphalt concrete shoulder or the rough surface of a bituminous surface treated shoulder does not necessarily deter trucks from encroachment. A total of 677 shoulder encroachments were recorded, which is an average of 3.3 encroachments per truck, per 10 miles of travel on rural interstate.

Table 2 provides a summary of the number of encroachments on the outside shoulder, by type of terrain. The percentage of trucks encroaching on the shoulder is approximately the same for both a flat and a rolling terrain. There is not enough data on a hilly terrain to make a conclusion, but it would be reasonable to assume that it would be approximately the same as the others.

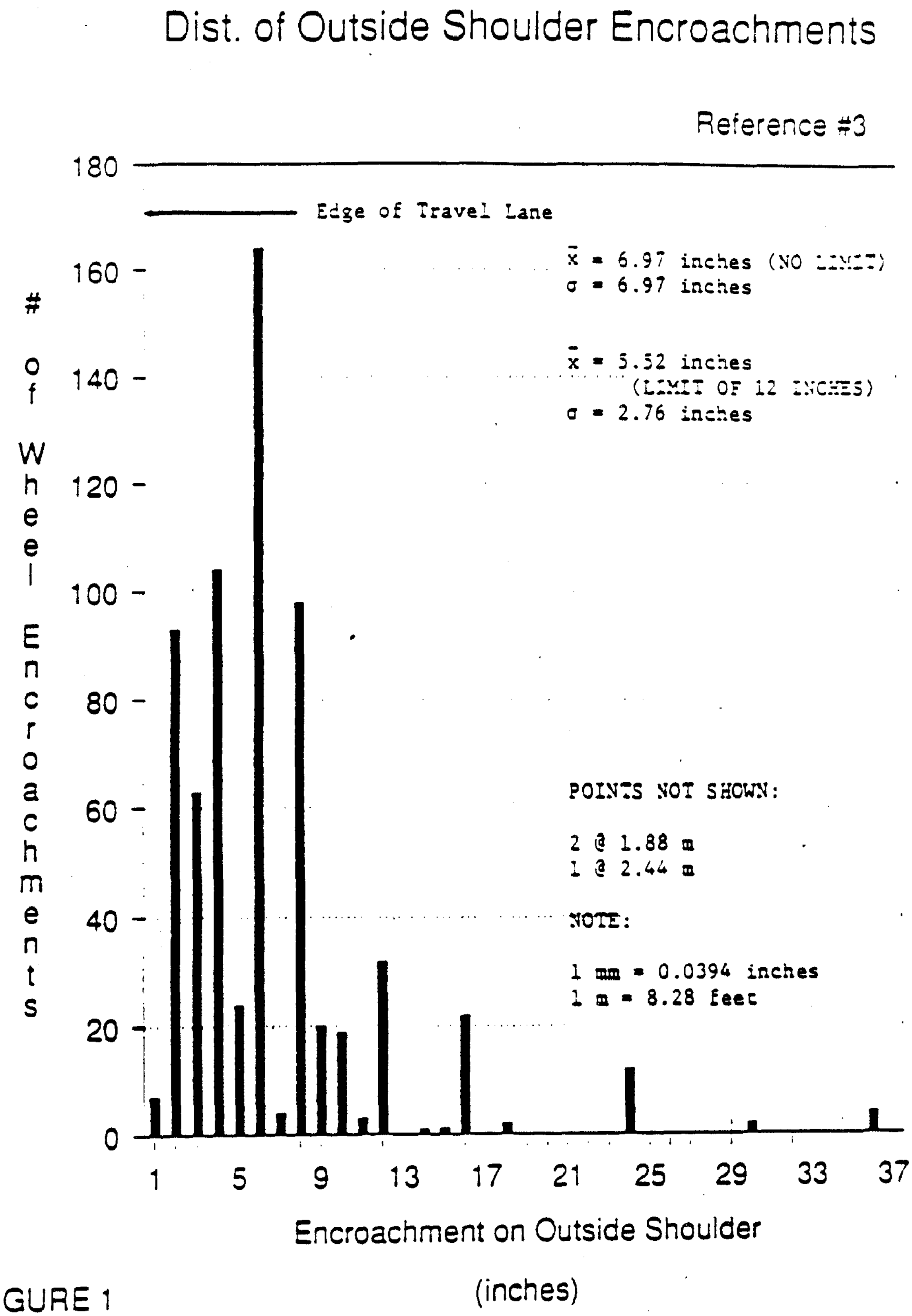

Dist. of Outside Shoulder Encroachments
Reference #3
Edge of Travel Lane
x̄ = 6.97 inches (NO LIMIT)
σ = 6.97 inches
x̄ = 5.52 inches
(LIMIT OF 12 INCHES)
σ = 2.76 inches
POINTS NOT SHOWN:
2 @ 1.88 m
1 @ 2.44 m
NOTE:
1 mm = 0.0394 inches
1 m = 8.28 feet
# of Wheel Encroachments
180
160
140
120
100
80
60
40
20
0
1
5
9
13
17
21
25
29
33
37
Encroachment on Outside Shoulder
(inches)

FIGURE 1

Table 2. Encroachments on outside shoulder, by type of terrain (reference 3).

Type of Terrain	Total Trucks	Number of Trucks Encroaching	Encroaching Percent	Number of Encroachments	Encroachments Per Trucks Encroaching	Average Encroachment Per Truck
Flat	67	43	64.2	190	4.42	2.84
Rolling	134	87	64.9	480	5.52	3.58
Hilly	4	3	75.0	7	2.33	1.75
All Terrain	205	133	64.9	677	5.09	3.30

Table 3 shows that the outside shoulder encroachments occurred for an average of 4.5 seconds. The average encroachment was for a longitudinal distance of approximately 400 ft (122 m) and 0.60 ft (0.18 m) laterally onto the shoulder structure.

Table 3. Average time and distances of outside shoulder encroachments (reference 3).

Item	Outer Shoulder	Median Shoulder
Average encroachments per truck in 10 Miles	3.30	0.25
Average time on shoulder per encroachment, secs.	4.50	3.40
Average longitudinal distance on shoulder per encroachment, ft.	383.86	344.16
Average transverse distance on shoulder per encroachment, ft.	0.59	0.05

Figure 1 is a summary of the transverse encroachments onto the shoulder. As can be seen, the highest frequency of encroaching trucks was approximately 6 in (152 mm) onto the shoulder. A high percentage of times when trucks encroach, their dual tires are literally split between the pavement edge and shoulder.

The fact that shoulder encroachments occur regularly is well illustrated by the Georgia study. In the 10-mi (16.1 km) test section, the average number of encroachments per truck was 3.3. Of the 65 percent trucks encroaching, the average number of encroachments per 10 miles was 5.1 (see table 1[8]). Why do certain trucks go onto the shoulder more than others? The study was done with no cross-winds present, which eliminates an obvious external factor from biasing the results. Truck

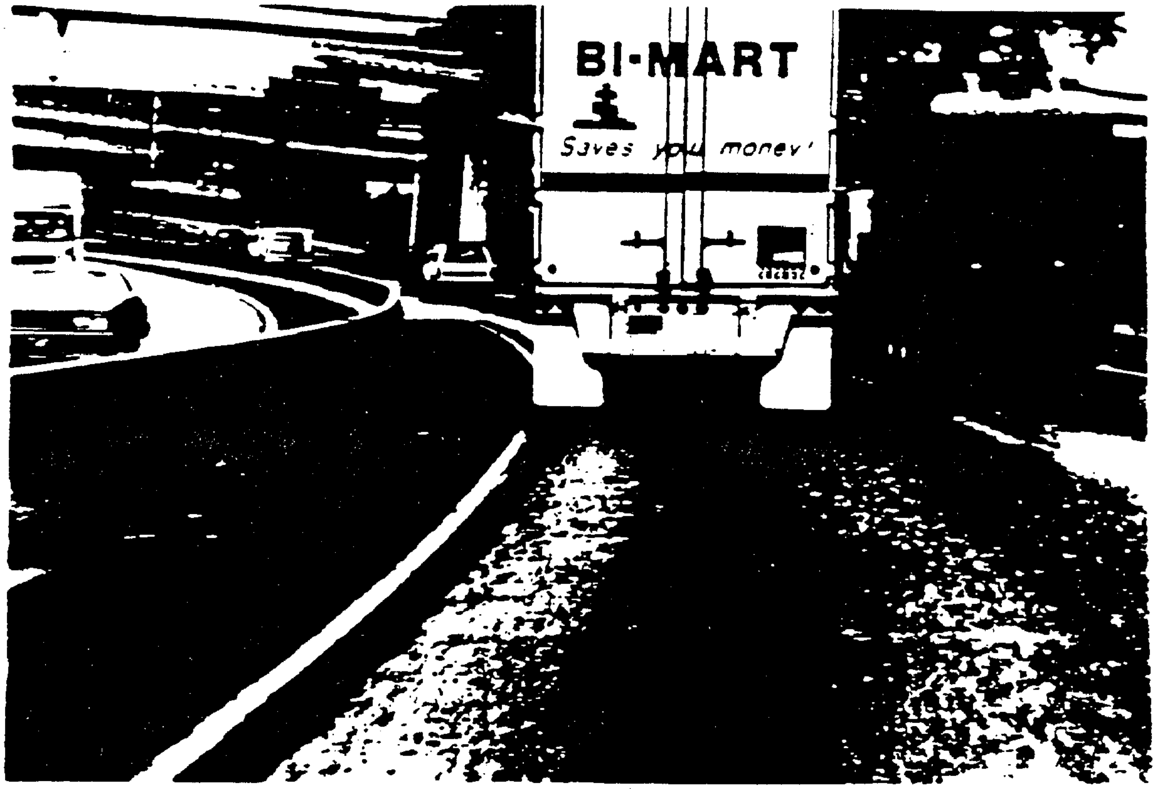


Photo 2 - Dual wheel split between shoulder and edge of PCC pavement. Note the pavement edge stripe location (12-foot PCC pavement slabs).

weight and the mental/physical condition of the driver may be factors, but their effect could not be determined from this study. The terrain seemed to have no effect on the percentage of trucks on the shoulder.

The main point to be made from this research is that, for whatever reason, trucks do encroach on the shoulder or beyond a 12-ft (3.7 m) channelized lane. The 3.3 encroachments per truck per 10 miles is not an alarming figure; however, when multiplied by the total truck miles accumulated, the total number of encroachments quickly adds up. As is discussed in the next section, the greatest damage to PCC pavements is done when loads are placed on the outside edge at the corner or at the midpoint of the slab. Obviously, encroaching truck traffic places loads at the pavement edge. Also, when an edge drop-off is present, the dual wheel splits the lane-shoulder joint and all of the load is transferred to only one wheel. This causes even higher load concentrations at critical points in the pavement structure. Because this edge loading can contribute to failure of the pavement, this is a subject that deserves more attention from the highway community. With the use of microcomputers and mechanistic design procedures, the pavement design engineer can now readily analyze critical load concentrations and calculate the necessary lateral support required to minimize damage and economize on thickness design.

The results presented by Emery were supported by a study done by Texas on the lateral placement of trucks on a highway during 1983.[8] This research concluded that approximately 0.5 percent of the trucks sampled completely encroached onto the shoulder. Complete encroachment is defined as occurring when the dual tire is off the pavement and entirely on the shoulder. Up to 12 percent of the trucks were partially on the shoulder, defined as occurring when the dual wheel is on the pavement and shoulder simultaneously. A main point of this study is that a higher percentage of trucks (12.0 compared to the 2.5 found by Tagarin and Emery) are running on the shoulder and are not being accounted for in current highway design practices.

ILLUSTRATION OF THE MEAN DISTANCE "D" FROM SLAB EDGE TO OUTSIDE OF DUAL TIRES

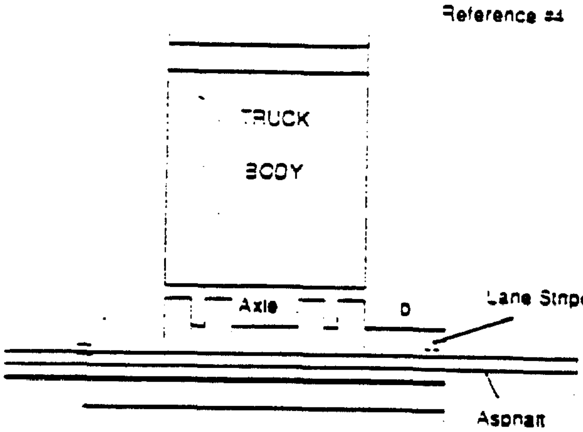


FIGURE 2 Typical control section cross section.

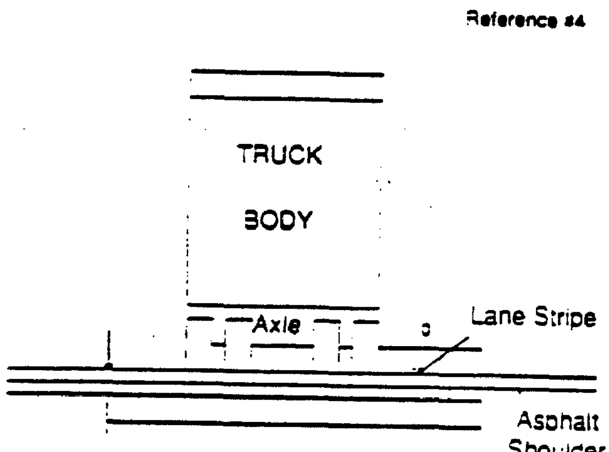


FIGURE 3 Cross section of the extended width concrete pavement.

A fifth study was recently completed by the Transportation Research Laboratory at the University of Illinois in March 1988.[4] This study compared the lateral distribution of truck wheel travel paths on a 12-ft (3.7 m) PCC freeway lane with AC shoulders with two extended PCC pavement slab widths (13.5 ft [4.1 m] and 13.7 ft [4.2 m]) that were also delineated into 12 ft (3.7 m) driving lanes. This is shown in figures 2 and 3.[4]

The extended PCC pavement slab test section was located on 4.1 mi (6.6 km) of rural freeway on I-57 in Illinois. The southbound outside lane was striped as a 12-ft (3.7 m) lane and the extended PCC pavement slab widened 18 in (457 mm) into the shoulder. The PCC pavement extended 18 in (457 mm) beyond the right lane stripe into the outer shoulder. The northbound outside slab extension was 20 in (508 mm), as shown in figure 3.[4] Both directions have asphalt concrete shoulders. Due to site limitations, the data collection for the section was limited to one location.

The type of truck monitored in this study was semi-tractor trailers. Passenger vehicles and smaller single unit trucks have considerably less impact on edge stresses and were not included in the sample. As with the Georgia study, no data was taken on days with a strong cross-wind. Also, the study section was on rural interstate and the terrain was flat, with no horizontal curvature within 2000 ft (610 m) of the test sections, in order to further reduce the effect of external factors. Film from 8mm movie cameras was reviewed by the University of Illinois staff to obtain the data.

A summary of the lateral encroachments on the edge of the pavement is given in table 4.[4] A comparison of the average lateral placement of the truck tires in the control section with the extended PCC pavement slab section leads to the conclusion that trucks will not significantly move outward in the designated traffic lane. Wheel path locations show that trucks tend to drive approximately 2 in (51 mm) closer to the edge line stripe on an extended PCC pavement slab. This would tend to indicate that with adequate slab width, the pavement stripe location controls the lateral wheel path in which the trucks travel rather than the overall width of the pavement itself.

The trucks stayed an average of 20 to 22 in (508 to 559 mm) away from the edge lane stripe, whether on a 12-ft (3.7 m) PCC slab or an extended PCC slab marked with a 12-ft (3.7 m) driving lane. At first, the 20 to 22 in (508 to 559 mm) distance may seem adequate, but after evaluating the data it can be seen that approximately 30 percent of the semi-tractor trailers travel 18 in (457 mm) or less

Table 4. Values of sample size, lateral distance D (figures 2 and 3[4]), standard deviation, and percent shoulder encroachments for the lateral distribution of truck wheel paths (reference 4).

	Sample Size (Vehicle)	D, in (Fig. 2&3)[4]	Distance From Edge Lane Stripe, in	Standard Dev., in	Percent Shoulder Encroachments (Beyond Lane Stripe)
Control Section	536	22.0	22.0	9.0	0.7
18-in Extended Section	691	38.1	20.1	9.1	1.7
20-in Extended Section	613	40.5	20.5	9.1	0.7

from the slab edge. The number of trucks within 12 in (305 mm) of the slab edge is approximately 10 percent (Figure 4).[4] This is a significant number of trucks travelling at the edge of the PCC pavements and, if these results can be considered representative of the rest of the country, indicates that a large number of edge loadings are not being considered in most of the State highway agencies design processes.

It should also be noted that none of the trucks sampled in the widened lane pavement test sections traveled on to the PCC pavement edge or AC shoulder. A small percentage did encroach on the edge lane stripe, but none went to the pavement edge itself.

Figures 4, 5, and 6 show the data collected for each test section and the control section.[4] These once again show that the average distance from the edge stripe is between 20 and 22 in (508 and 559 mm). A significant percent of the truck traffic does travel within 18 in (457 mm) of the PCC slab edge on the control section. The extended PCC pavement slab sections had very little traffic within 18 in (457 mm) of the edge of the PCC slab. This can be seen clearly in the graphs of the distribution of trucks, as indicated by the mean lateral distribution (x).

Photo 3 - A truck travelling near the edge of pavement. Note the pavement edge stripe location (12-ft PCC pavement slabs).

TRUCK LATERAL WHEEL DISTRIBUTION
(control section)

Reference #4

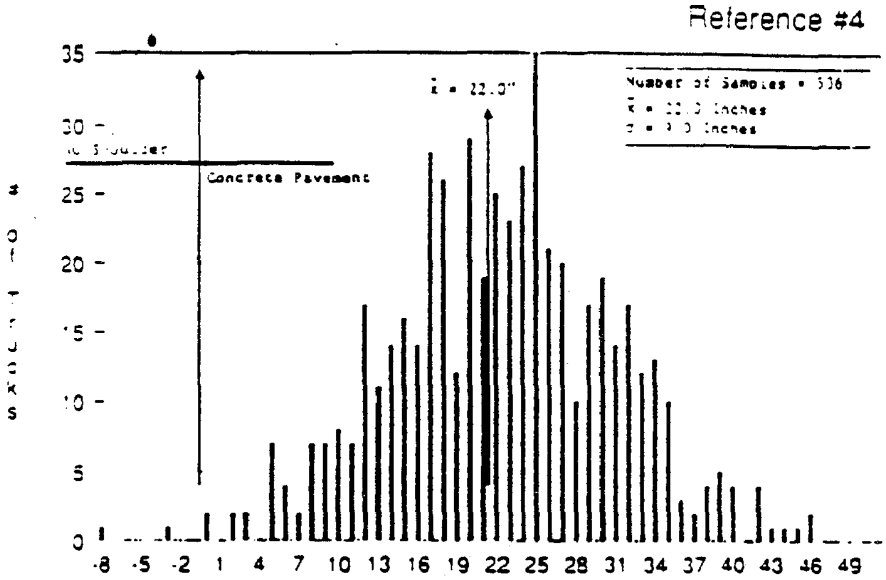


FIGURE 4

LATERAL WHEEL DISTRIBUTION
(Southbound - 18 inch slab extension)

Reference #4

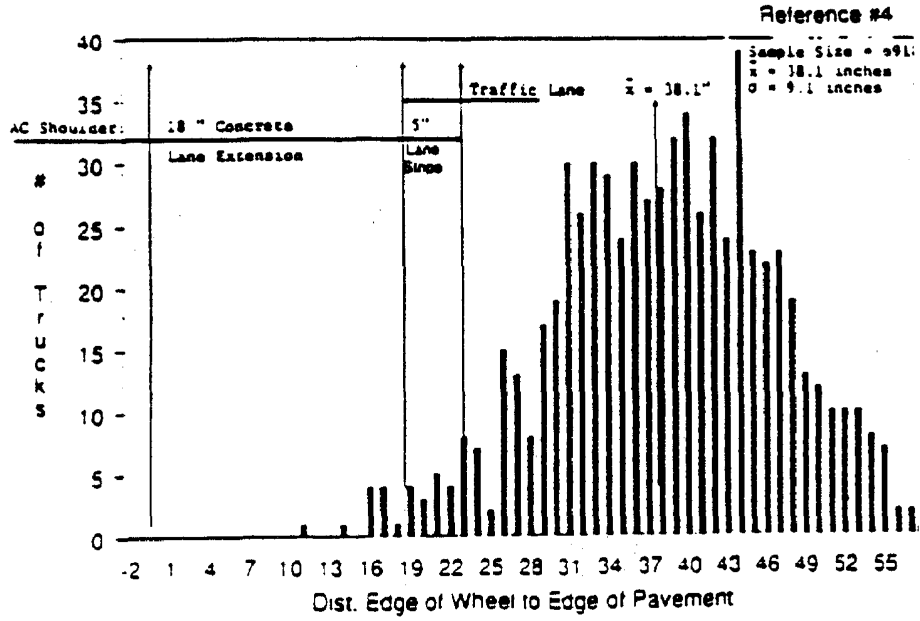


FIGURE 5

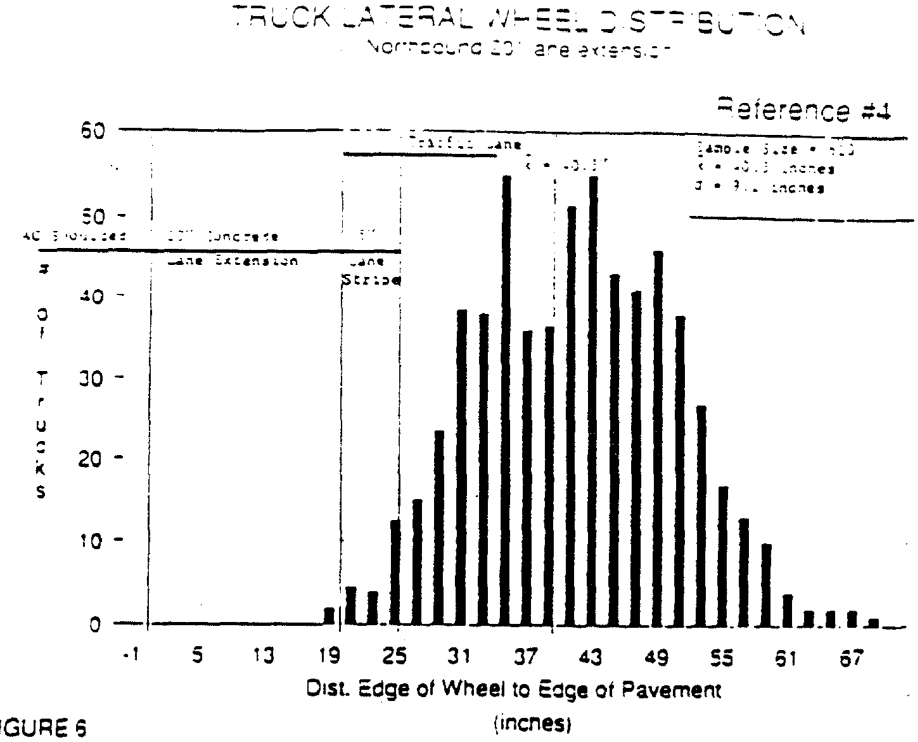


FIGURE 6

Since the extended PCC pavement slabs had little effect on the lateral wheel path of the trucks, it suggests that the pavement edge stripe has a major effect on the traffic distribution. However, opinions on this point differ among maintenance, traffic, design, and construction engineers. If the pavement edge stripe is located more than 12 ft (3.7 m) from the centerline and on an AC shoulder, where will the trucks' wheel paths be located? Will they be 20 in (508 mm) away from the edge of the PCC slab or 20 in (508 mm) away from the edge stripe itseif? It is a question that deserves much more thought by highway agencies.

ANALYSIS OF DATA ON PCC PAVEMENT DETERIORATION

Extended PCC pavement slabs refers to PCC slabs which are built wider than the conventional 12-ft (3.7 m) striped traffic lane. The normal slab extension that has typically been constructed varies from 1 to 3 ft (0.3 to 0.9 m). The basic concept behind the construction of extended slabs is to keep the heavy wheel loads away from the outside edge of the pavement so that traffic loads used in design can be considered interior loads. "Truck wheel loads placed at the outside pavement edge create more severe conditions than any other load position. As the truck placement moves inward a few inches from the edge, the effects decrease substantially."[9] Extended PCC pavement slabs are not, however, a replacement for an adequate shoulder pavement structure. An extended PCC pavement slab should be used with an adequate shoulder structure to meet an agency's own design standards.

In considering the potential benefits from the use of extended pavement slabs, their effect on pavement deterioration should be recognized. In this section, slab stress, strain, deflection, and moisture infiltration are considered as they are related to extended PCC slabs.

Stress and accumulated pavement fatigue are two parameters used to calculate the damage done to a PCC pavement by applied loads. It is widely accepted that "the most critical pavement stresses occur when the truck wheels are placed at or near the pavement edge and midway between the joints."[5] Since the critical stress occurs at the mid-point of the panel, load transfer devices at transverse joints do not have a great influence on the load stresses at the mid-panel. The effect of trucks running at the pavement edge can be shown by the stress-fatigue analysis in figure 7.[5] The fatigue was calculated at various locations on the PCC slab, inward from the slab edge, for different truck wheel load placements. "This factor, when multiplied by edge load stress, gives the same degree of fatigue consumption that would result from a given truck placement distribution."[5] As the lateral truck wheel distribution moves away from the PCC slab edge and inward on the slab, the total number of load repetitions increases, but the damage due to stress decreases. As illustrated by figure 7, the fatigue stress decreases as the percent trucks at the edge decreases.[5]

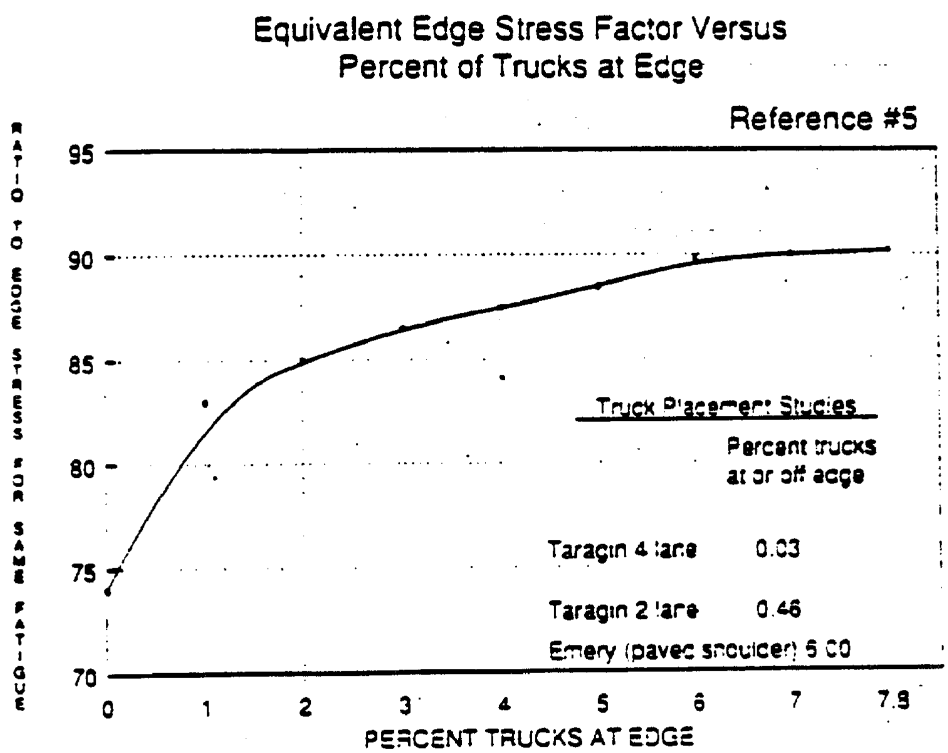


FIGURE 7

A theoretical evaluation of the effect of extended slabs on stress reduction was performed using the PC version of ILLISLAB. A 14 ft by 16 ft (4.3 by 4.9 m) slab of varying thicknesses was modeled on a subgrade with a load applied at the slab's midpoint on the outer edge. The induced stress was then calculated at that point (a 4-in [101 mm] offset, and at further offsets from the edge of 8 in, 16 in, and 24 in (203, 406, and 610 mm) by moving the load inward on the slab. As is shown in figure 8, there is a large reduction in stress that results from moving loads away from the slab edge. This reduction is greater for thinner slabs. Since stress is related to fatigue and, ultimately, deterioration of the slab, this reduction in stress is a desirable goal.

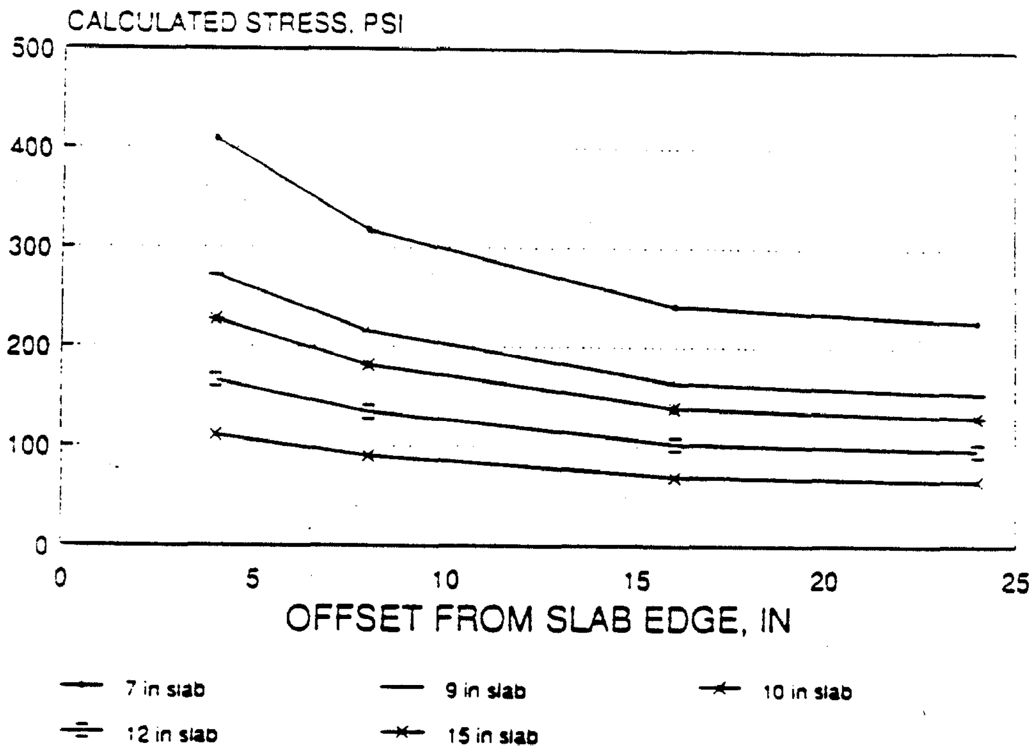


Figure 8

The effect of wheel load placement on pavement slab strains has also been documented. As measured in test sections by the Minnesota Department of Transportation (MnDOT), pavement strains are shown to be greatest at the free edge of the PCC slab (see figure 9).[6] The edge strain reduces quickly as the wheel load moves inward from the edge of pavement, as shown in figure 10 (from a MNDOT laboratory test slab) and tends to level off when the applied load is 18 to 24 in (457 to 610 mm) from the edge of pavement.[6] "In general, free-edge strains were 36 percent to 50 percent greater than interior strains."[6]

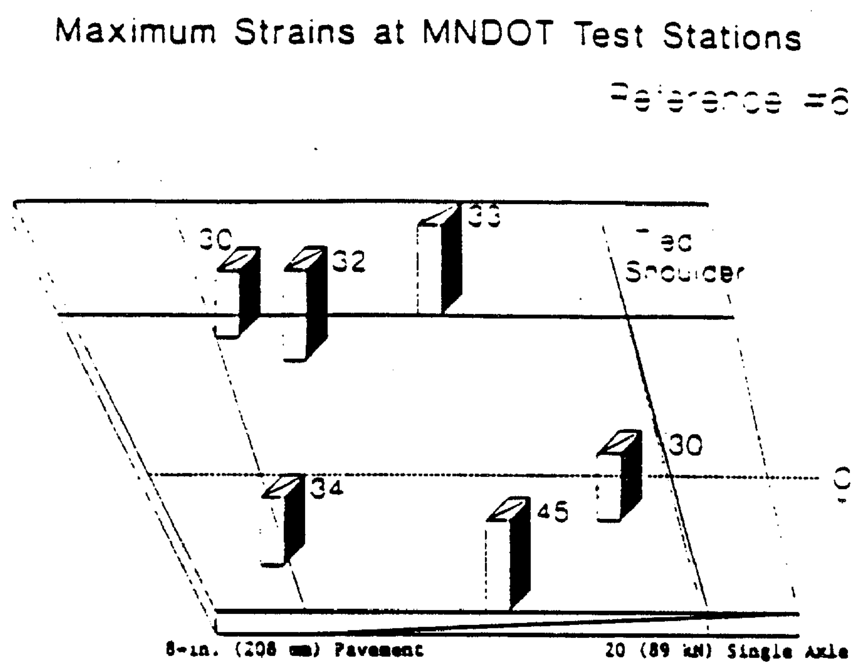


Figure 9

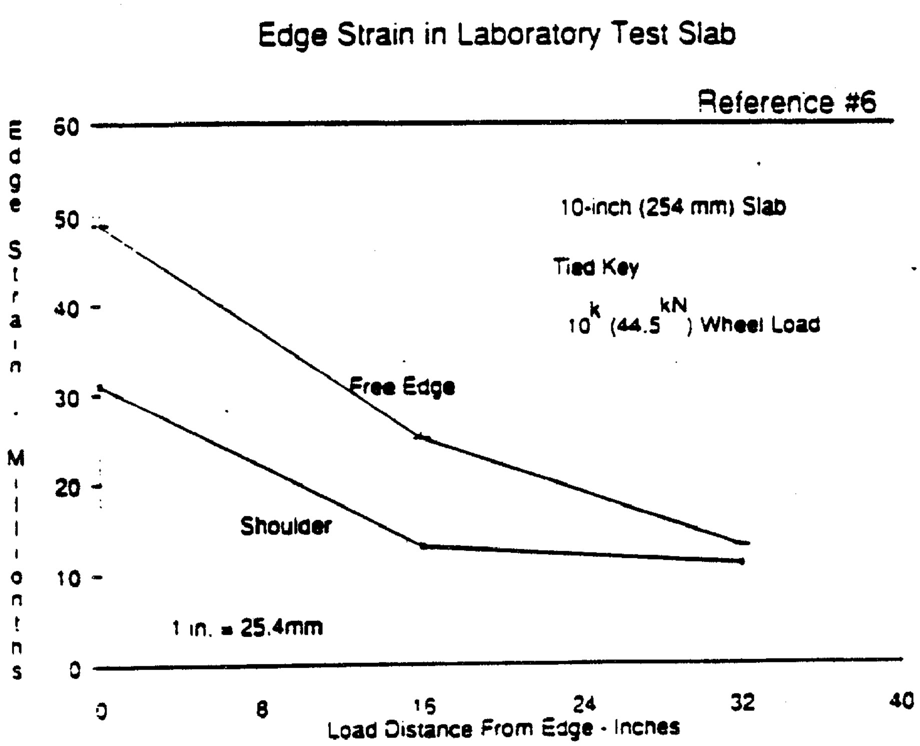


Figure 10

Measurements and established data on the warping and curling of PCC pavements and the effect of extended PCC pavement slabs with 12 ft (3.7 m) channelized traffic lanes is limited. "Warping leaves slabs unsupported for distances of as much as 4 to 5 ft (1.2 to 1.5 m) at slab corners and 2 to 3 ft (0.6 to 0.9 m) at slab edges."[10] The loss of support along the slab edges and the compressive forces of the concrete itself are two adverse effects of warping. Curling refers to the concrete slab behavior due to the differing temperatures in the slab depth. Slabs curl upward (corner support lost) during the night because the temperatures are cooler on the top surface than on the bottom of the slab. Conversely, slabs curl downward (corners downward) during the day due to the warmer temperatures on the top surface of the slab compared to the bottom. There is not enough information available to see any differences in warping and curling with the use of extended PCC pavement slabs.

Water infiltration underneath existing pavements is being emphasized as a major factor in the deterioration of some pavement structures. The use of extended pavement slabs would mean less traffic directly on the longitudinal shoulder joint (either PCC or AC shoulder). The required maintenance of the joint seal should be lower and the seal achieved during construction would function as an effective joint for a longer period of time because of the fewer applied loads. With a better performing joint, there should be less water infiltrating through the longitudinal edge joint to the underlying base material and less potential deflections due to the lateral wheel load location.

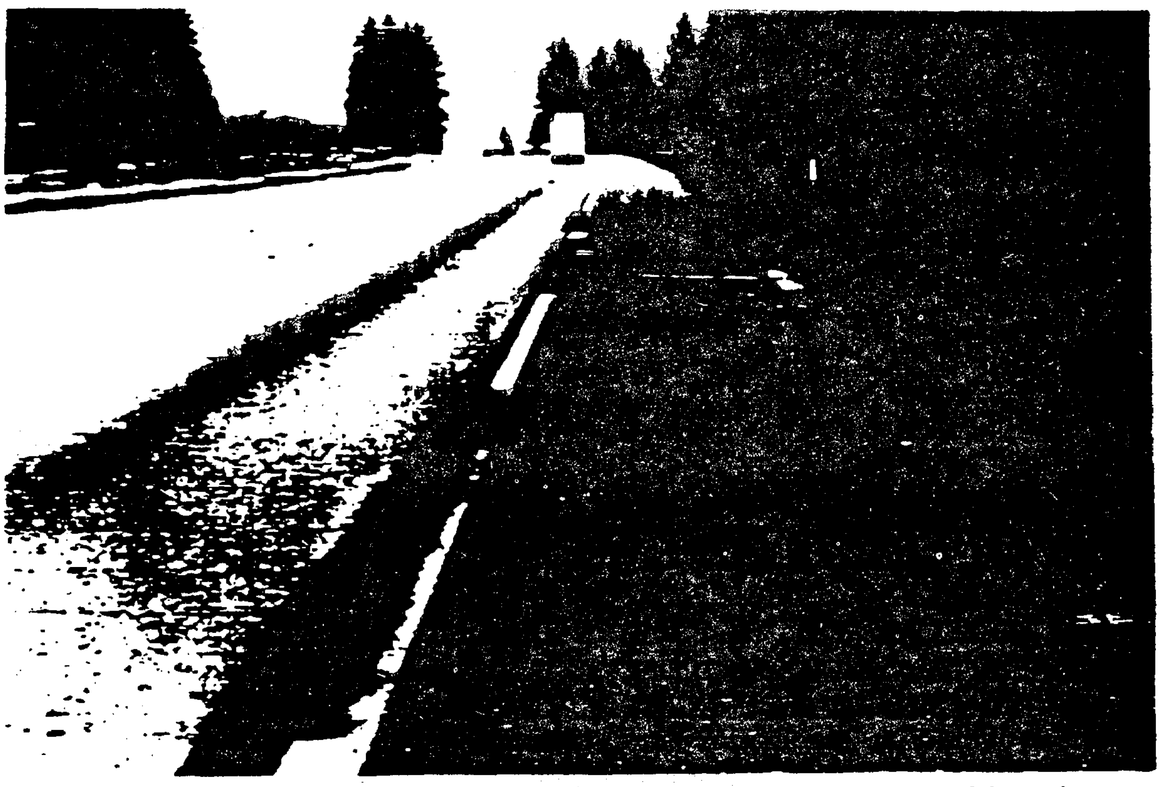


Photo 4 - Illustration of PCC pavement/shoulder joint deterioration. Note the pavement edge line location (12-foot lanes).

The use of extended pavement slabs will lead to improved pavement performance. The extended life will be due to lower edge strains, reduced overall stress, lower edge and corner deflections, and less water infiltration through the longitudinal pavement shoulder joint. It should be possible to design a thinner pavement section with extended pavement slabs and obtain the same performance as that of a thicker pavement without extended slabs. "Results of a study conducted by the Construction Technology Laboratories for the Federal Highway Administration indicate that lane widening is an efficient method of improving pavement response under traffic load."[11] Improved performance can realistically be expected from the use of extended pavement slabs without drastic changes of design. With the correct information available, highway designers will support the construction of extended pavement slabs to extend the life of the pavement, with no decrease in safety or increase in initial (unit price) costs.

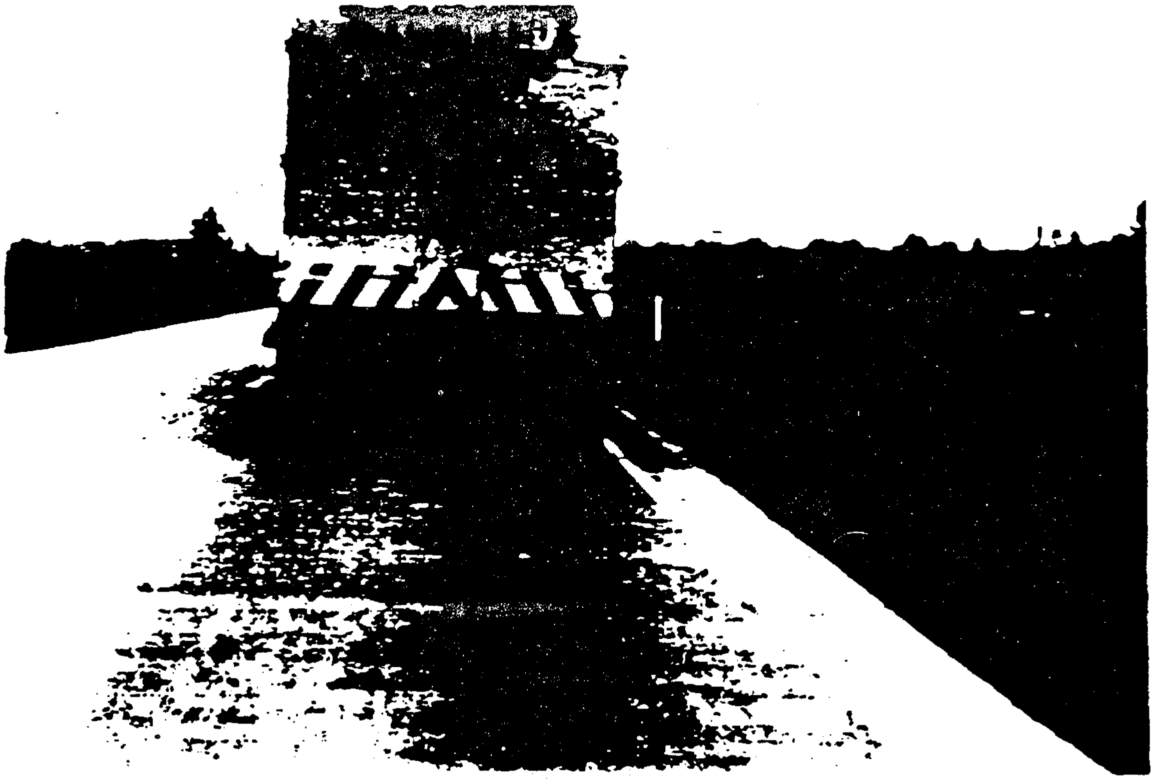
It should be noted that the benefits of extended slabs can only be realized if the slabs are properly striped. Since the stripe appears to control the lateral wheel distribution, placement of the lane-shoulder stripe must be done at the 12 ft (3.7 m) mark and not at the edge of the extended slab or even on the adjacent shoulder.

SAFETY CONCERNS

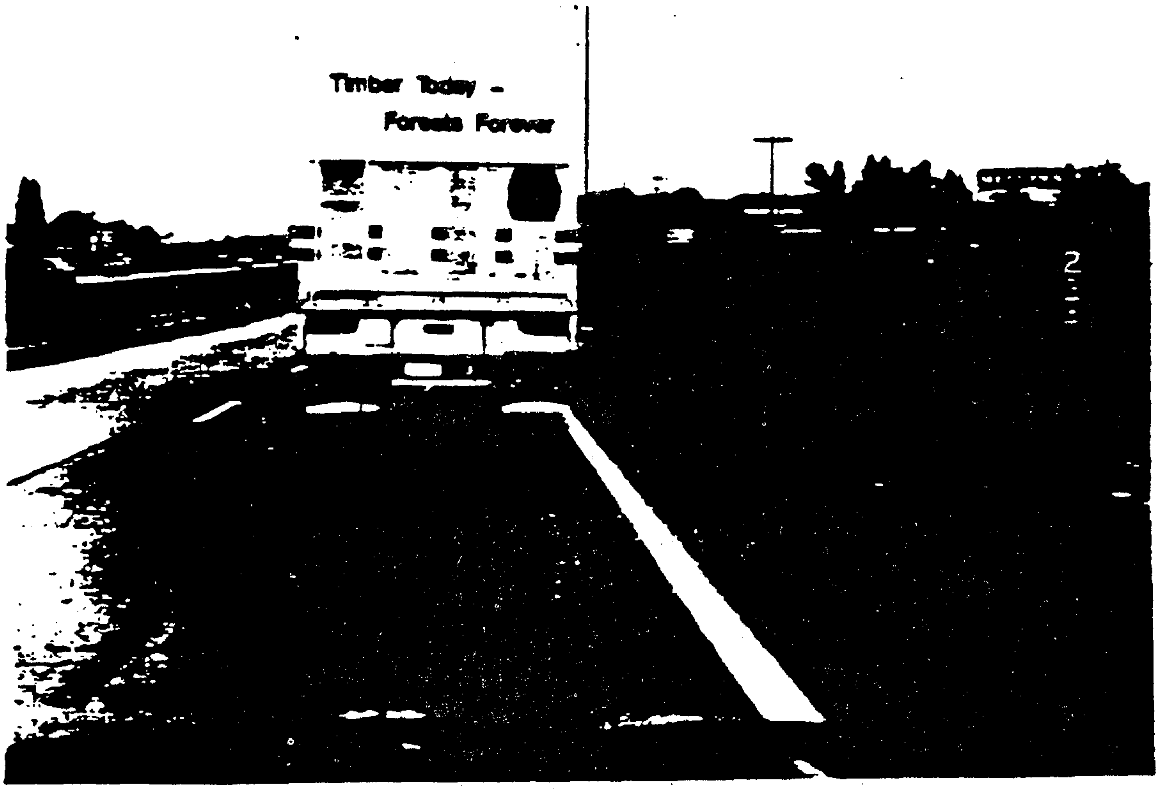
The issue of safety is also a subject that should be addressed when discussing the use of extended slabs with the traffic lane striped at 12 ft (3.7 m). Discussion with the Office of Highway Safety and Regional Safety Engineers indicates that there are no increased safety problems with the use of extended lanes when compared to standard 12 ft (3.7 m) lanes. In fact, widened lanes are equal or superior to the conventional lanes, from the standpoint of user safety. However, it should be noted that extended PCC slabs should not be considered a replacement for a shoulder structure.

In the area of user safety, lane edge stripe maintenance and location on extended pavement slabs has been a debated subject. Practices on standard 12 ft (3.7 m) pavements differ from State to State and even between different areas of a State. There are two basic theories about where to put the pavement lane edge stripe when shoulder structures differ from mainline pavement type. One practice places the edge line stripe on the PCC pavement (mainline) to keep the load off of the pavement/shoulder joint. The advantages of keeping the wheel loads 18 to 36 in (457 to 914 mm) away from the pavement have already been discussed and documented.

The second practice is to place the pavement edge line stripe beyond the pavement/shoulder joint and onto the shoulder (usually AC). One of the reasons for this practice is that the edge line will last a longer period of time, therefore reducing the associated maintenance costs (paint, trucks, crew). The other support for this practice lies in the color contrast between the white paint of the edge line and the



Photos 5 & 6 - Two examples of widened lane pavements performing as designed (inside lane 12 feet, outside lane 14 feet and striped at 12.5 feet).



black color of the asphalt shoulder. However, economic ramifications of accelerated pavement deterioration far exceed potential maintenance benefits of increasing the effective stripe life.

Most design engineers acknowledge the advantages of extended pavement slabs in keeping the traffic off of the shoulders. However, the control of the actual painting of the edge line stripes is in the hands of traffic and maintenance engineers. Until design concepts and concerns are thoroughly understood by this group, many extended slab designs will be wasted because of improper edge line placement. When comparing the costs of annual or bi-annual painting of edge lines and a 20-30 percent extended pavement life, the benefit-cost ratio supports correctly placed and maintained edge stripes. There is also little conclusive data that supports the concept of better edge line delineation on AC pavements than on PCC pavements.

CLOSING STATEMENTS

When gathering data for this paper, many research reports on differing subjects were reviewed that briefly mention extended pavement slabs and their benefits. Information regarding extended slabs and stresses, strains, deflections, and overall pavement deterioration is limited to portions of studies done on other subjects; there have been only a few studies performed recently that concentrate solely on lateral wheel load distribution and PCC pavement fatigue.

The greatest use of extended slabs is concentrated in the midwest (Iowa, Minnesota, South Dakota, and Wisconsin), with each State's design a little different. They are all building extended slabs for the same reason; to move the heavy truck wheel loads away from the edge of pavement. Extended pavement slabs have also been used in Georgia, Idaho, Louisiana, and Oregon. Delaware will be constructing its first extended pavement slab project during their 1989 construction season. As more pavements are built with extended slabs, much more information will be available on their performance.

There are points in the previous text that deserve repeating.

- Present wheel load distribution will be an average of 20 to 22 in (508 to 559 mm) away from the lane edge stripe on 12 ft (3.7 m) PCC slabs.

- The edge stripe, and not the overall width of the lane, controls lateral truck wheel distribution.

- Studies have shown that detrimental edge loads are reduced significantly at 16 to 20 in (406 to 508 mm) away from the PCC pavement edge.

- A PCC pavement that is widened 18 to 24 in (457 to 610 mm) and striped with a 12-ft (3.7 m) lane can expect a 20 to 30 percent increase in pavement fatigue life and reduced maintenance costs.

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U.S. Department of Transportation

Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: Longitudinal Cracking at Transverse Joints of New Jointed Portland Cement Concrete (PCC) Pavement with PCC Shoulders

Date: NOV 30 1988

From: Chief, Pavement Design and Rehabilitation Branch

Reply to Attn. of: HHO-12

To: Regional Federal Highway Administrators
Attn: Pavement Specialists

Attached is a report outlining a longitudinal cracking problem that occurred at the transverse joints near the edges of a mainline PCC pavement. This cracking is believed to be the result of the intrusion of mortar into the mainline transverse joints during shoulder construction.

The report indicates that the mainline PCC pavement was placed in warmer summer weather while the PCC shoulders were subsequently placed during cooler weather after the contraction joints had opened. When the shoulders were placed, mortar intruded into the mainline contraction joints and hardened. During the placement of the shoulders the contractor provided additional vibration along the lane/shoulder joint. This may have increased the flow of mortar into the open joints. With warmer weather, the slabs, unable to expand, developed longitudinal cracks near both edges of the mainline pavement.

It is recommended that, in similar situations, States consider sealing the sides of the contraction joints prior to placing adjacent pavement. This would prevent the intrusion of mortar into the joint.

L Kestner
for Paul Teng

Attachment

PROBLEM

On June 3, 1988 project personnel on 35-miles of a yet-to-be opened to traffic section of I-64 noted some longitudinal cracking on both sides of the transverse contraction joints in the portland cement concrete (pcc) pavement. At first, the longitudinal cracking appeared to be limited to a couple of interchange ramps, however, it was soon found to be scattered throughout the approximately eleven miles of pcc pavement having pcc shoulders. The other 24 miles of yet-to-be opened section of I-64 consisting of pcc pavement having asphalt shoulders was not experiencing any cracking. The cracks were typically 12-18 inches in length and were located approximately 9-12 inches from the mainline/shoulder joint. When cracking occurred, the cracks were always present on both sides of the transverse contraction joint and near the inside and outside of the mainline/shoulder longitudinal joints.

BACKGROUND

The eleven miles of pcc pavement were placed in 1987 under 3 paving projects involving two contractors. The mainline pcc consists of 12-inch jointed, mesh-reinforced concrete pavement having 40-foot joint spacing. The typical section consists of two travel lanes with an additional truck climbing lane for several sections. The tied shoulders consist of 10-inch non-reinforced concrete pavement having 20-foot joint spacing. The shoulders are tied to the mainline pavement with hook bolts. Free draining base material consisting of aggregate (No. 57 stone) treated with two percent of paving asphalt by weight of the aggregate is in place beneath the mainline and shoulder pcc pavement. Longitudinal and transverse joints were sawed and sealed using a low modulus silicone sealant. This section of I-64 was scheduled to be opened to traffic on July 15 so it was imperative that the cause of the problem be determined and an acceptable solution found before the road was opened.

On June 6, Messrs. W.T. Kelley, D.M. Hart and D.J. Voelker observed WVDOH personnel conducting a coring operation on the eastbound off-ramp at the Beaver Interchange. The core bit had caused a spall at the pavement/shoulder interface adjacent to the transverse contraction joint. It appeared that hardened mortar was in the transverse contraction joint below the backer rod. See attached drawing.

Although there were two different paving contractors, we believe they used similar methods and sequence of paving operations as outlined below. From discussions with project personnel, we believe the contractor slip-formed 24-foot wide mainline pcc pavement and 16-foot wide pcc interchange ramps. As soon as possible thereafter but no later than 24 hours after placing the new pcc, an initial saw cut 5/16-inch wide and 3-1/4 inch deep was made at the contraction joints. The contractor then placed a 3/8-inch backer rod in the saw cut to keep out incompressibles. This was later removed when the final saw cut was made. Final sawing of the transverse contraction joints in the mainline pavement was done before the pcc shoulders were placed. Final reservoir shape was 5/8-inch width by 1 1/2-inch depth. The contractor placed backer rod and silicone in the joint, however, the specifications did not require him to seal the edges of the transverse pcc joints at the mainline/shoulder joint. As shown in Section B-B of the attached drawing, the

.trong possibility existed there was an opening below the backer rod at the transverse contraction joint/shoulder edge. Also, in many cases, a vertical crack below the contraction joint had formed and was not sealed before the adjacent shoulder was placed.

Since the State earlier had some concerns over proper consolidation of the pcc at the mainline/shoulder longitudinal joint, the contractors placed vibrators near this edge and may have increased the number of rpms on the vibrators. This could have permitted intrusion of mortar into the opening below the backer rod and in the vertical crack as the shoulder paving operation progressed past the mainline transverse contraction joint. Inspections of some joints indicate this mortar had typically entered the opening below the backer rod about 4-5 inches but in some cases (high side of super-elevated sections) this intrusion was as much as 7-8 inches. The mainline pavement was placed during June-July 1987. Most of the interchange ramps were slip-formed in August-September 1987. The pcc shoulders were placed on the mainline pavement during September-October and on the ramp sections during October-November, 1987. The longitudinal joints between the mainline and shoulders were saw cut to a depth of 1 1/2 inches and 5/8-inch width.

Project personnel stated that in some places, the contractor's saw blades during the final saw cut were hardly touching the walls of the transverse contraction joints. In other words, it is possible the slabs had substantially contracted during cooler weather causing the joints to open.

 ll joint sealing operations were completed before the Winter of 1987-88. ıhere was no indication of any cracking in the pcc pavement during the Spring of 1988.

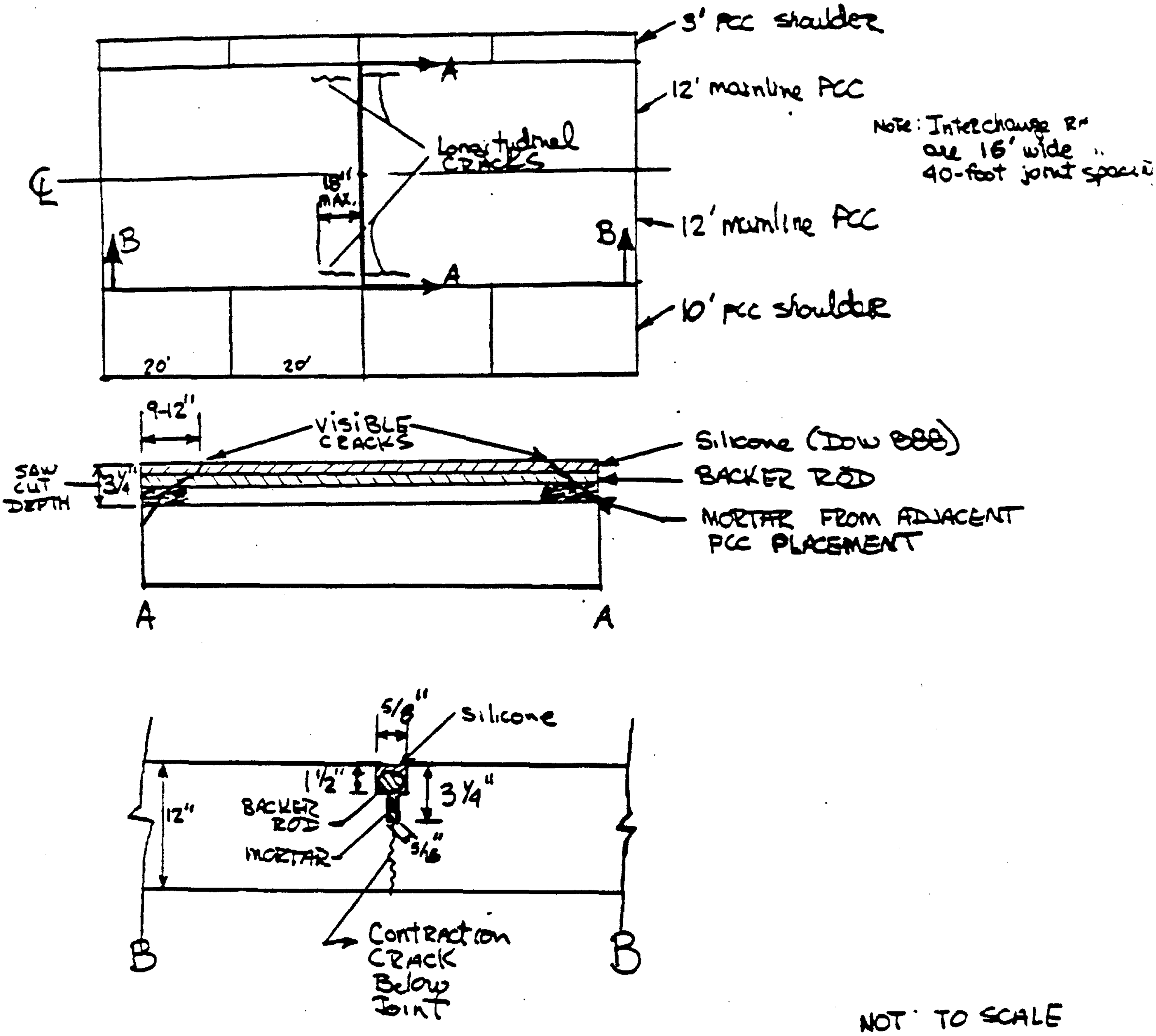
When the weather in May and June became very warm (highs in upper 80's and low 90's), cracks began to appear. It is believed the slabs were unable to properly expand due to the hardened mortar in the transverse contraction joints, causing the longitudinal cracking as shown in the attached drawing.

Solution:

It was agreed that all the transverse joints would be re-sawed at the mainline/truck lane and mainline/shoulder joints. It was felt whether a longitudinal crack was already present, it was imperative that each joint be sawed. A 5/8-inch wide by 6-8 inch deep saw cut was made. Care was taken to ensure the saw cut only extended about 6-7 inches into the mainline slab to prevent possible cutting of the dowel bars in the contraction joint. The new saw cuts were then cleaned using a wire brush mounted on a portable saw chassis and air blasted clean. If any incompressible appeared to remain in the joint, a second saw cut was made in an attempt to further breakdown the incompressible. The joints were then resealed with backer rod and low-modulus silicone.

Areas that already cracked were visually examined and sounded to determine if delaminations had occurred. Partial depth patches were made in delaminated areas. Cracks that had not caused delaminations were sealed with high .olecular weight methacrylate.

All operations were completed by July 1, allowing ribbon cutting ceremonies to proceed without delay on July 15. To prevent future problems, the State intends to modify their plans and specifications to require the contractor to prevent the intrusion of any incompressible materials into the transverse contraction joints when placing adjacent pavement. Also, consideration is being given to requiring final saw cutting within a reasonable time after initial saw cutting to prevent the potential of slabs significantly contracting or expanding due to weather.



I-64 Longitudinal Cracking

Technical Advisory

U.S. Department of Transportation
Federal Highway Administration

Subject

PORTLAND CEMENT CONCRETE MIX DESIGN AND FIELD CONTROL

Classification Code: T 5080.17

Date: July 14, 1994

1. PURPOSE. To set forth guidance and recommendations relating to portland cement concrete materials, covering the areas of material selection, mixture design, mixing, placement, and quality control.

2. BACKGROUND

a. Each year approximately 46 million cubic meters of concrete are used in all highway construction. The vast majority of States use a prescription type specification for portland cement concrete, often specifying minimum cement content, maximum water cement ratio, slump range, air content, and many times aggregate proportions. Admixtures such as fly ash are incorporated into mixes as a part of the prescription.

b. This system has worked fairly well in the past but may change as emphasis is placed on performance based specifications. States have begun to reduce or eliminate the amount of inspection at concrete plants as automation has increased productivity.

3. MATERIALS

a. Portland Cement. The proper type of portland cement should be specified for the conditions which exist.

OPI: HNG-23

(1) Types I, II, III, IP, and IS are typically used in highway construction. Type I is used when no special circumstances exist. Type II is used when sulfate exposure conditions are present. Type III is used when high early strengths are required. The use of Types IP and IS result in lower early strength gains and can be substituted for Type I cement when early strength is not a concern. In addition to the above mentioned types, Types IV and V are sometimes used in highway applications to meet special conditions. Further information about these cements can be found in the book Design and Control of Concrete Mixtures published by the Portland Cement Association (PCA).

(2) It is recommended that the acceptance of portland cement be based on certification by the supplier. The certification should contain the lot number of the cement. The supplier's test results should accompany the certification or be available to the State. Verification samples should be taken and used as part of the acceptance system.

(3) If alkali aggregate reactivity (AAR) is a concern, a maximum alkali content of 0.6 percent should be specified. Some State highway agencies consider this amount too high and recommend smaller amounts. If AAR is a problem in the State, a review of a States' Materials Manual is suggested. See Concrete Distress Conditions Section for other remedies.

b. Aggregates. Aggregates make up 60 to 70 percent of the volume of concrete mixes. A significant portion of poorly performing highway concrete can be traced to aggregate quality problems.

(1) The fine aggregate should meet the requirements of the American Association of State Highway and Transportation Officials (AASHTO) M 6.

(2) The range for the gradation of fine aggregate is quite broad. The fineness modulus (FM), calculated using AASHTO T 27, can be used as a tool for assessing the variability of the fine aggregate gradation. The specifications should limit the range of the FM between 2.3 and 3.1 according to AASHTO M6 and the variation of the FM should not be more than 0.20 from the value of the aggregate source.

(3) The FM is a means to control the influence that fine aggregate has on workability and the air content of the mix and is sometimes specified in the mix design. Further information regarding FM can be found in the Federal Highway Administration's manual FHWA-ED-89-006 (Portland Cement Concrete Materials Manual).

(4) It should also be noted that to provide good skid resistance, the PCA recommends that the siliceous particle content of the fine aggregate should be at least 25 percent. Consideration should be given, however, to the possibility of alkali-silica reactions when this is done.

(5) The coarse aggregate should meet the requirements stated in AASHTO M 80. For most parts of the country the severe exposure requirements should be used which means the use of class A aggregate for structural concrete and class B aggregate for pavements. The following table contains some of the more common information provided by Table 1 in AASHTO M 80.

	Class A Aggregate	Class B Aggregate
Clay lumps and friable particles	2%	3%
Chert	3%	3%
Sum of clay lumps, friable particles and chert	3%	5%
Material finer than No. 200	1%	1%
Coal and Lignite	0.5%	0.5%
Abrasion	50%	50%
Sodium Sulfate Soundness	12%	12%

c. Water

(1) The water serves as a key material in the hydration of the cement. In general, potable water is recommended although some non-potable water may also be acceptable for making concrete. Water of questionable quality should be examined since this can effect the strength and setting time. The following criteria is contained in Table 1 in AASHTO M 157 and is based on control tests made with distilled water:

Test	Limits
Compressive strength percent of control tests at 7 days	90
Time of set deviation from control	1 hour earlier to 1.5 hour later

(2) Wash water can be used to make concrete providing the resulting concrete mix water meets the following criteria in Table 2 in AASHTO M 157:

Chemical	Limits
Chloride as percent of weight of cement for the following uses:	
prestressed concrete	0.06
reinforced concrete in moist environment exposed to chlorides	0.10
reinforced concrete in moist environment not exposed to chlorides	0.15
sulfates	3000 ppm
alkalis	600 ppm
total solids	50,000 ppm

(3) If there is any question about the water, it should be tested using AASHTO T 26.

(4) It should be noted that the American Concrete Institute (ACI) provides more stringent tolerances for total chlorides in the mix. The chloride content for wash water in AASHTO M 157 is recommended for total chloride content in ACI 201.2R 22.

d. Admixtures. Admixtures are typically placed in mixes to improve the quality or performance. They can affect several properties and can have a adverse impact on the mix if not used properly. To avoid possible problems, it is suggested that trial batches be made to evaluate the mix.

(1) Air entraining admixtures should be specified when concrete will be exposed to freeze/thaw conditions, deicing salt applications, or sulfate attack. Recommendations for air content are contained in paragraph 4d.

(a) A vinsol resin type admixture should be added when fly ash having a variable loss on ignition (LOI) content (between 3 percent and 6 percent) is present. This is because of the effect that fly ash's fineness and carbon content has on the air entrainment system. Fly ashes not having a variable LOI do not have an adverse impact on entraining agents and therefore vinsol resin type admixtures may not be necessary.

(b) The specifications for air entraining admixtures are contained in AASHTO M 154.

(2) Chemical admixtures include water reducers, retarders, accelerators, high range water reducers (superplasticizers), corrosion inhibitors and combinations of the above. The specifications for chemical admixtures are contained in AASHTO M 194.

(a) Mixes containing admixtures are permitted an increase in shrinkage and a decrease in freeze thaw durability (as indicated in Table 1 AASHTO M 194) in comparison with mixes having no admixtures.

(b) Admixtures are usually accepted based on preapproval of the material and supplier certification. Verification tests should be performed on liquid admixtures to confirm that the material is the same as that which was approved. The identifying tests include chloride and solids content, pH, and infrared spectrometry.

(c) Water reducers and retarders may be used in bridge deck concrete to extend the time of set. This is especially important when the length of placement may result in flexural cracks created by dead load deflections during placement.

Often water reducers and retarders may increase the potential for shrinkage cracks and bleeding. Because of these concerns, increased attention needs to be placed on curing and protection.

(d) High range water reducers can be used to make high slump concretes at normal water cement (w/c) ratios or normal range slumps at low w/c ratios. The primary concern with the use of these admixtures is the loss of slump which occurs in 30 to 60 minutes. Redosing twice with additional admixture is allowed by ACI 212.4R; however, redosing typically reduces air entrainment. Type F and G high range water reducers may also be used. Type G has the added advantage of containing a retarding agent.

1 If transit mix trucks are used to mix high slump concrete, it is recommended that a 75mm slump concrete be used at a full mixing capacity to ensure uniform concrete properties. If transit mix trucks are used to mix low w/c ratio concrete, it is recommended that the load size be reduced to 1/2 to 2/3 the mixing capacity to ensure uniform concrete properties. Admixture companies are recommending additional mixing time with low w/c mixtures instead of decreasing the size of the load. This may have detrimental effects on some properties of the concrete such as the degradation of the aggregate resulting from over mixing.

2 High range water reducers may also affect the size and spacing of entrained air. If Freeze-Thaw

testing as described by ASTM C 666 indicates this to be a problem, it is recommended that the air content be increased by 1½ percent.

(e) Calcium chloride, the most commonly used accelerator, has been associated with corrosion of reinforcing steel and should not be used where reinforcing steel is present. In addition to the corrosion problem calcium chloride also reduces sulfate resistance, increases alkali-aggregate reaction, and increases shrinkage. Calcium chloride should not be used in hot weather conditions, prestressed concrete, or steam cured concrete. In applications using calcium chloride, the dosage rate should be limited to 2 percent by weight of cement.

(f) Non-Calcium Chloride accelerators are available and can be used where reinforcing steel is present. However, care must be taken in selecting these since some may be soluble salts which can also aggravate corrosion.

(g) Calcium Nitrate, which can be used as a corrosion inhibitor, also can function as an accelerator. There are no consensus standards available for the use of this material. Manufacturer specification sheets should be consulted for proper use.

(3) Mineral admixtures include fly ash, ground granulated blast furnace slag, natural pozzolans, lime, and microsilica (microsilica is also known as silica fume). Currently all of these materials are being used as additives or to reduce cement contents. Mineral admixtures are accepted based on approved sources with certifications and verification samples.

(a) According to the American Society of Testing and Materials (ASTM) C 618 and AASHTO M 295 there are two classes of fly ash, class C and class F. Since variability in fineness and carbon content can affect air content, the optional uniformity specifications in AASHTO M 295 should be specified when air entrained concrete is used. Fly ashes with LOI values less than 3 percent will typically not affect air content. Vinsol resin air entrainment admixtures should be specified when fly ash with LOI higher than 3 percent is used.

1 Fly ash may be used as a supplement or a replacement and is typically limited to 15 to 25 percent. If it is used as a replacement, it replaces cement on a 1.0 to 1.2:1 basis by weight.

2 Fly ash can be used to increase workability, reduce permeability, and mitigate alkali silica reaction (ASR); some Class C can make it worse. Class F fly ash with a calcium oxide content less than 10 percent can be used to mitigate ASR and sulfate attack. Fly ash with a calcium oxide content greater than 10 percent should be used in concrete which will be subjected to sulfate attack only with verification testing. This percentage and fly ash classification should only be used as a guide; further qualification should be based on ASTM C 452.

3 The cementing action with fly ash is pozzolanic in nature. The pozzolanic reaction with fly ash stops at approximately 4° Celsius.

Precautions need to be taken when using fly ash in concrete at lower temperatures. It should also be noted that fly ash can reduce early strength development and, therefore, should be monitored closely.

(b) Ground granulated blast furnace slag specifications are contained in AASHTO M 302.

1 Ground granulated blast furnace slag (GGBFS) is a cementitious material and can be substituted for cement on a 1:1 basis by weight for up to 50 percent of the cement in the mix.

2 For fresh concrete using GGBFS, the air entrainment agent dosage may need to be increased. The workability and finishability typically are improved but in mixes having high cementitious material content, mixes can be sticky and difficult to finish. Bleeding may be reduced and setting time may be longer.

3 Ground granulated blast furnace slag can reduce sulfate attack, alkali-aggregate reactions, and permeability. The rate of strength gain is usually decreased and sensitive to low temperature.

(c) Microsilica specifications are contained in AASHTO M 307. Microsilica can be used as an admixture or as a replacement for an equivalent amount of cement to produce high strength concrete. Microsilica will reduce permeability and help reduce alkali-aggregate reactions.

1 Microsilica has been used as an addition to concrete up to 15 percent by weight of cement, although the normal proportion is 10 percent. With an addition of 15 percent, the potential exists for very strong, brittle concrete. It increases the water demand in a concrete mix; however, dosage rates of less than 5 percent will not typically require a water reducer. High replacement rates will require the use of a high range water reducer.

2 Microsilica greatly increases the cohesion of a mix, virtually eliminating the potential for segregation. However, the cohesion may cause mixes to be sticky and difficult to finish. It may be necessary to specify a higher slump than normal to offset the increased cohesion and maintain workability. In addition, microsilica in the mix greatly reduces bleeding; therefore, mixes which contain microsilica tend to have a greater potential for plastic shrinkage cracking. It is imperative to use the proper curing methods to prevent the surface water from evaporating too quickly.

4. PROPORTIONING. Most of the concrete placed in highway facilities in the United States are under severe exposure conditions. State highway agencies specify a recipe for concrete mixes which includes minimum cement content, maximum water-cement ratio, air content range, and minimum strength. These requirements are necessary to achieve durability, as well as strength.

a. The maximum aggregate size should be as large as possible. This reduces total aggregate surface area and results in lower cement demand. The maximum aggregate size should be limited to 20 percent of the narrowest dimension of a concrete member, 75 percent of the clear spacing between reinforcing steel, or 33 percent of the depth of a slab for unreinforced concrete.

b. The minimum cement content refers to all cementitious and pozzolanic material in the concrete, including cement and any mineral admixtures that are being added to or substituted for cement. Replacement rates should be based on those contained in paragraph 3d(3).

(1) The PCA recommends a minimum cement content of 335 kg/m^3 for concrete placed in severe exposure conditions and ACI 316R recommends a minimum cement content of 335 kg/m^3 for concrete pavements in all locations unless local experience indicates satisfactory performance with lower cement contents. Even if strength requirements can be met with a lower cement content, a minimum cement content of 335 kg/m^3 should be used unless it can be demonstrated that the concrete will be durable.

(2) In cases where local experience allows a reduction in cement content below 335 kg/m^3 the cement content should not be reduced below the following minimum cement contents recommended by ACI 302.1R Table 5.2.4 for concrete slab and floor construction. The minimum cement contents listed below are based on the nominal maximum size of the aggregate. The cement content decreases as the nominal maximum aggregate size increases due to the decrease in aggregate surface area.

Nominal maximum size aggregate, mm	Cement content kg/m^3
37.5mm	280kg/m^3
25mm	310kg/m^3
19mm	320kg/m^3
12.5mm	350kg/m^3
9.5mm	365kg/m^3

(3) Low strength concrete in the field should not be addressed by arbitrarily increasing the cement content since an increase in cement content will increase the water demand leading to higher shrinkage and permeability. All changes in mix proportions should be evaluated with a trial batch.

c. The water-cement ratio in all cases should be as low as possible while maintaining workability. For freeze thaw resistance the following maximum water cement ratios are recommended in ACI 201.2R.

Thin sections (bridge decks, pavements and curbs) and sections with less than 25 mm cover and concrete exposed to deicing salts	0.45
all other structures	0.50

The water-cement ratio should include the weight of all cement, pozzolan, and other cementitious material.

d. The air content in the mortar fraction of the mix should contain approximately 9 percent air for concrete mixes exposed to severe conditions.

(1) The following recommendations are from ACI 201.2R Table 1.4.3.

Nominal maximum size aggregate, mm	Air content Percent
37.5mm	5-1/2
25mm	6
19mm	6
12.5mm	7
9.5mm	7-1/2

(2) The specified tolerance for air content should be ± 1½ percent.

5. <u>PROPERTIES OF CONCRETE</u>. Trial batches should be performed on all mixes at the expected placement temperatures. This is especially true for mixes containing multiple admixtures.

a. Workability. A concrete mix must be workable to ensure proper consolidation and finishing. The workability of a mix is a function of the gradation of the aggregate, amount and type of admixtures, water content, concrete temperature, and time. Once a workable mix is established during the trial batch process, slump can be used to monitor the consistency and uniformity of the mix. Slump, by itself, is not a measure of workability.

b. Durability

(1) Freeze-thaw durability depends on durable aggregates, proper air entrainment, low permeability, and a low water-cement ratio.

(2) D-cracking is strictly a pavement durability problem and is associated with aggregates. It should be addressed with the source approval of the aggregates.

(3) Alkali aggregate reactions are mostly the result of the alkali content of the cement in the concrete. The most common alkali aggregate reaction is associated with silicious aggregates although reactions have occurred with carbonate materials. If a reactive aggregate is encountered, several options are available: not using the source of aggregate, using a low alkali cement, using fly ash, or using microsilica. If alkali reactive aggregates are used, testing should be performed with the mix prior to its use to ensure a durable concrete.

(4) Resistance to or susceptibility to sulfate attack depends on the chemical composition of the cementitious portion of the concrete. Sulfate attack can occur from ground water, deicing salts, or sea water. Type II or Type V cement or some fly ashes, may be used to mitigate the problem.

c. Strength. The strength requirement is the compressive strength, f'_c, at 28 days. This must be equal to or exceed the average of any set of three consecutive strength tests. No individual test (average of two cylinders) can be more than 3.5 MPa below the strength requirements in the specification.

6. MIXING, AGITATION, AND TRANSPORTATION

a. In order to ensure proper operation, a concrete plant must be calibrated and inspected. Plant approval should include all the items covered in the Checklist for Portland Cement Concrete Plant Inspection (Attachment 1). This same checklist also discusses the inspection of truck mixers. The plant certification program operated by the National Ready Mix Concrete Association covers the same information contained in the attachment.

b. The mixing time for central mixers and approval of truck mixers should be determined by the uniformity test discussed in AASHTO M 157, Ready Mixed Concrete. The test is based on the comparison of tests on samples taken at the first and last 15 percent of the load. The following are maximum permissible differences to consider the mix properly mixed.

Test	Maximum Difference
Unit weight (air free basis)	15 kg/m^3,
Air content	1 percent
Slump	
less than 100mm	25mm
100 to 150mm	37.5mm
Coarse aggregate content	6.0 percent
Unit weight of air free mortar	1.6 percent
Compressive strength (7 day)	7.5 percent

c. Water added at the job site must be measured accurately. A water meter is the most accurate method for determining the amount of water added to the mix.

d. The recommendations for testing appear in paragraph 11, Quality Control and Testing, of this document.

e. The haul time should be limited to 90 minutes for truck mixers that agitate the mix and 30 minutes for trucks that do not agitate the mix. The maximum number of revolutions for truck mixers should be limited to 300.

f. No admixtures or water should be permitted to be added to the mix after the mixer has started unloading.

7. PLACEMENT AND CONSOLIDATION

a. Prior to placement of the concrete an inspection should occur covering the items in either the checklist for the placement of structural concrete (Attachment 2) or the checklist for the placement of concrete paving (Attachment 3).

b. Acceptance testing for pumped concrete should occur at the discharge end of the pump.

c. Aluminum pipe and chutes should not be used in concrete pumping operations.

d. Concrete can be conveyed to the location of placement by several commonly used methods including pumps, belt conveyors, buckets, chutes, and dropchutes. Care should be taken to ensure that there is no debris or blockages that will hinder or influence the properties or flow of the material. Concrete should not be allowed to free fall from distances greater than 1.2 meters to avoid segregation.

e. All concrete should be accompanied to the project with a delivery ticket. A sample delivery ticket appears as Attachment 4.

f. The proper consolidation of concrete is a significant factor in the ultimate performance of the concrete and it is achieved through vibration.

(1) The following are recommended frequencies for vibrators from ACI 309.

Diameter of head, mm	Frequency vibrations per minute
20 to 40 mm	10,000 - 15,000
30 to 65 mm	9,000 - 13,500
50 to 90 mm	8,000 - 12,000

8. CURING AND PROTECTION

a. Curing

(1) Curing is performed to maintain the presence of water in concrete and to provide a favorable temperature for cement hydration. Methods of curing include ponding, spraying, and fogging with water, wet covers such as burlap, plastic sheets, membranes, and the use of steam, electric forms, or insulation.

(2) The application rate of a particular curing compound should be based on the rate established during the approval process of the curing compound. The AASHTO M 148 indicates that a rate of application of $5m^2$/liter should be used for testing the material if no other rate is specified.

b. Protection

(1) Cold weather protection should be required when it is expected that the daily mean temperature for three consecutive days will fall below 4° Celsius. The following recommendations are for the minimum temperatures for delivered concrete as they appear in AASHTO M 157.

Air	Minimum Concrete Temperature	
Temperature	Thin	Thick
-1 to 7°C	16°C	10°C
-18° to -1°C	18°C	13°C
Below -18°C	21°C	16°C

Thin sections are defined as those less than 300 mm.

(2) Concrete should never be placed on a frozen subgrade. Care should be taken to assure that the subgrade is free from frost.

(3) Hot weather conditions can be defined as a condition of high temperature, low humidity, and high winds. The existence of these conditions can be determined by finding the evaporation rate described in ACI 305 and included in Attachment 5. An evaporation rate exceeding 1 $kg/m^2/hr$ has the potential of causing plastic shrinkage cracks. The evaporation rate is a function of concrete temperature, ambient temperature, relative humidity, and wind velocity. This chart has been incorporated into several State specifications. It may not completely apply in all cases, especially in mixes containing admixtures which reduce the amount of bleeding.

(4) In addition to the plastic shrinkage cracking problem, ultimate strength will decrease with higher temperatures. The ACI has not recommended a maximum concrete temperature since strength loss can be compensated for by other means.

However, significant strength loss occurs above 32°C. Due to the strength loss and increase in potential for plastic shrinkage cracking, many States have set a maximum ambient placement temperature of 32°C. In all cases, trial batches should be performed at the highest expected temperature to ensure that the concrete will have the desired properties.

9. <u>CONCRETE DISTRESS CONDITIONS</u>

a. Alkali aggregate reactivity can be one of two types, alkali-silica and alkali-carbonate. The most prominent problem is cracking of the concrete due to the alkali-silica reaction (ASR).

(1) A widely used test to determine ASR is ASTM C 227. The current test criteria allow a maximum expansion of 0.05 percent at 3 months and 0.1 percent at 6 months. Research by PCA indicates that the critical criteria is 0.1 percent ultimate expansion. Since some reactions take longer than others, testing should continue as long as expansion is occurring. Some aggregates may take several years to show expansion.

(a) Recently the Strategic Highway Research Program developed a test which can be used for rapid determination of ASR. It is called the Gel Fluorescence Test and can be performed easily and inexpensively by field personnel. With this test, a 5 percent solution of uranyl acetate is applied on the concrete surface. Ultraviolet light is then used to illuminate the surface and if ASR exists, a yellow-green fluorescent glow will appear. Some safety concerns may be associated with this test so proper precautions are recommended. It should also be noted that the test is limited to preexisting concrete and not to fresh concrete.

(b) Alkali-silica reaction can be mitigated by limiting the alkali content of portland cement to 0.6 percent, by using class F fly ash or microsilica admixtures, or by reducing the water to cement ratio. The success of this approach may be limited; therefore, laboratory testing should be conducted. Protecting the final structure from moisture also reduces ASR.

(c) Although PCA recommends 25 percent of the fine aggregate be siliceous material to improve skid resistance, the use of some siliceous material can promote the ASR reaction and requires care to ensure this will not occur.

(2) Alkali-carbonate reaction (ACR) may occur with dolomitic limestones which contain large amounts of calcite, clay, or silts. ASTM C 586 is used to screen dolomitic materials for alkali-carbonate reactions.

b. D-cracking occurs when freeze-thaw conditions combine with saturated concrete made from susceptible coarse aggregates. The problem is only associated with pavements. Some dolomites and limestones are susceptible due to their pore structure.

(1) The most common test for predicting D-cracking susceptible aggregates is AASHTO T 161. There are two methods contained in the procedure. In method A the specimens are immersed in water for freezing and thawing. In method B the specimens are frozen in air and thawed in water. The number of freeze thaw cycles varies between 300 to 350. The minimum durability factor specified by the States range between 80 and 95. Some States have also specified a maximum expansion criteria range between 0.025 percent and 0.06 percent. It should be noted that the test method allows a significant range of time for freezing and thawing cycles. This can account for the variation in the criteria used by the States. Care needs to be taken when establishing criteria so that it will correspond to the test equipment and the history of performance of the aggregates.

(2) The hydraulic fracture test developed under SHRP may be able to provide a determination of the D-cracking susceptibility of aggregates in only about 1 week compared with the 8 weeks for T 161. In this test, dry aggregates are submerged in a pressure chamber and the pressure is increased to force water into the pores. After releasing the pressure, D-cracking susceptible aggregate will fracture as the water is forced out of the pores.

10. <u>MANUFACTURED CONCRETE PRODUCTS</u> Concrete products consist of structural elements constructed at a plant and trucked to the jobsite. These precast products typically consist of beams, pipes, barriers, poles and other special elements. The criteria outlined within this document apply to these products as well. Additional information about prestressed products are contained in the Checklist for Prestressed Concrete Products in Attachment 6.

11. <u>QUALITY CONTROL AND TESTING</u>

a. All testing should be performed by certified technicians. The ACI and the National Institute for Certification in Engineering Technologies (NICET) administer a concrete technician certification program. Guidance for establishing a certification program for testing personnel appears in a FHWA paper titled "Laboratory Accreditation and Certification of Testing Personnel."

b. Process control testing should be performed on aggregate moisture content, aggregate gradation, air content, unit weight, and slump at the plant.

(1) The specifications should require that the contractor provide a process control plan. The State should also provide guidance on the minimum requirements for a process control plan. As a minimum, the process control plan should include the information contained in Attachment 7.

(2) All process control tests should be plotted on control charts. Control charts are a good visual tool for discovering trends quickly before major problems occur.

c. The acceptance procedures should include monitoring of the process control activities including aggregate gradation testing. In addition, acceptance testing at placement would include slump, strength, and air content. Close monitoring of the water-cement ratio is also required since this will ultimately affect the durability and strength of the concrete.

Additional information on acceptance procedures is provided in the Technical Advisory on Acceptance of Materials T 5080.11.

d. It is recommended that compressive strength be accepted using statistical criteria (based on average strength and standard deviation) to ensure that the strength, f'_c, at 28 days, is equal or exceeded by the average of any set of three consecutive strength tests. No individual test (average of two cylinders) can be more than 3.5 MPa below the specified strength. There are two strengths to be considered. One is the minimum specified strength (f'_c) which is a function of the structural requirements. The second is the average strength for mix design (f'_{cr}). The f'_{cr} must be higher than f'_c to ensure that the concrete will exceed the minimum specified strength. The following recommendations for f'_{cr} are from ACI 318.

(1) Unknown Standard Deviation

Specified compressive strength, MPa	Required average compressive. strength, MPa
f'_c	f'_{cr}
Less than 20MPa	$f'_c + 6.9$
20MPa to 35MPa	$f'_c + 8.3$
Over 35MPa	$f'_c + 9.6$

(2) Known Standard Deviation

For greater than 30 test results (one test result is the average of two cylinder breaks) f'_{cr} is the greater of the two values from the following equations.

MPa

$$f'_{cr} = f'_c + 1.4s$$
$$f'_{cr} = f'_c + 2.4s - 3.5$$

s = Standard deviation

(3) For 15 to 30 test results the standard deviation in the above formulas can be modified by the following factors.

No. of Tests	Modification factor for standard deviation
Less than 15	use table for unknown s
15	1.16
20	1.08
25	1.03
30	1.00

e. Air content and slump should be accepted based on an attribute system, i.e., pass/fail. The following is a recommended criteria.

Acceptance criteria	Air content deviation, %	Slump deviation, mm
Acceptable	< 1.5	< 25mm
Acceptable for trucks on the road	1.5 to 2	25 to 31.5mm
Reject	> 2	≥ 31.5mm

f. **Testing procedures for resistance to freeze-thaw damage, deicing salt attack, and abrasion resistance are long and involved and do not lend themselves to testing on a routine basis. These tests are usually conducted to determine the durability of the concrete. It should also be noted that high strength concrete does not always insure durable concrete.**

ARKane

Anthony R. Kane
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Attachments

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CHECKLIST FOR PORTLAND CEMENT CONCRETE PLANT INSPECTION

1. Materials

   A. Cements and Mineral Admixtures (cement, fly ash, etc.)

      (1) Is evidence of cement or fly ash acceptability present (certification, test results)?

      (2) Are bins or silos tight and provide for free movement to discharge opening?

      (3) Are bins or silos periodically emptied to check for caking?

      (4) Plants should provide separate storage for each type of cement or mineral admixture being used. Are the materials being isolated to prevent intermingling or contamination?

   B. Aggregates

      (1) Does the plant display evidence of source approval?

      (2) Are aggregates stockpiled to prevent segregation and degradation? The preferred method of stockpiling is in layers. Cone shaped stockpiles will segregate.

      (3) Are stockpiles adequately separated to prevent intermingling?

      (4) Does the plant maintain separate storage bins or compartments for each size or type of aggregate? Are the aggregates tested for gradation and moisture content?

      (5) What is the surface underneath stockpiles? Soil or paved? Are the stockpiles covered?

C. Water

(1) Does the plant have an adequate water supply with pressure sufficient to prevent interference with accuracy of measurement?

(2) Is there any evidence or history of contaminants in supply?

D. Liquid Admixtures

(1) Is there evidence of source approval?

(2) Is the admixture and dispensing equipment protected from freezing, contamination, or dilution?

(3) How often are the admixture metering and dispensing equipment periodically cleaned?

2. <u>Batching Equipment</u>

A. Scales

(1) Scales should indicate weight by means of a beam with balance indicator, full range dial, or digital display.

(2) For all types of batching systems the weighing devices must be readable by the batchman and the inspector from their normal stations.

(3) Scales should be certified or should be calibrated with a certified scale.

(4) Ten 25 kilogram test weights should be available at the plant at all times.

(5) Scale accuracy should generally be within plus or minus .4 percent of the scale capacity.

(6) Water meters will need to be calibrated to 1 percent of total added amount.

B. Batchers

(1) Cementitious material should be weighed on a scale that is separate and distinct from other materials.

(2) Bins with adequate separation should be provided for fine aggregate and each size coarse aggregate.

(3) Weigh hoppers should not allow the accumulation of tare materials and should fully discharge into the mixer.

(4) Batchers should be capable of completely stopping the flow of material and water batchers should be capable of leak free cut off.

(5) Separate dispensers will be provided for each admixture.

(6) Each volumetric admixture dispenser should be an accurately calibrated container that is visible to the batchman from his normal position.

(7) Aggregate should be measured to plus or minus 2 percent of the desired weight, cement to 1 percent, water to 1 percent and admixtures to 3 percent.

(8) Semi-automatic and automatic control mechanisms should be appropriately interlocked.

3. <u>Mixing</u>

A. Stationary Mixers

(1) Mixers should be equipped with a metal plate that indicates mixing speed and capacity.

(2) Mixers should be equipped with an acceptable timing device that will not permit discharge until the specified mixing time has elapsed.

(3) Mixers are to be examined periodically to detect changes in condition due to accumulation of hardened concrete or blade wear. A copy of the manufacturer's design, showing dimensions and arrangements of blades, should be available at the plant at all times.

B. Truck Mixers

(1) Mixers should be equipped with a metal plate that indicates mixing speed, capacity, mixing revolutions, agitating speed and agitating capacity.

(2) Mixers should be equipped with a revolution counter.

(3) Mixers are to be examined to determine satisfactory interior condition, that is, no appreciable accumulation of hardened concrete and no excessive blade wear. A copy of the manufacturer's design, showing dimensions and arrangements of blades, should be available at the plant at all times.

(4) Charging and discharge openings and chutes should be in good condition.

4. <u>Weather</u>

A. Hot Weather

(1) When concreting during hot weather, is plant equipped to cool ingredients? Is equipment available to produce acceptable ice?

(2) How are aggregates cooled? If by sprinkling, is provision made to account for excessive water?

B. Cold Weather

(1) When concreting during cold weather, is plant equipped to heat ingredients to produce concrete of applicable minimum temperature.

CHECKLIST FOR STRUCTURAL CONCRETE

1. TREATMENT OF FOUNDATION MATERIAL

   Has special care been taken not to disturb the bottom of any foundation excavation?

2. CURING

   Is the concrete being cured for 7 days, by one of the following methods?

   (a) Waterproof paper method

   (b) Polyethylene sheeting method

   (c) Wetted burlap method

   (d) Membrane curing method

3. REINFORCEMENT BAR STORAGE

   Are all delivered rebars being stored above the ground upon skids, platform, or other supports? A light coating of rust will not be considered objectionable.

   Are epoxy coated bars being stored on padded supports and handled to prevent damage to the bar coating?

4. FORMS

   Are the forms clean, braced, tight, and sufficiently rigid to prevent distortion?

   When wooden forms are used, are they dressed lumber or plywood and oiled prior to rebar placement?

   Are all sharp corners in forms being filleted with 20 millimeters molding, unless otherwise specified?

5. REINFORCEMENT BAR PLACEMENT

   Are all reinforcement bars tied securely in place? Are epoxy coated bars being tied with plastic or epoxy coated tie wire?

When epoxy coated bars are cut in the field, are they being sawed, sheared, or cut with a torch? Cutting with a torch is not acceptable. If cut in the field, the bars should be repainted at the cut ends with a similar type of epoxy paint.

Are at least 50 percent of the bar intersections being tied?

Are all rebar laps of the specified length?

Are all portions of metal bar supports in contact with any concrete surface galvanized or plastic coated? Are epoxy coated bars being supported with plastic, plastic coated, or epoxy wire chairs?

Are the reinforcement bar support in sufficient quantity and adequately spaced to rigidly support the reinforcement bars?

After epoxy coated bars are in place, are the bars inspected for damage to the coating and is the contractor repairing all scars and minor defects using the specified repair materials?

Is the finishing machine being used to detect high bars by making a "dry run" over the length of the deck prior to concrete placement? Is the proper coverage being maintained between the bars and any form work or surface, top, side, and bottom?

6. PRE-POUR INSPECTION

Prior to the placement of the concrete have the reinforcement bars, construction joints, and forms been cleaned of mortar, dirt, and debris?

Are the strike-off screeds set to crown, and other equipment on the job-site (such as vibrators) in good working condition?

7. USE OF RETARDING ADMIXTURE (BRIDGE DECK)

If the specified temperature is reached, is a retarding admixture being used in the bridge deck concrete?

8. TEMPERATURE CONTROL

Are proper precautions being taken for hot and cold weather concrete?

If outside temperatures warrant it, are temperature checks of the plastic concrete being taken?

9. TIME OF HAUL

Is all concrete that is being hauled in truck mixers being deposited within 90 minutes from the time stamped on the tickets?

If central-mixed concrete is hauled in nonagitor trucks, is the concrete being deposited within 30 minutes?

10. REVOLUTIONS

Have 70 to 100 mixing revolutions at mixing speed been put on the truck at the required speed (6-18 RPM)?

Have 30 mixing revolutions been placed on the truck at the required speed (6-18 RPM) after water has been added at the site?

Is the agitating speed between 2-6 RPM?

Are total number of revolutions being limited to 300?

11. CONCRETE DELIVERY TICKET

Are all truck tickets being properly completed, collected, and retained?

12. WATER CONTROL

Is all water that is being added to the mix accounted for and checked to ensure the w/c ratio is not exceeded?

13. AIR CONTENT DETERMINATION

Are air content tests being performed according to the required frequency?

14. <u>SLUMP TEST</u>

Are slump tests bring performed according to the required frequency?

15. <u>STRENGTH TEST</u>

Are concrete test specimens being cast at the site of work as per the required frequency?

16. <u>PLACING CONCRETE</u>

Is the concrete being deposited as near its final position as possible? (Moving concrete horizontally with vibrators is not permitted.)

Is the concrete being bucketed, belt conveyed, pumped, or otherwise placed in such a manner as to avoid segregation and is not being allowed to drop more than 1.2 meters?

17. <u>CONSOLIDATION</u>

Is all the concrete being consolidated with hand operated spud vibrators while it is being placed?

18. <u>FINISHING (DECKS)</u>

Is a finishing machine (having at least one reciprocating, nonvibratory screed operating on rails or other supports) being used to strike off and screed the bridge deck?

19. <u>STRAIGHTEDGE TESTING AND SURFACE CORRECTION (DECK)</u>

Is the plastic concrete being tested for trueness with a 3 meter straightedge held in contact with the slab in successive positions parallel to the centerline?

Are all depressions being immediately filled and all high areas being cut down and refinished?

20. <u>SURFACE TEXTURING</u>

Is the deck surface being textured with either a burlap drag or an artificial turf drag followed by tining with a flexible metal comb?

CHECKLIST FOR PORTLAND CEMENT CONCRETE PAVING

1. SUBBASE TRIMMING

   Has the subbase been trimmed prior to paving?

2. PAVING FORMS (IF USED)

   Are the forms: metal, not less than 3 meters in length, equipped with both pin locks and joint locks, within 2 millimeters along the length of its upper edge, within 7.5 millimeters along the length of its front face, and in sufficient supply.

   Is the height of form face at least the edge thickness of proposed pavement, the base width equal to or greater than the height, and are three steel pins being used to secure each section?

   Are the forms being set on a hard and true grade, built up in 12.5 millimeters maximum lifts of granular material in low areas (without using wooden shims) and oiled prior to the placing of concrete?

   When wooden forms are allowed, are they full depth, smooth, free of warp, not less than 50 millimeters thick when used on tangent, and securely fastened to line and grade?

   Are curved form of metal or wood being used on curves of 30 meters radius or less?

3. FORM ALIGNMENT

   Is the contractor checking the forms for line and grade and making necessary adjustments prior to concrete placement?

4. TEMPLATE

   Is the surface of the subbase being tested for crown and elevation by means of a template?

5. SUBBASE THICKNESS TEST

After trimming, is the thickness of the subbase being checked?

6. DRAINAGE

Is the subgrade being kept drained during all operations? Are all berms of earth deposited adjacent to the grade being kept drained by cutting lateral ditches through the berms?

7. LUG SYSTEMS (CONTINUOUSLY REINFORCED)

If concrete lug end anchorages are specified, are they staked and checked for dimensions and re-bar placement as shown in the plans?

Are they constructed of Structural Concrete at least 24 hours prior to pavement construction?

8. LONGITUDINAL JOINT KEYWAY AND BARS

Are the beginning and ending stations marked where adjacent curb, median, or pavement will necessitate the placement of keyway and/or bars in the edge of the proposed pavement?

9. SUPERELEVATION STAKING

Are the plan curb data examined for all curves to determine where to stake the beginning and ending stations for all superelevation transitions?

10. TEMPERATURE LIMITATIONS

Does the outside air temperature in the shade meet State specifications?

Does the temperature of the concrete meet State specifications at the time of placement?

11. REINFORCEMENT LAPPING

Are the locations and lengths of lap for bar or fabric reinforcement in conformance with the specifications.

Are all bar and fabric laps being tied?

12. TRUCK REQUIREMENTS

Is all concrete in a stationary mixer being deposited within 30 minutes when hauled in non-agitating trucks and within 90 minutes when hauled in agitator trucks?

Is transit mixed concrete being delivered and deposited within 90 minutes from the time stamped on the ticket?

If the contractor plans to use previously placed pavement as a haul road, are the truck weights checked to assure compliance with maximum weights permitted by State Law?

13. REINFORCEMENT PLACEMENT

Is the reinforcement being placed in accordance with one of the following methods?

Method A - After the full depth concrete is struck off the reinforcement should be placed into the concrete to the required depth by mechanical means.

Method B - The reinforcement should be supported on the prepared subbase by approved chairs having sand plates.

Method C - When the concrete is being placed in two layers the reinforcement should be laid full length on the struck-off bottom layer of concrete in its final position without further manipulation. (Cover within 30 minutes.) The depth of the first lift is 2/3 the depth of the pavement.

Method D - The reinforcement may be placed in the pavement using a method which does not require transverse steel or support chairs for support of the longitudinal steel. Tie bars at longitudinal joints are still required.

14. SEQUENCES OF FORM TYPE PAVING

Is all of the required concrete finishing equipment on the job and in acceptable working condition? Are the following sequences for form type paving being properly followed:

(a) Placing concrete. As little rehandling as possible. If equipment used can cause segregation, is the concrete being unloaded into an approved spreading device?

(b) Strike-off. Is the concrete being struck full width to the approximate cross section of the pavement?

(c) Consolidation. Is one pass of an approved surface vibrator or internal vibrator being made?

(d) Screeding. Are at least two passes with a machine having two oscillating screeds, and a finisher float being made?

(e) Straightedging - Are at least two 3 meter long shoulder operated or surface operated surface trueness testers (straightedges) being used?

(f) Surfacing Texturing - Are State specifications for texturing and tining being followed?

15. <u>SEQUENCES OF SLIPFORM PAVING</u>

When the contractor uses this optional method for the construction of the pavement are the following sequences being properly followed:

(a) Is the formless paver capable of spreading, consolidating internally, screeding and float finishing the newly placed concrete in one pass to the required line and grade?

(b) Is the pavement being straightedged, edged, and textured as required in the previous question 14?

(c) Does the contractor have available at all times metal or wooden sideforms and burlap or curing paper for the protection of the pavement in case of rain?

(d) Is the contractor immediately repairing all slumping edges in excess of 12.5 millimeters?

16. THICKNESS TEST

Is the thickness of the pavement being checked?

17. AIR CONTENT

Is the air content being tested as required by the frequency chart?

18. SLUMP

Is the slump being checked as required by the frequency chart?

19. REINFORCEMENT, DOWEL, AND TIE BAR DEPTH CHECKS

Is the concrete being probed to check the vertical and horizontal positioning of the pavement reinforcement, dowels, and tie bars?

20. STRENGTH

Are test specimens being cast at the site of work at the required frequency:

(a) at least one set per day
(b) one set for every 150 meters of two lane pavement (300 meters of one lane pavement)

21. LONGITUDINAL JOINT

(a) **Are tie bars placed properly?**

(b) **Are the joints sawed at the same time as the transverse joints with pavement widths greater than 7.3 meters? Are they cleaned and immediately filled with sealer?**

22. TRANSVERSE JOINTS

(a) **Are the smooth dowel bars positioned parallel to the grade at a depth of ½ t.**

Are the dowel bars coated with a thin bond breaker?

Are the capped ends of the bar coated with a debonding agent? (Expansion joints)

(b) Is a 1/3T deep groove being sawed over each assembly as soon as possible after concrete placement? Cleaned immediately?

(c) Are all joints being sealed after the curing period and before opening to traffic?

23. TRANSVERSE CONSTRUCTION JOINTS (CONTINUOUSLY REINFORCED CONCRETE)

(a) Are construction joints being placed at the end of each day's operation or after an interruption in the concreting operation of 30 minutes or more?

(b) Are construction joints being placed at least 1 meter from nearest bar lap?

(c) Are construction joints strengthened by supplementary 1.8 meter long bars of the same nominal diameter as the longitudinal steel so that the area of steel through the joint is increased by at least 1/3?

(d) Are construction joints formed by means of a clean (not oiled) split header board conforming to the cross section of the pavement?

(e) Is the concrete at construction joints being given supplemental internal vibration along the length of the joint both at the end of the day's operation and once again at the resumption on the next day? This is critical.

24. TRANSVERSE CONSTRUCTION JOINTS (JOINTED PAVEMENT)

(a) Are construction joints being placed at the end of each day's operation or after an interruption in the concreting operation of 30 minutes or more?

(b) Are construction joints being placed at least 3 meters from any transverse joint?

(c) Are construction joints being strengthened by epoxy coated dowel bars of the same size and positioning as specified for contraction joints?

Is a thin coating of bonding breaking agent applied to the dowels?

(d) Are construction joints being formed by means of a suitable header board conforming to the cross-section of the pavement?

25. SURPLUS - DEFICIENCY DETERMINATION

Is a daily check being made on the yield of produced concrete?

26. CURING

Are the pavement surface and edges being cured by one of the following methods:

(a) Waterproof Paper Method. Are the surfaces being covered as soon as possible with blankets or tear-free reinforced kraft paper, with 300 millimeter laps, properly weighted? Has the pavement been wetted with a fine spray first?

(b) Polyethylene Sheeting Method. Are surfaces covered as soon as possible with 30 meter long sheets of white polyethylene, with 300 millimeter laps, properly weighted? Has the pavement been wetted with a fine spray first?

(c) Wetted Burlap Method. Are surfaces covered as soon as possible with two layers of wet burlap, with 150 millimeter laps? Kept saturated by means of a mechanically operated sprinkling system or an impermeable covering? (Alternate: one burlap and one burlene blanket)

HAUL TICKET FOR TRUCK MIX CONCRETE

PROJECT NO. ____________ DATE: ____________
BATCHED FROM (PLANT) ____________ TRUCK NO. ____________
NO. CUBIC METERS ____________ CLASS OF CONCRETE ____________

BATCH WEIGHTS

CEMENT BRAND ____________
kg ____________
FINE AGGR. SOURCE ____________
kg ____________
COARSE AGGR. SOURCE ____________
kg ____________
FLY ASH SOURCE ____________
kg ____________

AIR ENTRAINMENT BRAND ____________
grams ____________
RETARDER BRAND ____________
grams ____________
WATER REDUCER BRAND ____________
ml ____________

WATER
MAXIMUM WATER ALLOWED, Liter ____________
FREE MOISTURE
CA Liters ____________
FA Liters ____________
WATER ADDED AT PLANT Liters ____________
MAXIMUM WATER THAT CAN BE ADDED AT THE SITE Liters ____________

PLANT

TIME WATER ADDED TO MIX ____________ AM PM

NUMBER OF MIXING

Signature ____________

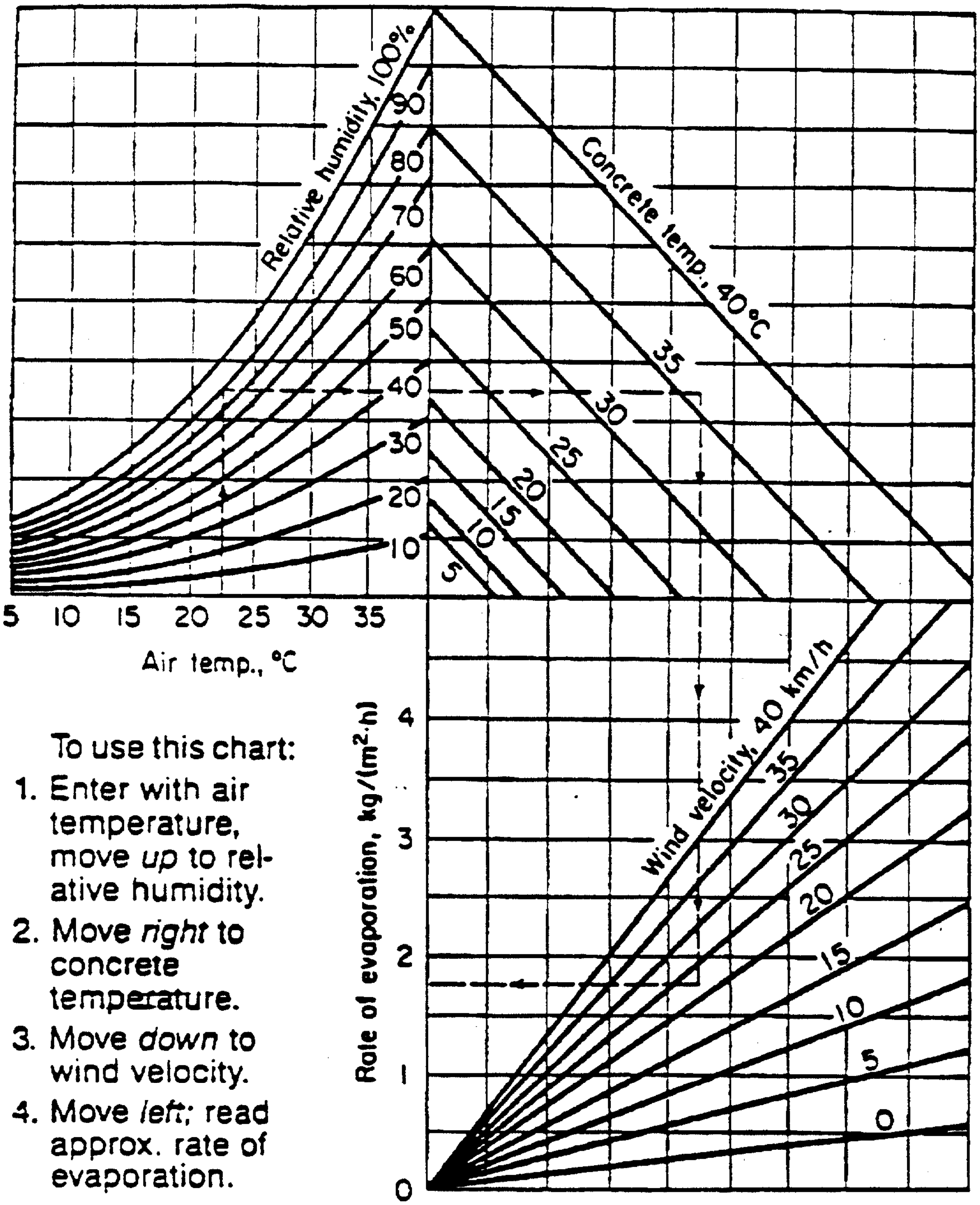
SITE

TIME DISCHARED COMPLETED ____________ AM PM
WATER ADDED AT JOBSITE Liters ____________
TOTAL WATER IN BATCH Liters ____________
MIXING REVOLUTIONS AT SITE ____________
TOTAL NO. OF REVOLUTIONS
SLUMP ____________ AIR ____________

UNIT WEIGHT ____________
CONC. TEMP ____________
AIR TEMP ____________

Signature ____________

NOMOGRAPH USED TO
DETERMINE EVAPORATION RATE



To use this chart:

1. Enter with air temperature, move *up* to relative humidity.
2. Move *right* to concrete temperature.
3. Move *down* to wind velocity.
4. Move *left*; read approx. rate of evaporation.

CHECKLIST FOR QUALIFICATION OF FACILITIES FOR PRESTRESSED CONCRETE PRODUCTION

1. Items which require written approval: (check applicable blanks)

   (a) Plans and computations of facilities ________

   (b) Concrete mix design (should include curves for 28-day strength) vs W/C Ratio: ________

   (c) Curing method ________

   (d) Epoxy-sand mortar, if used ________

   (e) Coal tar epoxy, if used ________

   (f) Water reducer-retarder ________

   (g) Design Engineer should be approved by State DOT ________

   (h) Gauge calibration should be certified ________

   (i) Computations regarding beam tests (2 weeks prior to testing) ________

2. What is length and capacity of stressing bed(s)

   Bed No. ______ Length ______ Capacity ______
   Bed No. ______ Length ______ Capacity ______
   Bed No. ______ Length ______ Capacity ______

3. Procedure of prestressing (pretensioning) and stress release:

   (a) Jacks, carriages, and struts are adequate to attain and maintain design stress.
   Yes ________ No ________
   Comments:
   ____
   ____

(b) Stressing of straight strands: (check applicable blanks)
Single strand method ______
Multiple strand method ______
Comments:
______
______

(c) Stressing of draped strands (check applicable blanks)
Single strand method ______
Multiple strand method ______
Final draped position ______ both ends ______
Partial draped position ______ one end ______
Comments:
______
______

(d) Single strand jack available.
Yes ______ No ______

(e) Is an accurate dynamometer available for use in applying initial tension to the strands?
Yes ______ No ______

(f) What is proposed initial load to be applied
______ lbs.

(g) Is there a permanent, accurate linear gauge with which to measure elongation?
Yes ______ No ______

4. Forms: (Make comments in spaces provided)

(a) Metal
______
______

(b) True to shape and dimensions
______
______

(c) Adequate in number
______
______

(d) Condition and composition of bulkheads

____

____

(e) Type of hold-down device to be used

____

____

(f) Is provision being made to maintain 25 millimeter concrete cover over hold-down device?

____

____

(g) Are bulkheads and hold-down devices adequate to maintain dimensions of strand centers as shown on the plans?

____

____

5. Are facilities adequate for proper storage and handling of bridge members?
Yes __________ No __________

(a) Approximate available storage area ____________________

(b) Condition of storage area ____________________

6. Are facilities available for properly testing a member of the design type to be fabricated?
Yes __________ No__________ (if No explain)

____

____

7. Are adequate lighting facilities available in the event that placing of concrete at night is necessary?
Yes __________ No __________

8. Vibrating equipment:

(a) Condition ______________________________

(b) Number to be used in placing ______________

(c) Two spaces available ____________________

9. Source of Materials:

(a) Steel Wire and Strand (manufacturer)

____

(b) Cement (type and brand name)

____

(c) Coarse Aggregate (producer and location)

____

(d) Sand (producer and location)

____

(e) Retarder (brand name)

____

(f) Form Oil (type and name)

____

(g) Reinforcing Steel (producer)

____

10. Type of concrete mixing facilities: mixed at plant ____________________
Ready Mix concrete ____________________

(a) Are concrete batching facilities adequate to ensure good quality and sufficient quantity to avoid delays under all working conditions?
Yes __________ No __________

11. Testing equipment available: (check applicable blanks)

(a) Plastic cylinder molds ____________________
No. Available ____________________

(b) Slump Cone ____________________

(c) Air content device ____________________
(pressure __________ volumetric __________)

(d) Facilities for testing cylinders available at (proposed location)

____

12. Requirements for steam cure method:

(a) Three (3) recording thermometers available

____

(b) Temperature record charts

____

(c) Adequate temperature control valves

____

(1) What are the increments of spacing of control valves?

____

13. Are facilities available for proper protection and handling of component materials in storage? (Rate "S" if satisfactory, "U" if unsatisfactory, and "NA" if not applicable)

(a) Wire and/or strand ________
(b) Reinforcing steel ________
(c) Structural steel ________
(d) Cement ________
(e) Coarse Aggregate ________
(f) Sand ________

14. Is there a suitable shelter (at least 14 square meters floor space, facilities for lights, heat, desk(s), etc.) available for the inspector's use?

____

15. Personnel present during inspection of plants:

Producers/Contractors	Highway Department
________	________
________	________
________	________
________	________
________	________

GUIDE FOR QUALITY CONTROL PLAN FOR PORTLAND CEMENT CONCRETE

REQUIREMENTS

1. <u>General Requirements</u>:

The contractor should provide and maintain a quality control system that will provide reasonable assurance that all materials and products submitted to the State for acceptance will conform to the contract requirements whether manufactured or processed by the contractor or procured from suppliers or subcontractors or vendors. The contractor should perform or have performed the inspections and tests required to substantiate product conformance to contract document requirements and should also perform or have performed all inspections and tests otherwise required by the contract. The quality control inspections and tests should be documented and should be available for review by the engineer throughout the life of the contract.

2. <u>Quality Control Plan</u>:

The contractor should prepare a Quality Control Plan detailing the type and frequency of inspection, sampling and testing deemed necessary to measure, and control the various properties of materials and construction governed by the Specifications. As a minimum, the sampling and testing plan should detail sampling location and techniques, and test frequency to be utilized. The Quality Control Plan should be submitted in writing to the engineer at the preconstruction conference.

The Plan should identify the personnel responsible for the contractor's quality control. This should include the company official who will act as liaison with State personnel, as well as the Certified Portland Cement Concrete Technician who will direct the inspection program.

The class or classes of concrete involved will be listed separately. If existing mix designs are to be utilized, the Mix Design Numbers should be listed.

Quality control sampling, testing, and inspection should be an integral part of the contractor's quality control system. In addition to the above requirements, the contractor's quality control system should document the quality control requirements shown in Table 1. The quality control activities shown in Table 1 are considered to be normal activities necessary to control the production and placing of a given product or material at an acceptable quality level. To facilitate the States' activities, all completed gradation samples should be retained by the contractor until further disposition is designated by the State.

It is intended that sampling and testing be in accordance with standard methods and procedures, and that measuring and testing equipment be properly calibrated. If alternative sampling methods, procedures and inspection equipment are to be used, they should be detailed in the Quality Control Plan.

3. Documentation:

The contractor should maintain adequate records of all inspections and tests. The records should indicate the nature and number of observations made, the number and type of deficiencies found, the quantities approved and rejected, and the nature of corrective action taken as appropriate. The contractor's documentation procedures will be subject to the review and approval of the State prior to the start of the work and to compliance checks during the progress of the work.

4. Charts and Forms:

All conforming and non-conforming inspections and tests results should be kept complete and should be available at all times to the State during the performance of the work. Batch tickets and gradation data will be submitted to the State as the work progresses. All test data will be plotted on control charts. It is normally expected that testing and charting will be completed within 48 hours after sampling.

All charts and records documenting the contractor's quality control inspections and tests should become property of the State upon completion of the work.

5. Corrective Action:

The contractor should take prompt action to correct conditions which have resulted, or could result, in the submission to the State of materials and products which do not conform to the requirements of the Contract documents.

6. Non-Conforming Materials:

The contractor should establish and maintain an effective and positive system for controlling non-conforming material, including procedures for its identification, isolation, and disposition. Reclaiming or reworking of non-conforming materials should be in accordance with procedures acceptable to the State.

All non-conforming materials and products should be positively identified to prevent use, shipment, and intermingling with conforming materials and products. Holding areas, mutually agreeable to the State and the contractor, should be provided by the contractor.

7. Acceptance:

The State will monitor the performance of the contractor's quality control plan and will perform verification testing to ensure that proper sampling and testing procedures are used by the contractor. The State may shut down the contractors operations for failing to follow the approved process control plan. All acceptance testing will be performed by State personnel.

TABLE 1

CONTRACTOR'S QUALITY CONTROL REQUIREMENTS

Minimum Quality Control Requirement	Frequency
A. PLANT AND TRUCKS	
1. Mixer Blades	Prior to Start of Job and weekly
2. Scales	Prior to Start of Job and weekly
a. Tared	Daily
b. Calibrate	Prior to Start of Job
c. Check Calibration	Weekly
3. Gauges and Meters - Plant and Truck	
a. Calibrate	Yearly
b. Check Calibration	Weekly
4. Admixture Dispenser	
a. Calibrate	Prior to Start of Job
b. Check Operation and Calibration	Daily
B. AGGREGATES	
1. Fine Aggregate	
a. Gradation	21 Days
b. Deleterious Substances	Daily
c. Moisture	Daily
2. Coarse Aggregates	
a. Gradation	21 Days
b. Percent Passing No. 200 Sieve	Daily
c. Moisture	Daily
C. PLASTIC CONCRETE	
1. Entrained Air Content	One Per 1/2 Day Operation
2. Consistency	One Per 1/2 Day of Operation
3. Temperature	One Per 1/2 Day of Operation
4. Yield	One Per 1/2 Day of Operation

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: Summary of State Highway Practices on Rigid Pavement Joints and Their Performance

Date: MAY 19 1987

From: Chief, Pavement Division

Reply to Attn of: HHO-13

To: Regional Federal Highway Administrators
Regions 1-10

The American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Construction is presently preparing a new edition of the AASHTO "Guide Specifications for Highway Construction." The AASHTO decided to survey the States' current practices on rigid pavement joints to help rewrite Section 514 titled "Joints." We agreed to assist them by preparing and then summarizing that survey (spacing, skew, dowel cages, epoxy coated bars, filler material, etc.). Attached for your information is a summary of the survey results. Please note there were five State highway agencies that did not respond to the survey.

The survey is intended to cover the States' current practices and recent performances with rigid pavement joints. It may not necessarily reflect each States's current standard specifications. However, we believe the summary contains worthwhile information that can be used as a reference tool for highway engineers.

We will appreciate your forwarding copies to the division offices. Copies of the survey have already been sent to the State highway agencies by the AASHTO Subcommittee on Construction. Any questions or comments may be directed to Mr. Don Voelker of my staff at FTS 366-1333.

Norman J. Van Ness

Norman J. Van Ness

Attachment

Quotations

(1). Inserts are no longer allowed as of 10/86.
(2). For CR pavements, there are four expansion joints @30 ft.; sealant is AASHTO M-33.
(3). Depends on a Bridge Movement Rating.
(4). Every second transverse joint is sawed within 4-12 hours.
(5). Sheet steel is used to form the keyway.
(6). Plastic coatings (17 mils) and powdered epoxy resins (7 mils) are also allowed.
(7). Only plain pavement joints are skewed at 2/12.
(8). Ravelling during sawing is not allowed but sawing must be done to preclude random cracking.
(9). Preformed bituminous, cork, or rubber plus compression seal.
(10). Only plain pavement is skewed at 2/12.
(11). Plastic coatings (11 mils) and red lead paint (no thickness specified) are also allowed.
(12). Yield strength of 40 ksi and ultimate strength of 70 ksi.
(13). Type A is low bond strength Doubl Coat by Republic Steel.
Type B is high bond strength, ie. Scotchkotz 202, Flintflex S31-6080, etc. but must have bond breaker HC-70, MS-2a or RC-250.
(14). At PC and PT of curves 2 deg. 30 min. and greater and at every eighth joint constructed between 9/15 and 4/15.
(15). Faulting occurs on plastic soils where dowels are not present.
(16). One coat of paint conforming to Federal Spec. TT-P-866 Type II or TT-P-645 or TT-P-310 or steel str. painting council spec. SSPC Paint II.
(17). Inside 4 ft.-13.3 ft.; centerline 3.3 ft.; outside 10 ft.-5.3 ft..
(18). Initial sawing is contractors option. Sawing for joint reservoir is a minimum of 72 hours.
(19). Sawing for preliminary crack control is done on approx. 50 ft. intervals with a 1/8-in. blade and a depth of D/4. Final sawing is done within 24-36 hours after concrete pour.
(20). New York's minimal problems related to slab cracking and joint spalling result from sawing too late. Faulting problems are present only in older pavements where a two-piece malleable iron load transfer device was used.
(21). Control joints (92-ft. intervals) are sawed as soon as possible with only minor ravelling allowed; remaining joints are sawed between 24-48 hours.
(22). Reinforced dowelled pavement is not sawed on skew. All others are at 2/12.
(23). Required but type not specified.
(24). Any grade of steel conforming ASTM A615 is permitted.
(25). (Concrete to Concrete) Low modulus silicone (cold) is preferred.
(Concrete to Asphalt) Hot rubberized asphalt ASTM D-3406 and ASTM D-3405.
(26). Rubberized asphalt over filler and/or polychloroprene compression seals.
(27). Either epoxy (7 mil thickness) or plastic (25 mil thickness) coatings are allowed.
(28). Initial sawing is 2 inches for plain pavement and 1 3/8 inches for plain dowelled pavement.
(29). Plain pavement initial saw depth is d/4. Plain Dowelled initial saw depth is d/3.

Quotations (con't)

(30). Alignment tolerances are plus or minus one-half inch of specified depth.
(31). Longitudinal sawed joints shall be cut before any equipment or vehicles are allowed on the pavement.

Other areas to be considered.

(1). California believes positive drainage mitigates transverse joint faulting.
(2). Georgia and Indiana believe a minor amount of ravelling during sawing is acceptable. If no ravelling is occurring, sawing has been delayed too long.
(3). Indiana reports formed groove-type contraction joints shall be used if early sawing causes erratic cracking.
(4). Iowa has a specification on maximum loading from the weight of saws. See attached.
(5). Kansas limits the use of inserts to the period May-Sept. to prevent longitudinal cracks.
(6). Louisiana believes transverse joint problems are attributable to moistu and incompressibles not the metnod of construction.
(7). Mississippi is experiencing transverse cracking on continuous reinforced pavements.
(8). New Jersey believes TRB Synthesis of Highway Practice 19 contains useful information.
(9). Ohio's keyed longitudinal joints have a proven poor performance.
(10). Delaware recommends that edges of construction joints shall be tooled to a 1/8 inch radius; sawed joints are chamfered similarly. Also, joints shall be thoroughly cleaned by brushing, air blasting, sand blasting, or other means to completely remove all foreign materials.
(11). Puerto Rico reports pumping problems due to no joint sealing caused by lack of proper maintenance.
(12). Colorado requires longitudinal sawed joints to be cut before any equipment or vehicles are allowed on the pavement. Also, every second transverse joint shall be sawed within 4 to 12 hours after pavement placement. The intermediate joints shall be sawed within 48 hours after pavement placement.

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: Bondbreakers for Portland Cement Concrete Pavement with Lean Concrete Bases

Date: JUN 13 1988

From: Chief, Pavement Division

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators

During the past 2 years, we have reviewed several projects with Portland Cement Concrete (PCC) pavements constructed over lean concrete bases, which have experienced premature cracking. We have suspected that the principal cause of the distress was the partial bonding of the PCC slab to the lean concrete bases, during the period of joint and crack formation in jointed and continuous PCC pavements. Generally, this bond was believed to be weak, and would be lost within 6 to 12 months, because of stresses caused by loading and/or temperature variations. This weak bond would also be broken during coring or following the development of pavement distress. However, recently on two projects, cores were retrieved with the slab bonded to the lean concrete base. These projects which lend support to our theory are described below:

1. A Continuously Reinforced Concrete Pavement (CRCP) began experiencing premature punchouts. The pavement section consisted of 9 inches of CRCP over 6 inches of lean concrete base on a cement treated subgrade. During coring operations 5-plus years after construction, approximately 30 percent of the cores indicated the slab was bonded to the base. Failure of this pavement is believed to have resulted because the amount and location of steel was designed based on the unbonded condition. When bonding occurred, the slab was significantly under reinforced, and the reinforcement was located well above the neutral axis of the composite section. As a result, the steel was overstressed causing excessive crack widths, steel ruptures, and ultimately punchouts.

2. An 8-inch Jointed Plain Concrete Pavement (JPCP) over an 8-inch lean concrete base experienced random cracking within 6 months after construction. Coring revealed that the cracks were forming from the top of the slab downward, and were not reflective cracks. Also, cores of numerous sawed joints revealed that cracking had not occurred at the joints. Project records and discussions with project personnel indicated that sawing was done in a timely manner. There was no correlation between cracking, and temperature extremes at the time of construction. A number of the cores taken during the investigation of the cracking were retrieved with the slab bonded to the lean concrete base. We now believe that partial bonding during the joint formation period resulted in the saw cuts being an inadequate depth to force cracking at the joints. The depth of the saw cuts was based on the thickness of the slab in the unbonded condition.

We recognize that some States have been working in strengthening their asphalt concrete mix design and field control practices. These efforts are appropriate and continued involvement of all the field offices in encouraging conformance with the attached TA will be expected.

Other factors such as truck weights, high tire pressures, etc., also contribute to the rutting and stripping problems and we are working on these issues. We are convinced though that significant gains in solving rutting and stripping problems can be achieved by using quality materials and strengthening specifications and construction practices. We expect those States where rutting and stripping is a problem to include a priority effort to improve the design and construction of asphalt concrete pavements. The Pavement Division and the Construction and Maintenance Division are available upon request to provide technical support and guidance, which may be necessary in achieving these actions.

R. D. Morgan
R. D. Morgan
Executive Director

Attachment

The use of polyethylene sheeting is not recommended for use as a bondbreaker, because of construction problems which have occurred on projects where it was specified.

We are also currently evaluating the magnitude of slab curling on pavements, constructed over lean concrete bases. Actual field measurements of curling and deflection are being made on pavements in four States in Region 4. We believe the stiffness of the lean concrete base tends to cause higher curling stresses. In longer slabs, the combined curling and load stresses can exceed the slab strength resulting in transverse slab cracking. We suggest that to be on the safe side, when JPCP pavements are constructed over lean concrete bases, the joint spacing be limited to a maximum of 15 feet.

We intend to closely monitor the performance of PCC pavements over lean concrete bases, and would appreciate receiving feedback on the performance of this type of pavement in your region. Please contact Mr. John Hallin at FTS 366-1323, if you have any questions or comments on the use or performance of lean concrete bases.

Norman J. Van Ness

Chapter 4

Flexible Pavement

CHAPTER 4

FLEXIBLE PAVEMENT

4.1 TA 5040.27, Asphalt Concrete Mix Design and Field Control, February 16, 1988.

4.2 Prevention of Premature Distress in Asphalt Concrete Pavements, Technical Paper 88-02, April 18, 1988.

4.3 Guidelines on the Use of Bag-House Fines, April 7, 1988.

4.4 Reserved.

4.5 State of the Practice on the Design and Construction of Asphalt Paving Materials with Crumb Rubber Modifier, Report Number FHWA-SA-92-022, June 9, 1992.

4.6 Reserved.

4.7 Processed Used-Oil and Heavy Fuel Oils for Use in Hot Mix Asphalt Production, June 21, 1990.

4.8 Aggregate Gradation for Highways - 0.45 Particle Size Distribution Curve, 1962.

- Aggregate Gradation: Simplification, Standardization, and Uniform Application
- A New Graphical Chart for Evaluating Aggregate Gradation

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: ACTION:
Asphalt Mix Design and Field Control

Date: February 16, 1988

From: Executive Director

Reply to Attn of: HHO-12
HHO-30

To: Regional Federal Highway Administrators
Direct Federal Program Administrator

There are presently about 1,420,000 miles of intermediate or high type flexible pavements on State highways and local roads. This represents about 70 percent of the paved mileage on all public roads and streets. In 1986, about $2 billion of asphalt concrete was placed on Federal-aid projects and this amount will likely increase in the future. Information that has been gathered over a number of years by the States, FHWA, and the asphalt industry has revealed that a number of asphalt concrete pavements are experiencing premature distress and significantly reduced pavement performance periods. Types of distress identified have included bleeding, cracking, shoving, rutting, stripping, and raveling.

Two distress types, rutting and stripping, have had a high frequency of occurrence over wide areas of the United States. The reduction in pavement performance due to rutting or stripping is potentially severe from a national perspective. Due to the continuing major investment which is being made in asphalt concrete pavements and as a result of reports indicating premature rutting and stripping problems, we appointed an Ad Hoc Task Force to examine the problems of asphalt concrete pavement rutting and stripping, and to develop FHWA policy recommendations. The Task Force has completed its assignment and a copy of its report was provided to each region and division office.
In accordance with one of the Task Force's major recommendations, our Technical Advisory (TA) on this subject has been updated to reflect current knowledge. Attached for your immediate use is a copy of the TA "Asphalt Concrete Mix Design and Field Control." This TA sets forth guidance and recommendations relating to asphalt concrete paving. It covers the areas of materials selection, mixture design, mixture production, and mixture placement. The TA is intended primarily for application on high type facilities.

Each division office is to initiate an effort to compare the updated TA to present State specifications and construction practices. Differences and/or deviations are to be discussed with the State and, if appropriate, industry representatives. Some States have found it beneficial to have a formal committee composed of State, FHWA, and industry personnel to scrutinize the State's mix design, and field control procedures, and iron out differences with the TA. The TA is a consensus of current best practice, and serious consideration should be given to adopting its recommendations. Sound engineering judgment must be used in determining what is best for each particular State but deviations from the TA recommendations should be supportable.

We recognize that some States have been working in strengthening their asphalt concrete mix design and field control practices. These efforts are appropriate and continued involvement of all the field offices in encouraging conformance with the attached TA will be expected.

Other factors such as truck weights, high tire pressures, etc., also contribute to the rutting and stripping problems and we are working on these issues. We are convinced though that significant gains in solving rutting and stripping problems can be achieved by using quality materials and strengthening specifications and construction practices. We expect those States where rutting and stripping is a problem to include a priority effort to improve the design and construction of asphalt concrete pavements. The Pavement Division and the Construction and Maintenance Division are available upon request to provide technical support and guidance, which may be necessary in achieving these actions.

R. D. Morgan
Executive Director

Attachment

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

SUBJECT	FHWA TECHNICAL ADVISORY
ASPHALT CONCRETE MIX DESIGN AND FIELD CONTROL	T 5040.27 March 10, 1988

1. PURPOSE. To set forth guidance and recommendations relating to asphalt concrete paving, covering the areas of materials selection, mixture design, and mixture production and placement. The procedures and practices outlined in the Technical Advisory (TA) are directed primarily towards developing quality asphalt concrete pavements for high-type facilities. The TA can also be used as a general guide for low-volume facilities.

2. Cancellation. Federal Highway Administration (FHWA) Technical Advisory T 5040.24, Bituminous Mix Design and Field Control, dated August 22, 1985, is cancelled.

3. BACKGROUND

a. Over one-half of the Interstate System and 70 percent of all highways are paved with hot-mix asphalt concrete. Asphalt concrete is probably the largest single highway program investment today and there is no evidence that this will change in the near future. However, there is evidence that the number of premature distresses in the nation's recently constructed asphalt pavements is increasing. Heavier truck axle weights, increased tire pressures, and inadequate drainage are some of the factors leading to the increase in premature distress. The FHWA has been concerned with the deterioration in quality of asphalt concrete pavements for many years and in 1987 a special FHWA Ad Hoc Task Force studied two of the most common distresses existing today and subsequently issued a report titled "Asphalt Pavement Rutting and Stripping." The report contained both short-term and long-term recommendations for improving the quality of asphalt pavements.

b. With the variables of environment, component materials, and traffic loadings found throughout the United States, it is not surprising that there are many State-to-State or regional variations of design and construction requirements. No one set of specifications can achieve the same results in all States because of the factors mentioned above. However, there are many things that States can do to improve their current mix design and field control procedures to ensure that quality asphalt pavements will be constructed. This TA incorporates many of the FHWA Task Force recommendations and presents the current

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state-of-the-art in materials, mix design, plant operation, laydown and compaction, and other areas relating to quality hot-mix asphalt pavements.

4. MATERIALS

a. Aggregate is the granular material used in asphalt concrete mixtures which make up 90-95 percent of the mixture weight and provides most of the load bearing characteristics of the mix. Therefore, the quality and physical properties of the aggregates are critical to the pavement performance. The following is recommended:

(1) Aggregates should be non-plastic. The presence of clay fines in an asphalt mix can result in problems with volume swell and adhesion of asphalt to the rock contributing to stripping problems. The minus #4 sieve material should have a minimum sand equivalent value of 45 using the test method described in the American Association of State Highway and Transportation Officials (AASHTO) specification (AASHTO T176).

(2) A limit should be placed on the amounts of deleterious materials permitted in the aggregates. Specifications should limit clay lumps and friable particles to a maximum of one percent.

(3) Durability or weathering resistance should be determined by sulfate soundness testing. Specifications should require a sodium or magnesium sulfate test using the limits described in the AASHTO specification M29.

(4) Aggregate resistance to abrasion should be determined. Specifications should require a Los Angeles abrasion loss of 45 percent or less (AASHTO T96).

(5) Friction between aggregate particles is dependent on aggregate surface roughness and area of contact. As surface friction increases, so does resistance of the mix to deformation. Specifications should require at least 60 percent of the plus #4 sieve material to have at least two mechanically induced fractured faces.

(6) The quality of natural sand varies considerably from one location to another. Since most natural sands are rounded and often contain a high percentage of undesirable materials, the amount of natural sand as a general rule should be limited to 15 to 20 percent for high volume pavements and 20 to 25 percent for medium and low volume pavements. These percentages may increase or decrease depending on quality of the natural sand and the types of traffic to which the pavement will be subjected.

(7) For adequate control, aggregate gradations should be specified from the maximum particle size to the #200 sieve so each successive sieve opening is about 1/2 the previous sieve opening (for example, 1 inch, 1/2 inch, #4, #8, #16, #30, #50, #100, #200). The only accurate method to determine the amount of minus #200 sieve material is to perform a wash gradation in accordance with AASHTO T27 and AASHTO T11.

(8) The ratio of dust (minus #200 sieve material) to asphalt cement, by mass, is critical. Asphalt concrete mixes should require a maximum dust asphalt ratio of 1.2 and a minimum of 0.6.

(9) A tool which is very useful in evaluating aggregate gradations is the 0.45 power gradation chart. All mixes should be plotted on these charts as part of the mix design process (Attachment 1).

(10) An aggregate's specific gravity and absorption characteristics are extremely important in proportioning and controlling the mixture. It is recommended that AASHTO T209 be used to determine the maximum specific gravity of asphalt concrete mixes. States not using AASHTO T209 should be aware of the difficulty of determining the theoretical maximum density using individual ingredient specific gravities and their percentages in the mixture. These difficulties will result in inaccuracies in determining the specific gravity of the mixture. These inaccuracies will carry through to the calculation of the densities in the compacted mat and may result in improperly compacted pavements. It is also necessary to determine the bulk dry specific gravity of the aggregate in order to determine the voids in the mineral aggregate (VMA).

The target value for VMA should be obtained through the proper distribution of aggregate gradation to provide adequate asphalt film thickness on each particle and accommodate the design air void system. In addition, tolerance used in construction quality control should be such that the mix designed is actually produced in the field.

b. Asphalt grade and characteristics are critical to the performance of the asphalt pavement. The following is recommended:

(1) Grade(s) of asphalt cement used in hot-mix paving should be selected based on climatic conditions and past performance.

(2) It is recommended that asphalt cement be accepted on certification by the supplier (along with the testing results) and State project verification samples. Acceptance procedures should provide information on the physical properties of the asphalt in a timely manner.

(3) The physical properties of asphalt cement that are most important to hot-mix paving are shown below. Each State should obtain this information (by central laboratory or supplier tests) and should have specification requirement(s) for each property except specific gravity.

(a) Penetration 77° F

(b) Viscosity 140° F

(c) Viscosity 275° F

(d) Ductility/Temperature

(e) Specific Gravity

(f) Solubility

(g) Thin Film Oven (TFO)/Rolling TFO; Loss on Heating

(h) Residue Ductility

(i) Residue Viscosity

(j) Low temperature cracking is related to the physical properties of the asphalt and may be increased by the presence of wax in the asphalt. The low temperature ductility test at 39.2° F (4° C) can indicate where this may be a problem. The test is performed at a pull speed of 1 cm/min. Typical specification requirements are:

AASHTO M226	Table 2
AC 2.5	50 + cm
AC 5	25 + cm
AC 10	15 + cm
AC 20	5 + cm

(4) The temperature viscosity curves or absolute and kinematic viscosity information should be available at the mixing plant for each shipment of asphalt cement. This can identify a change in asphalt viscosity which necessitates a new mix design. Each State should provide temperature/viscosity information on the asphalt used in the laboratory mix design to the projects. Differences in the viscosity (as well as the penetration) of the asphalt from the asphalt used in the mix design may indicate the necessity to redesign the mix (Attachment 2).

5. MIX DESIGN

a. Asphalt concrete mixes should be designed to meet the necessary criteria based on type of roadway, traffic volumes, intended use, i.e., overlay on rigid or flexible pavements, and the season of the year the construction would be performed. Each State's mix design criteria should be as follows.

Property	Heavy Traffic Design (>1,000,000 ESAL*)	Medium Traffic Design (10,000-1,000,000 ESAL)	Light Traffic Design (<10,000 ESAL)
Marshall			
Compaction Blows	75	50	35
Stability (min.)	1,500	750	500
Flow	8-16	8-18	8-20
Hveem			
Stability (min.)	37	35	30
Swell	0.030 in.	0.030 in.	0.030 in.
Void Analysis			
Air Voids	3-5	3-5	3-5

* Equivalent Single Axle Load

MINIMUM PERCENT VOIDS IN MINERAL AGGREGATE (VMA)

Nominal Maximum Particle Size U.S.A. Standard Sieve Designation	Minimum Voids in Mineral Aggregate Percent
No. 16	23.5
No. 8	21
No. 4	18
3/8 in.	16
1/2 in.	15
3/4 in.	14
1 in.	13
1-1/2 in.	12
2 in.	11.5
2-1/2 in.	11

b. Standard mix design procedures (Marshall, Hveem) have been developed and adopted by AASHTO, however, some States have modified these procedures for their own use. Any modification from the standard procedure should be supported by correlation testing for reasonable conformity to the design values obtained using the standard mix design procedures.

c. Stripping in the asphalt pavements is not a new phenomenon, although the attention to it has intensified in recent years. Moisture susceptibility testing should be a part of every State's mix design procedure. The "Effect of Water on Compacted Bituminous Mixtures" (immersion compression test) (AASHTO T165) and "Resistance of Compacted Bituminous Mixture to Moisture Induced Damage" (AASHTO T283) are currently the only stripping test procedures which have been adopted by AASHTO. The AASHTO T283, commonly known as the Lottman Test, requires that the test specimens be compacted so as to have an air void content of 7 ± 1 percent, while AASHTO T165 does not. This air void content is what one would expect in the mat after construction compaction. There is considerable research underway on developing better tests for determining moisture damage susceptibility of the aggregate asphalt mixtures. One of the most promising test procedures is that developed by Tunnicliff and Root as reported in the National Cooperative Highway Research Program (NCHRP) Report 274. This test is similar to AASHTO T283, but it takes less time to perform. In the majority of cases hydrated lime and portland cement have proven to be the most effective anti-stripping additives.

d. The determination of air voids in the laboratory mix is a critical step in designing and controlling asphalt hot-mix. In order to determine air voids, the theoretical maximum density or the maximum specific gravity of the mix must be determined. This can be accomplished by using the "Maximum Specific Gravity of Bituminous Paving Mixtures" (Rice Vacuum Saturation) (AASHTO T209).

e. Proper mix design procedures require that each mix be designed using all of the actual ingredient materials including all additives which will be used on the project.

f. The complete information on the mix design should be sent to the plant. The following information should be included in the mix design report and sent to the plant.

(1) Ingredient materials sources

(2) Ingredient materials properties including:

(a) Specific gravities

(b) L. A. Abrasion

(c) Sand equivalent

(d) Plastic Index

(e) Absorption

(f) Asphalt temperature/viscosity curves or values

(3) Mix temperature and tolerances

(4) Mix design test property curves

(5) Target asphalt content and tolerances

(6) Target gradations for each sieve and tolerances

(7) Plot of gradation on the 0.45 power gradation chart, and

(8) Target density

g. Formal procedures should be established to require that changes to mix designs be approved by the same personnel or office that developed the original mix design.

h. After start-up, the resulting mixture should be tested to verify that it meets all of the design criteria.

6. PLANT OPERATIONS

a. In order to assure proper operation, an asphalt plant must be calibrated and inspected. Plant approval should be required and should cover each item on the asphalt plant checklist (Attachment 3).

b. To avoid or mitigate unburned fuel oil contamination of the asphalt mixture, the use of propane, butane, natural gas, coal or No. 1 or No. 2 fuel oils is recommended.

c. If the asphalt cement is overheated or otherwise aged excessively, the viscosity of the recovered asphalt will exceed that of the original asphalt by more than four times. However, if the viscosity of the recovered asphalt is less or even equal to the original viscosity, it has probably been contaminated with unburned fuel oil.

d. For drum mixer and screenless batch plants there should be three separate graded stockpiles for surface courses and four for binder and base courses. Each stockpile should contain between 15 to 50 percent by weight of the aggregate size in the mix design. The plus #4 sieve aggregate stockpile should be constructed in lifts not exceeding 3 feet to a maximum height of 12 feet. There should be enough material in the stockpiles for at least 5 days of production. The plant should be equipped with a minimum of four cold feed bins with positive separation.

e. Control testing of gradation and asphalt content should be conducted to assure a quality and consistent mixture. In many States, the contractor or supplier is required to do this testing.

f. Acceptance testing should be conducted for gradation and asphalt content of the final mixture.

g. The plotting of control and acceptance test results for gradation, asphalt content, and density on control charts at the plant provides for easy and effective analysis of test results and plant control.

h. The moisture content of the aggregate must be determined for proper control of drum mixer plants. The asphalt content is determined by the total weight of the material that passes over the weigh bridge with the correction made for moisture. Sufficient aggregate moisture contents need to be performed throughout the day to avoid deviations in the desired asphalt content.

i. Moisture contents of asphalt mixtures is also important. The extraction and nuclear asphalt content gauge procedures will count moisture as asphalt. For this reason, a moisture correction should be made. In addition, high moisture contents in asphalt mixtures can lead to compaction difficulty due to the cooling of the mix caused by evaporation of the moisture. This is particularly important with drum mixer mixes which require moisture for the mixing process. Some States specify a maximum moisture content behind the paver. A recommended maximum moisture content behind the paver is 0.5 percent.

7. LAYDOWN AND COMPACTION

a. Prior to paving start-up, equipment should be checked to assure its suitability and proper function. Project equipment approval should include the items shown on the project inspection checklist (Attachment 4).

b. Paving start-up should begin with a test strip section. This will allow for minor problems to be solved, establishment of roller patterns and number of passes, and will assure that proper placement and compaction can be attained.

c. In order to assure proper placement and compaction, it is essential that the mat be placed hot. Establishment of and compliance with the following items should be included; minimum mix, underlying pavement, and ambient temperatures. Cold weather and early or late season paving should be avoided. The practice of raising the temperature of the mixture to combat the cold conditions should not be permitted, as this will contribute to excessive aging of the asphalt cement.

d. The use of a pneumatic roller in the compaction process is strongly encouraged. When used in the intermediate rolling it will knead and seal the mat surface and aid in preventing the intrusion of surface water into the pavement layers. It will also contribute to the compaction of the mat.

e. Density requirements should be established to result in an air void system in the mat of 6-8 percent immediately after construction. This allows for the inherent additional densification under traffic to an ultimate air void content of about 3-5 percent. Density acceptance specifications should require a percentage of maximum density as determined by AASHTO T209. A percentage of test strip density or Marshall laboratory density can be used provided each is related to the maximum density. The specified density should be attained before the mat temperature drops below 175^{o} F.

f. Density measurement should be accurate, taken frequently, and the results made available quickly for each day of production. Density should be determined by test cores, or by properly calibrated nuclear test gauges. Specifications should require several tests to be averaged to determine density results for acceptance.

g. Successive hot-mix courses should not be placed while previous layers are wet. To avoid, or minimize the penetration of water into base and binder courses, paving operations should be scheduled so that the surface layer(s) is placed within a reasonable period after these courses are constructed. To the greatest extent possible, construction should be planned to avoid the necessity of leaving layers uncovered during wet seasons of the year.

8. MISCELLANEOUS

a. Some States have established procedures to accept out-of-specification material and pavement with a reduction in price. These procedures include definition of lot size/production time, tolerances, and pay factor reductions for ingredient materials, combined mixture properties, pavement density, pavement smoothness, and lift thickness.

b. Prior to the start of production and placement operations, a preplacement conference, including all the paving participants, should be held. This conference would define duties and responsibilities for each phase of the operation as well as problem solving procedures.

c. During start-up it is very effective to have a construction and/or materials specialist at the project site to assist in identifying and solving any problem that develops.

d. Because asphalt hot-mix pavement construction is complex, it requires that each person involved understand his/her function thoroughly. It is also helpful if each person has a basic understanding of each of the many phases involved. It is recommended that States develop or use existing training to address these phases of asphalt paving.

Ronald E. Heinz
Associate Administrator for
Engineering and Program Development

4 Attachments

AGGREGATE GRADATION

It has long been established that gradation of the aggregate is one of the factors that must be carefully considered in the design of asphalt paving mixtures, especially for heavy duty highways. The purpose in establishing and controlling aggregate gradation is to provide sufficient voids in the asphalt aggregate mixture to accommodate the proper asphalt film thickness on each particle and provide the design air void system to allow for thermal expansion of the asphalt within the mix. Minimum voids in the mineral aggregate (VMA) requirements have been established and vary with the top aggregate size.

Traditionally, gradation requirements are so broad that they permit the use of paving mixtures ranging from coarse to fine and to either low or high stability. To further complicate matters, different combinations of sieve sizes are specified to control specific grading ranges. Standardization of sieve sizes and aggregate gradations, which has often been suggested, is not likely to occur because of the practice of using locally available materials to the extent possible.

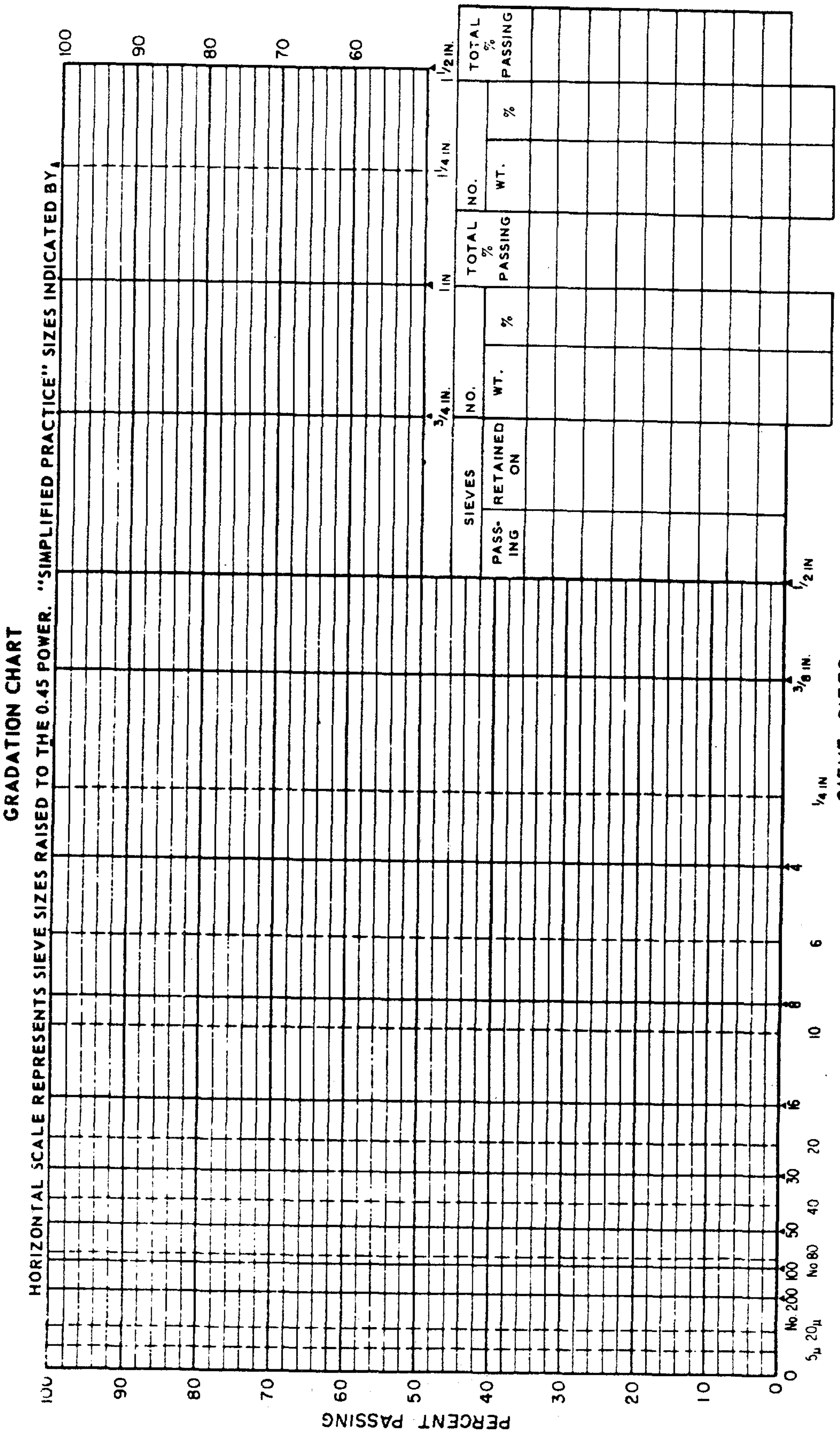
In the early 1960's, the Bureau of Public Roads introduced a gradation chart (Figure #1) which is especially useful in evaluating aggregate gradations. The chart uses a horizontal scale which represents sieve size openings in microns raised to the 0.45 power and a vertical scale in percent passing. The advantage in using this chart is that, for all practical purposes, all straight lines plotted from the lower left corner of the chart, upward and toward the right to any specific nominal maximum particle size, represent maximum density gradations. The nominal maximum particle sieve size is the largest sieve size listed in the applicable specification upon which any material is permitted to be retained. An example is shown in Figure #2.

The gradations depicted in Figure #3 and #4 are exaggerated to illustrate the points being made. By using the chart, aggregate gradations can be related to maximum density gradation and used to predict if the mixture will be fine or coarse textured as shown in Figure #3.

Soon after the chart was developed, it was used to study gradations of aggregate from several mixtures that had been reported as having unsatisfactory compaction characteristics. These mixtures could not be compacted in the normal manner because they were slow in developing sufficient stability to withstand the weight of the rolling equipment. Such mixtures can be called "tender mixes." This study identified a consistent gradation pattern in these mixes as is illustrated in Figure #4.

Most notable is the hump in the curve near the #40 sieve and the flat slope between the #40 sieve and the #8 sieve. This indicates a deficiency of material in the #40 to 8# sieve range and an excess of material passing the #40 sieve. Mixtures with an aggregate exhibiting this gradation characteristic are susceptible to being tender, particularly if the fines are composed of natural sand.

As part of the bituminous mix design process, the aggregate gradation should be plotted on the 0.45 power gradation chart.



GPO 882-415

Figure #1

0.45 Power Gradation Chart

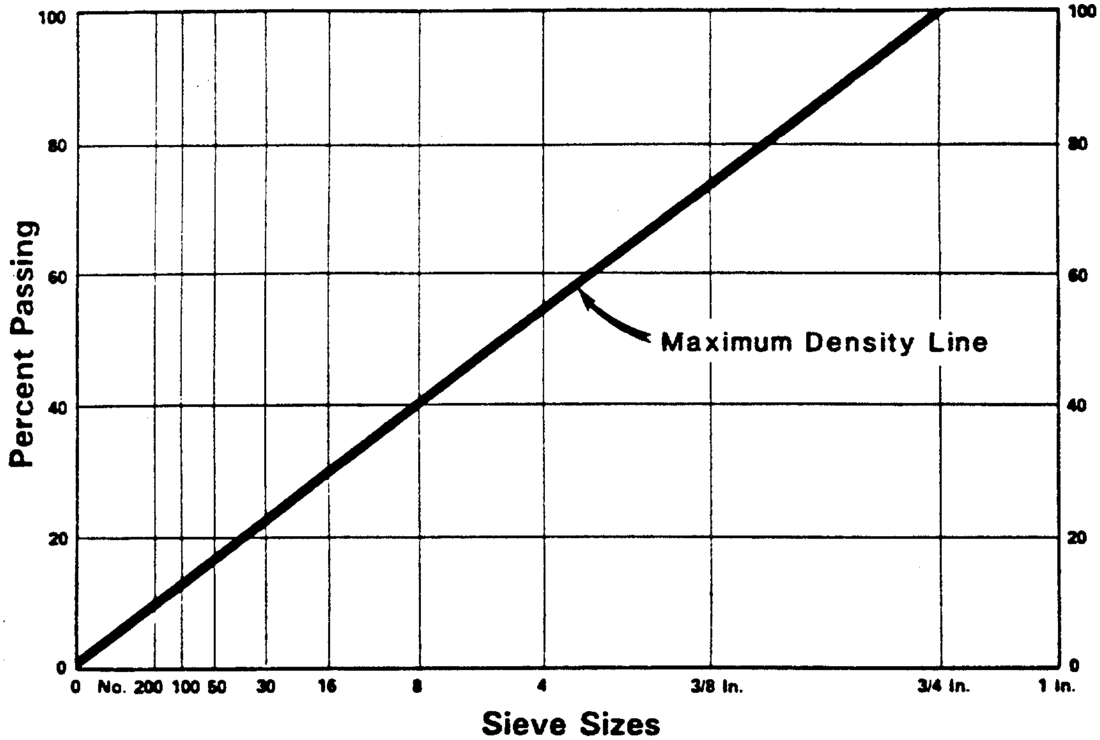


Figure #2

0.45 Power Gradation Chart

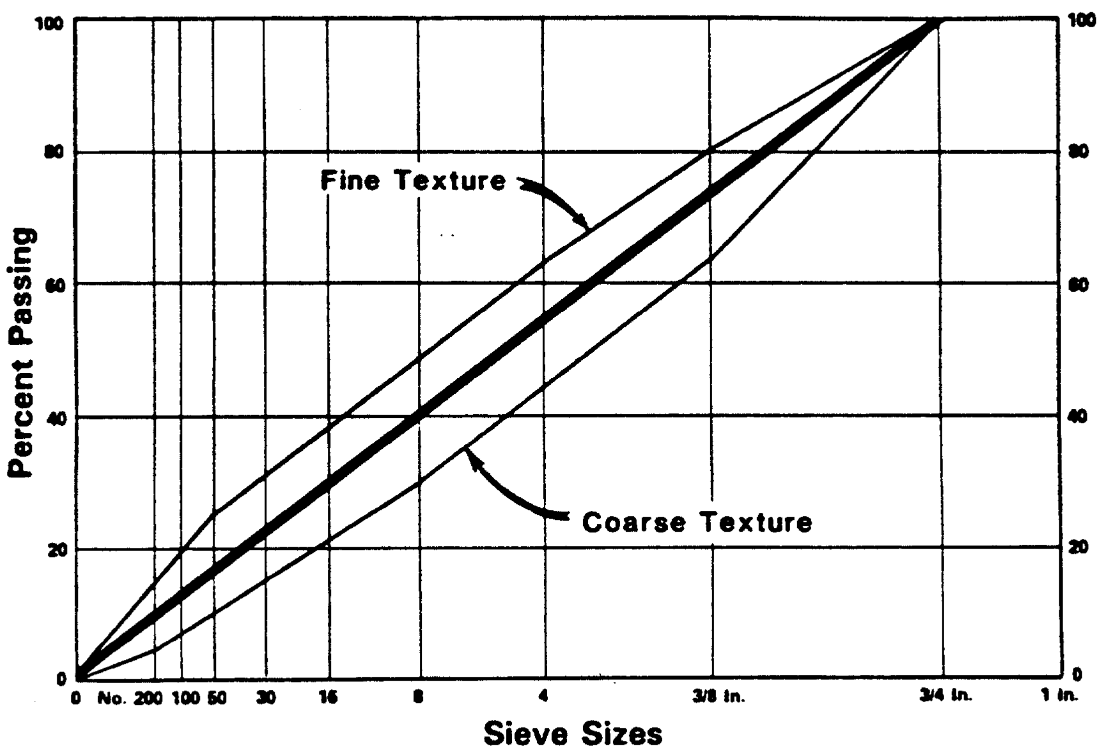


Figure #3

0.45 Power Gradation Chart

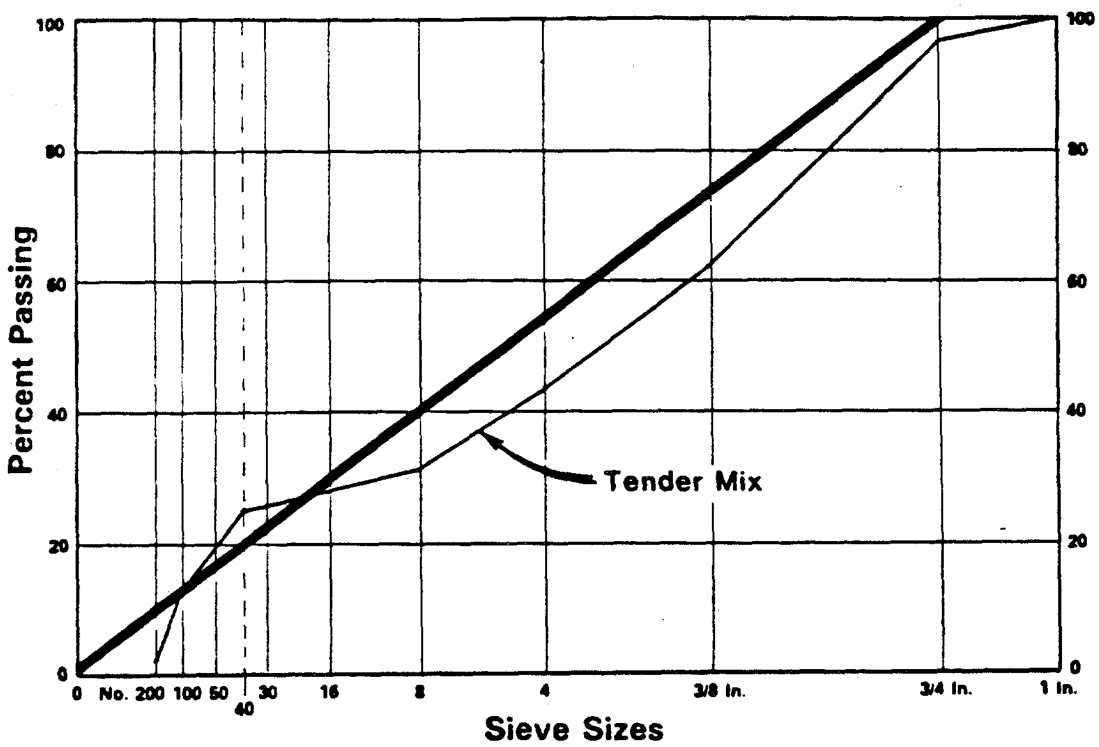


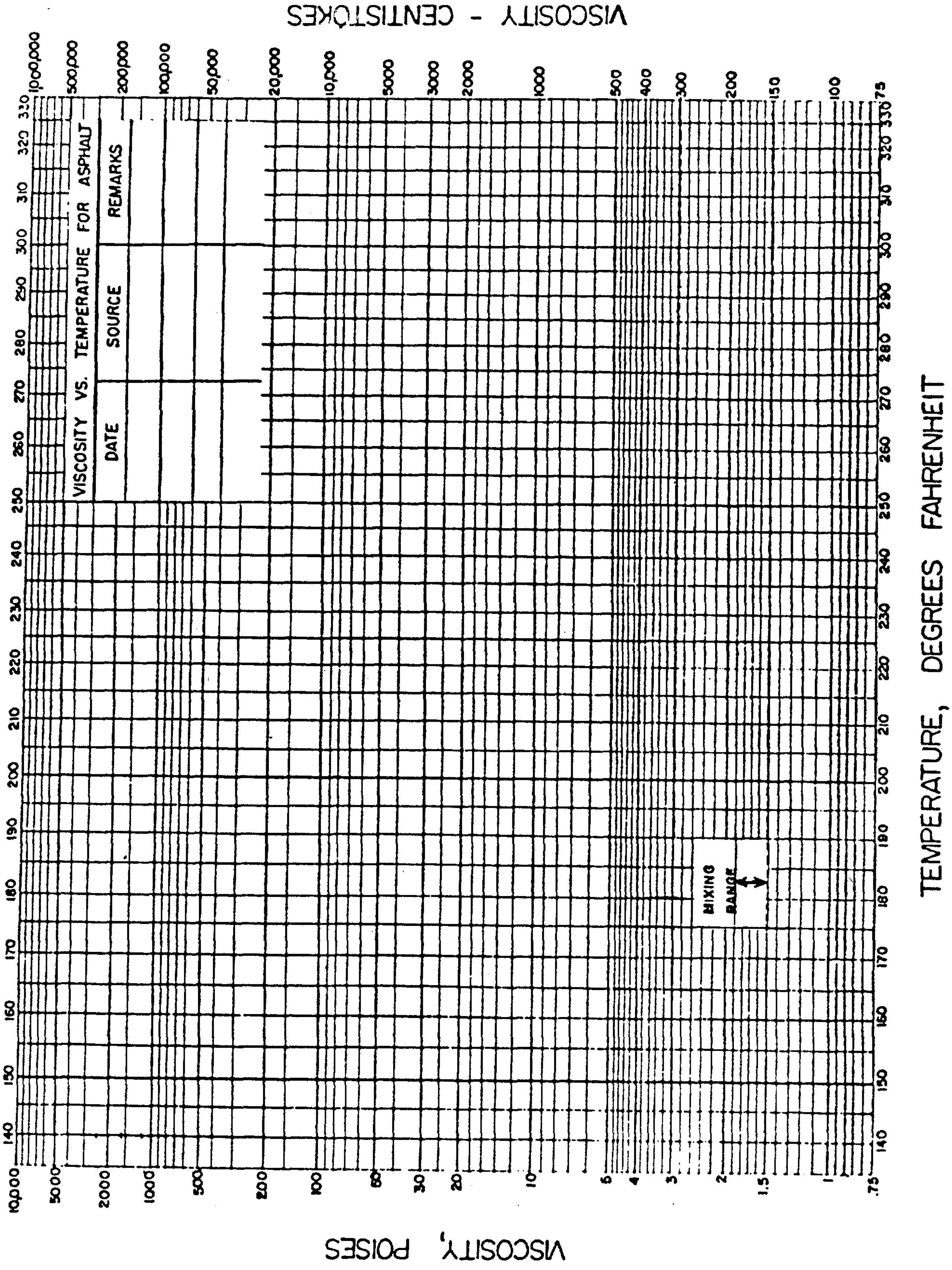
Figure #4

ASPHALT VISCOSITY

Each particular asphalt has a unique temperature-viscosity relationship. This relationship is sometimes described as temperature susceptibility. This temperature-viscosity relationship can be plotted on a modified semi-log chart as shown on the attached chart. These charts are very useful in determining the optimum mixing and compacting temperature of a particular asphalt. Past research has identified the optimum mixing temperature as that corresponding to a viscosity of 170 ± 20 centistokes, and the optimum compaction temperature as that corresponding to a viscosity of 280 ± 30 centistokes for laboratory mix design. The optimum mixing temperature should be identified for the asphalt used in the mix design and included in the mix design report which is sent to the production plant.

Prior to the oil embargo, there was a relatively fixed distribution system for crude oil. This allowed for a relatively uniform asphalt cement from each refinery. Highway agencies became familiar with the handling and performance characteristics of those asphalt cements. As a result of the embargo, a new variable distribution system is in place which allows shifting and blending of crude oils resulting in production of asphalt cements with very different temperature viscosity characteristics.

The attached chart will allow plotting the temperature-viscosity curve for the asphalts used in a State or a particular asphalt from a project. If the kinematic viscosity (275° F) of the asphalt being used changes from the kinematic viscosity of the asphalt used in the mix design by a factor of more than about two, a new mix design should be required.


VISCOSITY - CENTISTOKES
VISCOSITY VS. TEMPERATURE FOR ASPHALT
DATE
SOURCE
REMARKS
MIXING RANGE
TEMPERATURE, DEGREES FAHRENHEIT
VISCOSITY, POISES

7. Is conveyer system covered and insulated (if necessary) so as to prevent excessive loss of heat during transfer of material from mixing plant to storage bin?
8. Does storage bin have acceptable heating system?
9. Has surge or storage bin received prior evaluation and approval before using?

IX. Safety and Inspection Provisions

1. Are gears, pulleys, chains, sprockets, and other dangerous moving parts thoroughly protected?
2. Is an unobstructed and adequately guarded passage provided and maintained in and around the truck loading space for visual inspection purposes?
3. Does plant have adequate and safe stairways or guarded ladders to plant units such as mixer platforms, control platforms, hot storage bins, asphalt storage tanks, etc. where inspections are required?
4. Is an inspection platform provided with a safe stairway for sampling the asphalt mixture from loaded trucks?

X. Truck Scales

1. Are scales capable of weighing the entire vehicle at one time?
2. Do scales have digital printing recorder or automatic weight printer?
3. Have scales been checked and certified by a reputable scale company in the presence of an authorized representative of the highway department?
4. Date checked ________________ Agency Name ______________________
5. Is copy of certification available?
6. Remarks ____
____
____
____
____

XI. Transportation Equipment

1. Are truck bodies clean, tight, and in good condition?
2. Do trucks have covers to protect material from unfavorable weather conditions?
3. Is soapy water or other approved products available for coating truck bodies to prevent material from sticking? Diesel fuel should not be used.
4. Type of material used. _______________

XII. Provisions for Testing

1. Does size and location of laboratory comply with specifications?
2. Is laboratory properly equipped?
3. Is laboratory acceptable?

SPECIAL REQUIREMENTS FOR BATCH PLANTS

XIII. Weigh Box or Hopper

1. Is weigh box large enough to hold full batch?
2. Does gate close tightly so that material cannot leak into the mixer while a batch is being weighed?

XIV. Aggregate Scales

1. Are scales equipped with adjustable pointers or markers for marking the weight of each material to be weighed into the batch?
2. Are ten 50-lb. (22.7 kg) weights available for checking scales?
3. Has accuracy of weights been checked?
4. Have scales been checked and certified by a reputable scales company in the presence of an authorized representative of the highway department?
   Date checked ______________ Agency Name ____________________
   Is copy of certification available?
   Remarks ____
   ____
   ____
   ____
   ____
5. If the plant is equipped with beam type scales, are the scales equipped with a device to indicate at least the last 200 lb. (97 kg) of the required load?

XV. Asphalt Cement Bucket

1. Is bucket large enough to handle a batch in a single weighing so that the asphalt material will not overflow, splash or spill?
2. Is the bucket steamed, or oil-jacketed or equipped with properly insulated electric heating units?
3. Is the bucket equipped to deliver the asphalt material over the full length of the mixer?

XVI. Asphalt Cement Scales

1. Have scales been checked and certified by a reputable scale company in the presence of an authorized representative of the highway department?
   Date checked ________________ Agency Name ______________________
   Is copy of certification available?
   Remarks ____

   ____

2. Are scales equipped with a device to indicate at least the last 20 lb. (9.1 kg) of the approaching total load?

XVII. Screens

1. Condition of screens. Satisfactory _______ Unsatisfactory _______
2. Do the plant screens have adequate capacity and size range to properly separate all the aggregate into sizes required for proportioning so that they may be recombined consistently?

XVIII. Hot Bins

1. Number of bins? ________________
2. Are bins properly partitioned?
3. Are bins equipped with overflow pipes?
4. Will gates cut off quickly and completely?
5. Can samples be obtained from bins?
6. Are bins equipped with device to indicate the position of aggregate at the lower quarter point?

XIX. Asphalt Control

1. Are means provided for checking the quantity or rate of flow of asphalt material?
2. Time required to add asphalt material into pugmill.

XX. Mixer Unit for Batch Method

1. Is the plant equipped with an approved twin pugmill batch mixer that will produce a uniform mixture?
2. Can the mixer blades be adjusted to ensure proper and efficient mixing?
3. Are the mixer blades in satisfactory condition?
4. What is the clearance of the mixer blades? ______________ in.
5. Does the mixer gate close tight enough to prevent leakage?
6. Does the mixer discharge the mixture without appreciable segregation?
7. Is the mixer equipped with time lock?
8. Does timer lock the weigh box gate until the mixing cycle is completed?

9. Will timer control dry and wet mixing time?
10. Can timer be set in 5 second intervals throughout the designated mixing cycles?
11. Can timer be locked to prevent tampering?
12. Is a mechanical batch counter installed as part of the timing device?

XXI. Automation of Batching

1. If the plant is fully automated, is an automatic weighing, cycling and monitoring system installed as part of the batching equipment?
2. Is the automatic proportioning system capable of weighing the materials within ± 2 percent of the total sum of the batch sizes?

SPECIAL REQUIREMENT FOR DRUM MIXERS

XXII. Aggregate Delivery System

1. Number of cold feed bins? ________________
2. Are cold feed bins equipped with devices to indicate when the level of the aggregate in each bin is below the quarter point?
3. Does the cold feed have an automatic shut-off system that activates when any individual feeder is interrupted?
4. Are provisions available for conveniently sampling the full flow of material from each cold feed and the total cold feed?
5. Is the total feed weighed continuously?
6. Are there provisions for automatically correcting the wet aggregate weight to dry aggregate weight?
7. Is the flow of aggregate dry weight displayed digitally in appropriate units of weight and time and totaled?
8. Are means provided for diverting aggregate delivery into trucks, front-end loaders, or other containers for checking accuracy of aggregate delivery system?
9. Is plant equipped with a scalping screen for aggregate prior to entering the conveyor weigh belt?

XXIII. Asphalt Cement Delivery System

1. Are satisfactory means provided to introduce the proper amount of asphalt material into the mix?
2. Does the delivery system for metering the asphalt material prove accurate within ± 1 percent?
3. Does the asphalt material delivery interlock with aggregate weight control?
4. Is the asphalt material flow displayed in appropriate units of volume or weight and time and totaled?
5. Can the asphalt material be diverted into distributor trucks or other containers for checking accuracy of delivery systems?

XXIV. Drum Mixer

1. Is the drum mixer capable of drying and heating the aggregate to the moisture and temperature requirements set forth in the specifications, and capable of producing a uniform mix?
2. Does plant have provisions for diverting mixes at start-up and shutdowns or where mixing is not complete or uniform?

XXV. Is plant approved for use?
If not, explain what needs to be corrected. (Show Item Number)

PROJECT INSPECTION CHECKLIST

Compaction of Foundation

1. Have all courses of the foundation been compacted to required density?

Old Asphalt Pavement

1. Have all potholes been patched?
2. Have all necessary patches been made?
3. Have all loose material and "fat" patches been removed?
4. Have all depressions been filled and compacted?
5. Has fog seal been used on surface that has deteriorated from oxidation?
6. Has an emulsified asphalt slurry seal been applied on old surfaces with extensive cracking?

Rigid Type Pavement

1. Has pavement been under sealed where necessary?
2. Has premolded joint material and crack filler been cleaned out?
3. Have all "fat" patches been removed?
4. Has badly broken pavement been removed and patched?
5. Have all depressions been filled and compacted?

Incidental Tools

1. Do incidental tools comply with specifications? ________
2. Are all necessary tools on job before work begins?

The Engineer and the Contractor

1. Have the engineer and inspectors held a preliminary conference with the appropriate contractor personnel?
2. Has continuity of operations been planned?
3. Has the number of pavers to be used been determined?
4. Have the number and type of rollers to be used been determined?
5. Has the number of trucks to be used been determined?
6. Has the width of spread in successive layers been planned?
7. Is it understood who is to issue and who is to receive instructions?
8. Have weighing procedures and the number of load tickets to be prepared been determined?
9. Have procedures for investigation of mix been agreed upon?
10. Has method of handling traffic been established?

Preparation of Surface

1. Have all surfaces that will come into contact with the asphalt mix been cleaned and coated with asphalt?
2. Has a uniform tack coat of correct quantity been applied?

Asphalt Distributor

1. Does the asphalt distributor comply with specifications?
2. Are the heaters and pump in good working condition?
3. Have all gauges and measuring devices such as the bitumeter, tachometer, and measuring stick been calibrated?
4. Are spray bars and nozzles unclogged and set for proper application of asphalt?

Hauling Equipment

1. Are truck beds smooth and free from holds and depressions?
2. Do trucks comply with specifications?
3. Are trucks equipped with properly attached tarpaulins?
4. For cold weather or long hauls, are truck beds insulated?
5. When unloading, do trucks and paver operate together without interference?
6. Is the method of coating of contact surfaces of truck beds agreed upon?

Paver

1. Does the paver comply with specifications?
2. Is the governor on the engineer operating properly?
3. Are the slat feeders, the hopper gates, and spreader screws in good condition and adjustment?
4. Are the crawlers adjusted properly?
5. Do the pneumatic tires contain correct and uniform air pressure?
6. Is the screed heater working properly?
7. Are the tamper bars free of excessive wear?
8. Are the tamper bars correctly adjusted for stroke?
9. Are the tamper bars correctly adjusted for clearance between the back of the bar and the nose of the screed plate?
10. Are the surfaces of the screed plates true and in good condition?
11. Are mat thickness and crown controls in good condition and adjustment?
12. Are screed vibrators in good condition and adjustment?
13. Is the oscillating screed in proper position with respect to the vibrating compactor?
14. Is the automatic screed control in adjustment and is the correct sensor attached.

Spreading

1. Are the required number of pavers on job?
2. Is the mix of uniform texture?
3. Is the general appearance of the mix satisfactory?
4. Is the temperature of the mix uniform and satisfactory?
5. Does the mix satisfy the spreading requirements?
6. Has proper paver speed been determined?
7. Is the surface smoothness tolerance being checked and adhered to?
8. Is the depth of spread checked frequently?
9. Has the daily spread been checked?

Rolling

1. Are the required number of rollers on the job?
2. Is proper rolling procedure being followed?
3. Is the proper rolling pattern being followed?
4. Are joints and edges being rolled properly?

Miscellaneous

1. Are all surface irregularities being properly corrected?
2. Is efficient control of traffic being maintained?
3. Are sufficient samples being taken?
4. Are samples representative?
5. Have assistant inspectors been properly instructed?
6. Are inspection duties properly apportioned among assistants?
7. Are records complete and up-to-date?
8. Are safety measures being observed?
9. Has final clean-up and inspection been made?

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Washington, D.C. 20590

Subject: Prevention of Premature Distress In Asphalt Concrete Pavements

Date APR 18 1988

From: Chief, Pavement Division

Reply to Attn. of: HHO-12

To: Regional Federal Highway Administrators
Director Federal Program Administrator

Attached for your use are two copies of a technical paper on the prevention of premature distress in asphalt concrete pavements. The paper is an implementing action to one of the recommendations of the Ad Hoc Task Force report on "Asphalt Pavement Rutting and Stripping." The paper is intended as a companion document to Technical Advisory 5040.27. Please furnish copies to the engineers in the divisions who are responsible for implementing the recommendations of the TA. The paper may also be of use to area engineers as it provides additional background information on some points in the TA and responds to some of the questions that have been asked.

We plan to issue a paper this summer on investigating and rehabilitating rutted and stripped pavements. If you are aware of a good practice in your region on this subject, we would appreciate receiving a short writeup on it.

We appreciate the efforts of the regions and division that have offered comments on this paper. If you have any question concerning this paper or suggestions for the second one, please contact Mr. Peter Kleskovic at FTS 366-2216.

Norman J. Van Ness

TECHNICAL PAPER 88-02 - Prevention of Premature Distress in Asphalt Concrete Pavements

Asphalt concrete rutting, a channelized depression in the wheel paths, is the result of deformations either in the pavement or the subgrade. It occurs because of consolidation in the subgrade or in the pavement structure, or because of shear failure in the mix. Rutting can cause vehicle handling problems and it increases the potential for hydroplaning.

Stripping is a form of moisture damage that can severely weaken an asphalt pavement. It can take a number of forms. Water can get between the asphalt and the aggregate and break the bond between them. Water can also get between and separate the coated particles in the mix. Still another form occurs when the asphalt cement emulsifies in water. High voids or segregation can accelerate stripping by letting water into the mat. Stripping can rob a pavement of a significant portion of its life. It also contributes to early rutting because of strength loss in the stripped layer.

Rutting and stripping are being experienced throughout much of the United States. In 1987, an FHWA Ad Hoc Task Force issued a report entitled, "Asphalt Pavement Rutting and Stripping." It discussed the national and regional nature of the problems, identified causes, and made short- and long-term recommendations to correct them. One of these called for updating Technical Advisory 5040.24 to reflect current knowledge. In response to this recommendation, a new Technical Advisory 5040.27, Asphalt Mix Design and Field Control was issued on March 10, 1988.

Increased traffic loads and tire pressures have been identified as accelerating factors in the occurrence of these distresses. However, upgrading mix design procedures and construction practices to the state-of-the-art can help to mitigate the problem. Even though some States and regions may not have rutting or stripping problems now, there is probably room for improvement in every State's procedures which could add extra years of life to their pavements.

Materials Considerations

The use of locally available aggregates is an important economic factor in highway construction. Asphalt concrete is typically made using local aggregates. However, in the case of marginal materials, the economic benefit may be at the expense of durability. The use of good quality aggregates plays a major role in preventing rutting and stripping. The recommendations in Technical Advisory (TA) 5040.27 should all be considered when evaluating an aggregate.

One way to make a mix more resistant to rutting is to increase the interparticle friction between the aggregates by increasing their angularity and roughness. This is accomplished by using crushed aggregates and by limiting the amount of natural sands. The TA recommends that specifications require at least 60 percent of the plus #4 sieve aggregates to have at least two mechanically induced fractured faces. Mechanically induced fractured faces are usually rougher than flat faces made by natural processes. This rougher texture helps to keep aggregate particles from slipping by each other and also provides more bonding area for the asphalt.

The TA recommends limiting natural sands to 15 to 20 percent of the total weight of the aggregates for high volume roads and 20 to 25 percent for medium and low volume roads. Since natural sands tend to be round and smooth, they generally do not have good frictional properties. In addition, natural sand deposits may contain clay or organic particles. Asphalt does not adhere well to clay coated particles. Clay can also have an emulsifying effect on the asphalt in the presence of water. Both of these effects can accelerate the occurrence of stripping.

Some natural sands also consist of largely one sized particles. This can produce a gap graded mix, which is more likely to segregate. The gap grading may result in a flat slope on the 0.45 power gradation curve between the #4 and #30 sieves. If this flat slope results in more than a 3 percent hump above the maximum density line, the mix might be tender, i.e., too unstable to compact at proper temperatures.

The gradation of the aggregates is an important factor in the construction and the performance of the mat. Aggregate gradation can affect the strength and the durability of the mat, as well as its resistance to stripping or deformation. Certain gradations are more prone to segregation. For example, the larger the aggregate top size, the more care is required to prevent segregation. Gap graded aggregates are also likely to segregate due to the absence of intermediate sizes which help to stop the larger aggregate from rolling.

Asphalts can be graded by penetration, by the viscosity of the original asphalt (AC Grading) and by the viscosity of the aged residue (AR Grading). The AR-graded asphalts may have problems when used in drum mixer plants, since the asphalt may not harden as much as in a batch type plant. This could lead to rutting, since the asphalt in the mat would be softer than was assumed in the design, making the mix less stable.

AASHTO Specification M-226 sets requirements for AC and AR-graded asphalts. Tables 1 and 2 of this specification set absolute viscosity ranges at 140^{o} F and a minimum kinematic viscosity at 275^{o} F for AC-graded asphalts. It does not however set a maximum value for kinematic viscosity. Therefore, it is possible for two asphalts to meet the requirements of M-226 and yet have substantially different kinematic viscosities at mixing temperatures. To control this variable, attachment 2 of the TA recommends that a new mix design be performed if the kinematic viscosity of the asphalt being used at the plant changes by more than a factor of 2 from the asphalt used in the mix design.

The amount of asphalt aging occurring in the mixing process must be controlled, since highly aged asphalts will be more brittle and likely to crack. The aging process is simulated by the Thin Film Oven Test (TFOT). The viscosity of an AC-graded asphalt subjected to the TFOT should not exceed four times the nominal viscosity of the grade. For example, an AC 20 has a viscosity range of 1600 to 2400 poises. After the TFOT, its viscosity should not exceed 8000 poises.

A new feature of the TA is the low temperature ductility test. The test gives an indication of the mix's potential for low temperature cracking. The test is based on AASHTO T51, but is performed at a lower temperature of 39.2^{0} F and a slower pull speed of 1 cm per minute.

Mix Design Considerations

The purpose of a mix design procedure is to determine for a given gradation and quality of aggregates the optimum asphalt content and grade which will meet specification requirements for stability, voids, VMA, flow (Marshall), swell (Hveem), and will be close to a maximum unit weight. The selection of the optimum asphalt content is a balancing operation among these factors.

It is essential that mixes be designed using the actual project materials since certain interactions occur between aggregates, asphalts, and additives. Changing one of the ingredients may set up a completely different interaction.

Relating the density of a laboratory compacted specimen to the pavement's ultimate density under traffic is a major element in designing mixes. Asphalt mixes should be designed for 3 to 5 percent air voids, which is the desired voids level of the mat after several years of traffic. The goal is to build a pavement that is dense enough to minimize air and water intrusion but has enough voids for the asphalt to expand.

To try to simulate the effects of traffic, various levels of compactive effort are used to make laboratory design specimens. In the Marshall

Method, specimens are subjected to different numbers of hammer blows depending on the design ESALs. Specimens should be compacted with 35 blows per side for pavements that will have less than 10,000 ESALs, 50 blows for ESALs between 10,000 and 1,000,000, and 75 blows for ESALs above 1,000,000. It is important to note that specimens compacted with mechanical hammers generally have lower densities than those compacted with manual hammers. However, AASHTO allows the use of the mechanical hammer provided it has been calibrated to give results comparable to the manual hammer.

Problems may result if lab specimens have been over-compacted or under-compacted in relation to expected traffic. For a light traffic highway, a 75 blow design could cause problems because the mat will probably not reach design density under traffic and will have excessive voids, making the mix susceptible to rapid asphalt oxidation and water damage. Similarly, a heavy traffic pavement designed with only 50 blows, might be densified by traffic to the point that the mat would be deficient in voids. The mat may not have enough void spaces to permit for thermal expansion of the asphalt and the pavement would be prone to bleeding, stability loss, and rutting.

The TA provides stability values for the three ESAL ranges. Although stability is desirable, excessively high stability values may be achieved at the expense of durability. For example, to an extent stability can be increased by reducing the asphalt content. However, reducing the asphalt content too much results in thin asphalt films on the aggregate which makes the mix more prone to stripping and to age hardening.

In designing mixes, the air voids content should be based on the maximum theoretical density of the mix as determined by AASHTO T209, the Rice Test. The Rice Test is the best available procedure for determining the maximum theoretical density because of the short-test time and because it accounts for the asphalt absorbed by the aggregate. It also eliminates the error that can occur by trying to calculate a maximum specific gravity based on the percentages and specific gravities of the component aggregates.

In the past, base and binder mixes have been constructed with higher air void levels and somewhat lower asphalt contents. Typical values have been as high as 8 to 11 percent. These high void mixes are more likely to strip, because the mat is open to moisture penetration. Additionally, the aggregates typically have thinner asphalt films, which lowers mix strength and makes the mix more likely to strip. To prevent problems, bases and binder mixes should also be designed for 3 to 5 percent air voids.

The Voids in Mineral Aggregate (VMA) is a measure of the space between the aggregate particles which can accommodate the asphalt cement and the air voids. If the VMA is low, the mix will have either a low voids or asphalt content, because there is not enough empty space in the mix to accommodate both. If the VMA is too large, then too much asphalt cement will be required to achieve density and stability will be lowered. Minimum VMA values are included in the TA. These values are based on the nominal maximum aggregate size, i.e., the largest sieve on which some material can be retained.

It is recommended that the ratio of dust or minus #200 material to asphalt be kept between 0.6 and 1.2 based upon weight. For a mix containing 5 percent asphalt, the dust content should be held between 3 and 6 percent, using washed gradations. The effect of dust on the mix is complex and depends on the size, shape, gradation, and quantity of the dust. A certain amount of dust is needed to produce a dense cohesive mix. However, in some cases too much dust can result in low air voids and can stiffen the binder, resulting in a harsh mix. Fine dust can act as an asphalt extender and cause the mix to be tender. Single sized particles may increase the asphalt demand of the mix.

Evaluation of Stripping Potential

A mix design is not complete until an evaluation has been made of the mix's resistance to moisture damage. Moisture damage susceptibility can be estimated by strength loss after moisture conditioning by a number of test procedures. Probably, the most promising procedure is the Root-Tunnicliff test. It is faster than the Lottman test (AASHTO T283) and is adaptable to field control. Also, the samples are more severely conditioned than in the Immersion Compression test (AASHTO T165).

Asphalt concrete mix design is complex, therefore, it is also important to identify the reason for the loss of strength in the stripping test. The worst scenario is to use an anti-stripping additive when the problem may be due to a low asphalt content caused by a low VMA. In this case, it would be better to change the gradation to increase the VMA so that more asphalt could be put into the mix.

Once a set of materials is found to be susceptible to water damage, an additive should be selected. The type, dosage rate and method of application should be determined through laboratory testing, preferably using the Lottman or the Root-Tunnicliff Tests. Hydrated lime and portland cement have been found to be very effective in protecting mixtures from water damage. Some liquid anti-stripping additives have also been effective.

Agencies should guard against the use of any additive without adequate testing and evaluation. Some States have developed approved lists of anti-stripping additives. This is an acceptable practice provided the proposed additives are then tested with the actual project asphalt and aggregates. Reliance solely on the approved list may result in the use of additives which have no effect or which may even be detrimental to the mix.

Plant Operations

The TA covers a number of specific points necessary to ensure proper plant operations. Attachment 3 of the TA is a model checklist for inspecting asphalt plants. A few points should be stressed. Agencies using AR-graded asphalt should verify that the asphalt hardens sufficiently during mixing. This is accomplished by recovering the extracted asphalt and testing it for viscosity. This testing can also give a hint if unburned fuel oil is contaminating the asphalt.

The dust/asphalt ratio should be monitored during production. The dust/asphalt ratio has become increasingly important since the advent of baghouse dust collectors. Typically, the contractor wants to recirculate all of the captured dust into the mix. However, it is difficult to properly compensate for the extra dust during mix design, since it is difficult to meter a constant flow of dust into the actual production mix. High dust contents make the mix more sensitive to asphalt content. Depending on the size of the dust particles, the mix may have either a lower or higher asphalt demand. If the flow of dust varies during production, the mix will alternate between having too much or too little asphalt.

The asphalt plant testing program should include verification testing of the mix design. This should occur after plant start-up and periodically during production. The purpose is to verify that the production mix has the same properties as the mix that was designed. As well as being tested for asphalt content and gradation, the plant produced mix should also be run through the standard design tests such as stability, voids, density, etc. Plant produced mix may not have the same properties as specimens compacted in the lab even though they were made from the same ingredients since some aggregates produce additional mineral dust when processed through the plant. Failure to do verification testing can result in a mix being produced which has different gradations, voids, stabilities, and densities from the laboratory mix. If this happens, the benefits of doing a mix design are lost.

Laydown Operations and Compaction

Paving operations should start with the construction of a test strip. A nuclear gauge should be used to determine the number of passes needed to obtain optimum density. Cores should then be taken from the test strip to determine the bulk density, which is the true density of the mat. A correction for the nuclear gauge due to aggregate type is obtained by comparing the nuclear density values to the bulk density values. To determine if the rolling is providing the proper level of compaction and voids, the bulk density of the cores is compared to the maximum theoretical density of the mix, which should be obtained from the Rice Test.

Density specifications can be based on meeting percentages of maximum theoretical density, of lab density or of test strip density. Whichever specification is used, acceptance levels should be set so that after rolling, the mat will have air voids of 6 to 8 percent, or a density of 92 to 94 percent of maximum theoretical density.

Not enough can be said with regard to the detrimental effects of paving in cold or wet weather. Adverse ambient conditions can result in inadequate compaction, poor bonding between layers, or moisture being trapped in the pavement. Raising the temperature of the mix to compensate for cold weather conditions may age the asphalt cement excessively. Also, hot asphalt cement flows more readily and is more easily absorbed by the aggregates. The hot asphalt cement will tend to drain to the bottom of the storage bins or the truck beds resulting in fat spots in the mat corresponding to each truck load. Also, if absorption becomes significant, the mix will have a low effective asphalt content.

Paving on a cold surface can cause the temperature of the mix to drop below minimum before rolling is completed. AASHTO has recently adopted into its Guide Specifications the following suggested minimum surface temperature values for different lift thicknesses.

Compaction Thickness	Surface Course	Sub-surface Course
< 1 1/2 in.	60^{o} F suggested	55^{o} F suggested
1 1/2 to 2 1/2 in.	50^{o} F suggested	45^{o} F suggested
2 1/2 in. +	40^{o} F suggested	35^{o} F suggested

All rolling, including the finish rolling, should be completed before the mat temperature drops below 175^{o} F. The 175^{o} F should be considered an absolute minimum value, since continued rolling with a steel wheel roller can cause the mat to decompact. Compaction of the mat should begin as soon as possible after laydown.

The TA strongly encourages the use of pneumatic rollers because of their ability to knead and seal the surface of the mat. There is also some indication that they provide a more uniform level of compaction across the mat and provide better compaction at joints.

Again attention is needed during paving to make sure that a uniform mat has been produced and that no segregation is occurring in the paving operation. A number of paving practices can result in segregation such as uneven paver demand, emptying the hopper between loads, or in the loading of the paver hopper.

Conclusion

There will always be a need for additional research and new products. One area needing study, is how to relate mixture properties to pavement design procedures. Work is ongoing through SHRP to evaluate tests methods for asphalt cement and asphalt/aggregate mixtures and to monitor the long term performance of pavements. In NCHRP project 9-6(1), the Development of Asphalt-Aggregate Mixtures Analysis System, work is ongoing to develop better mix design procedures.

However, much of the current rutting and stripping problem could be corrected by using state-of-the-art knowledge in mix design and construction. This effort requires the commitment of the highway community to designing and producing high quality pavements.

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: Guidelines on the Use of Bag-House Fines

Date: APR - 7 1988

From: Chief, Pavement Division

Reply to Attn. of: HHO-12

To: Regional Federal Highway Administrators
Direct Federal Program Administrator

Attached is one copy of a recent National Asphalt Pavement Association (NAPA) publication on the above subject. It is the most comprehensive and informative study on this subject to date. The NAPA has made a distribution of this report to its member companies.

You will note on page 21, Summary and Conclusions, that the author recommends a maximum dust asphalt ratio of 1.2 to 1.5 by weight in Item 1. This is not completely in concert with Item 4.a (8) of the March 10, 1988, Technical Advisory T 5040.27, which recommends a maximum dust asphalt ratio of 1.2. We have no quarrel with the authors recommendations provided that his other recommendations and particularly number 3 can be, and is, strictly adhered to at the mixing plants.

We suggest that this report be given to your pavement or materials specialist.

Norman J. Van Ness

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: Transmittal of Report FHWA-SA-92-022

Date: JUN 9 1992

From: Director, Office of Engineering
Director, Office of Technology Applications

Reply to Attn of: HNG-42

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Attached are copies of the *State of the Practice on the Design and Construction of Asphalt Paving Materials with Crumb Rubber Modifier* (Publication No. FHWA-SA-92-022). Sufficient copies are provided for distribution of two copies for the region office, one copy to each division office, and four copies to each State department of transportation, or as you see as appropriate.

The passage of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) has generated interest in the highway community on the subject of using scrap tire rubber in asphalt paving. The State DOT's are all aware of the provisions of Section 1038(d) which will require them to satisfy the minimum utilization of crumb rubber modifier (CRM) beginning in 1994.

The Pavement Division, with the cooperation of the Office of Technology Applications, has completed this report on the design and construction of asphalt paving materials with CRM. We believe this document will assist the State DOT's to develop an understanding of the technologies associated with CRM. This understanding is essential to form educated decisions on the implementation of ISTEA Section 1038(d).

The FHWA distribution of this document will be limited to public agencies and highway industry associations. Direct distribution is being made to the Technology Transfer Centers. Further distribution to individuals and private companies will be provided through the National Technical Information Service at (703) 487-4650.

Ray B. Griffith
for Douglas A. Bernard

Thomas O. Willett

Attachments

Memorandum

U.S. Department of Transportation

Federal Highway Administration

Subject: Asphalt Concrete Mix Design and Field Control TA 5040.27

Date: JUN 2 ! 1990

From: Chief, Pavement Division
Washington, D.C. 20590-0001

Reply to Attn of: HHO-10
HHO-30

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Attention: Regional Pavement and Materials Engineers

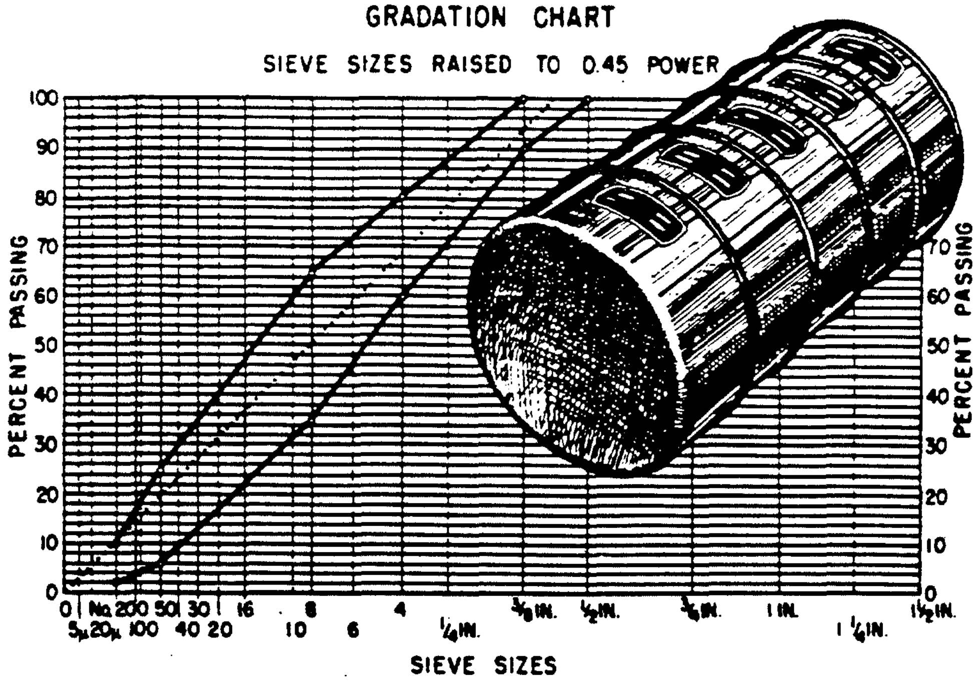
The subject Technical Advisory (TA) was issued in February 1988. The Section 6, Plant Operation, on page 8 offers suggestions and recommendations on items of importance at the mixture production plants. Item 6.b. states, "To avoid or mitigate unburned fuel oil contamination of the asphalt mixture, the use of propane, butane, natural gas, coal, or No. 1 or No. 2 fuel oils is recommended." The processed used-oil and heavy fuels oils were not included. The reason we did not include processed used-oil was that in the early to mid 1980's, there were some very shoddy practices by some used-oil processors which resulted in both combustion and environmental problems. In 1989, the Environmental Protection Agency (EPA) codified regulations on the maximum levels of metallic contents of processed used-oil. To comply, the used-oil must be treated and those processors who can do so successfully are licensed by EPA. These processed used-oils should be satisfactory for use in hot mix asphalt production.

With regard to heavy fuels, they were omitted from those recommended in the TA since they require preheating to lower their viscosities to the point where atomization and full combustion can be attained. Equipment is available to achieve full combustion and if properly used in the asphalt mixture production process, the heavy fuel oils can give satisfactory results.

In summary, it is not our intent to preclude the use of fuels that do not contaminate the mix. Regardless of the type of fuel used, our concern is that the hot mixed asphalt concrete is not damaged in the production process. The only way, that we are aware of, to assure ourselves of this is to follow the suggestion contained in 6.c. of the TA which states, "If the asphalt cement is overheated or otherwise aged excessively, the viscosity of the recovered asphalt will exceed that of the original asphalt by more than four times. However, if the viscosity of the recovered asphalt is less or even equal to the original viscosity, it has probably been contaminated with unburned fuel oil."

Louis M. Papet

AGGREGATE GRADATION FOR HIGHWAYS



Simplification, Standardization, and Uniform Application

and

A New Graphical Evaluation Chart

U.S. DEPARTMENT OF COMMERCE

BUREAU OF PUBLIC ROADS

WASHINGTON : 1962

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AGGREGATE GRADATION FOR HIGHWAYS

Aggregate Gradation: Simplification, Standardization, and Uniform Application

and

A New Graphical Chart for Evaluating Aggregate Gradation

By the Bureau of Public Roads

U.S. DEPARTMENT OF COMMERCE

Luther H. Hodges, *Secretary*

BUREAU OF PUBLIC ROADS

Rex M. Whitton, *Administrator*

United States Government Printing Office, Washington, D.C. : May 1962

For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C. - - - - - - - - - - - - - - Price 25 cents

AGGREGATE GRADATION: SIMPLIFICATION, STANDARDIZATION, AND UNIFORM APPLICATION

BY THE BUREAU OF PUBLIC ROADS

This report was prepared by a special committee appointed by Assistant Federal Highway Administrator and Chief Engineer Francis C. Turner and representing the Bureau of Public Roads Offices of Engineering, Operations, and Research. The committee included Ardery R. Rankin, chairman, Office of the Assistant Administrator: Carl A. Carpenter and Russell H. Brink, Physical Research Division: Morley B. Christensen, Construction and Maintenance Division; and William B. Huffine and Norman J. Cohen, Equipment and Methods Division

The Need for Simplification

Because of the magnitude of the nationwide highway construction program and the enormous amount of public funds required to finance it, every effort must be made to develop and apply ways and means of reducing construction costs while at the same time assuring the production of only high quality work. In its continuing mission of contributing toward the accomplishment of that objective, the Bureau of Public Roads has made a study of the possibility of effecting economies through simplification, standardization, and uniform application of aggregate gradations.

In performing this study, analyses were made of the current standard specifications of the highway departments of the 50 States, the Commonwealth of Puerto Rico, and the District of Columbia. The analyses disclosed a wide diversity in the requirements pertaining to aggregate gradations. Some 215 dissimilar gradations are specified for coarse aggregates for portland cement concrete. Of these gradations 88 are for both structures and pavement, 91 are for structures only, and 36 are solely for pavements. In contrast, Part I of the *Standard Specifications for Highway Materials* of the American Association of State Highway Officials includes only 19 gradations of coarse aggregates for all highway construction (see AASHO Designation M 43–49), with only 7 designed for use in concrete pavements or bases, bridges, and incidental structures (see AASHO Designation M 80–51). Similarly, the 52 highway departments specify a total of 58 fine aggregate gradations for both pavement and structural concrete whereas AASHO specifies only 1 (see AASHO Designation M 6–51).

In addition, there is considerable lack of consistency among the States in the number and sizes of sieves used to determine the gradations; furthermore, there is no uniform method in actual use by the States for designating aggregate gradation sizes. Only two States refer to the size designations used in AASHO Designation M 80–51. Some States have their own systems of size designations and other States use no designations at all.

Obviously, a greater degree of simplicity, standardization, and uniformity of usage for aggregate gradations would be highly desirable. For example, a commercial supplier who presently furnishes aggregates under numerous varying specification requirements for several Federal, State, county, and municipal highway organizations for identical construction purposes, would certainly find it much simpler and less costly if the same few gradations with identical specification requirements were used by all these agencies. Similarly, construction contractors bidding in more than one jurisdiction could prepare their bids much more intelligently and probably at lower prices if the specification requirements and the materials designations were the same for all jurisdictions.

For reasons of economy and because of the growing scarcity of high-quality aggregates in some areas, it is essential to make as much use as possible of aggregates that are locally available. This frequently necessitates tailoring the specification requirements to fit the characteristics of such local aggregates to whatever extent may be compatible with producing high-quality construction at economical prices. Nevertheless, a much greater degree of standardization and uniform use of aggregate gradations can undoubtedly be achieved. The problem has long been recognized and has here been approached with three specific objectives:

1. To develop a minimum number of standard aggregate gradations that can be uniformly adopted nationwide for general usage, while at the same time recognizing the need for some variations by special provisions to fit locally available materials.
2. To achieve uniformity in the number and sizes of sieves to be used in specifying the aggregate gradations.
3. To develop and adopt a simple and uniform system for identification of the standard aggregate gradations.

The Simplified Practice Recommendation

A major step toward accomplishing these objectives was taken on June 30, 1948, when the Department of Commerce approved and issued Simplified Practice Recommendations R 163–48[1] for coarse aggregates, including crushed stone, gravel, and slag. A predecessor recommendation had originally been approved for promulgation in June 1936 and issued as R 163–36. It was proposed by the Joint Technical Committee of the Mineral Aggregates Association, composed of representatives of the National Sand and Gravel Association, the National Crushed Stone Association, and the National Slag Association. Producers, distributors, and users of mineral aggregate all cooperated in developing the simplified practice recommendation. An intermediate revision was approved and published in 1939 and some additional revisions subsequent to 1939 resulted in the publication of the current issue of 1948. Table 1 shows the SPR gradings that are currently in effect.

As will shortly be described, the SPR system has been essentially adopted by both the American Association of State Highway Officials and the American Society for Testing and Materials.

Value of the SPR system

The simplified practice recommendation R 163–48 embodies a number of highly logical and useful features:

1. *Standard sieves.*—The SPR gradings employ a simple and convenient, square-opening, sieve-size series based primarily on the logarithmic principle.

[1] *Coarse Aggregates (Crushed Stone, Gravel, and Slag), Simplified Practice Recommendation R 163–48*, approved June 30, 1948, National Bureau of Standards, U.S. Department of Commerce, 1948.

The basic logarithmic sieve series employed begins with a sieve having clear openings of 3 inches and each smaller sieve has clear openings the diameter of which is one-half that of the next larger one. Thus the basic series is 3-inch, 1½-inch, ¾-inch, ⅜-inch, No. 4, No. 8, No. 16, No. 30, No. 50, No. 100, and No. 200. Because some consumer interests consider that the logarithmic series does not provide enough control in the larger sizes while others desire greater freedom in selecting maximum sizes, the gaps have been reduced in the SPR series by superimposing upon the logarithmic series, the arbitrary sizes 4-inch, 3½-inch, 2½-inch, 2-inch, 1-inch, and ½-inch. Also, two of the logarithmic sizes were left out of the SPR series—the No. 30 because it was felt that it serves no useful purpose in grading control of coarse commercial aggregates, and the No. 200 because material of this size (soil fines and commercial mineral filler for bituminous paving mixtures) is not and should not be considered an ingredient of commercial coarse aggregates. Both the No. 30 and the No. 200 sieves are required in specifying sands and fillers, as in the ASTM and AASHO standards, and both fit in the logarithmic series.

2. *Simple system.*—The SPR gradings embody a simple and readily understandable system of individual size and grading designations consisting basically of single-digit numbers.

The single-digit numbering series starts with No. 1 for the standard commercial aggregate having the largest top-size particles and progresses from No. 1 through No. 9 as the individual standard coarse aggregates decrease in size, as shown in table 2.

Because of consistent demands for certain longer gradings than the relatively short ones represented by the basic series, shown in the first column of table 2, a secondary

Table 1.—Sizes of coarse aggregate (crushed stone, gravel, and slag) from Simplified Practice Recommendation, R 163–48[1]

SPR size number	Nominal size,[2] square openings	Amounts finer than each laboratory sieve (square openings), percentage by weight														
		4-in.	3½-in.	3-in.	2½-in.	2-in.	1½-in.	1-in.	¾-in.	½-in.	⅜-in.	No. 4	No. 8	No. 16	No. 50	No. 100
1	3½-1½	100	90-100		25-60		0-15		0-5							
1F[3]	3½-2	100	90-100			0-10	0-5									
2F[3]	3-1½		100	90-100			0-10	0-2								
2	2½-1½			100	90-100	35-70	0-15		0-5							
24	2½-¾			100	90-100		25-60		0-10	0-5						
3	2-1				100	90-100	35-70	0-15		0-5						
357	2-No. 4				100	95-100		35-70		10-30		0-5				
4	1½-¾					100	90-100	20-55	0-15		0-5					
467	1½-No. 4					100	95-100		35-70		10-30	0-5				
5	1-½						100	90-100	20-55	0-10	0-5					
56	1-⅜						100	90-100	40-75	15-35	0-15	0-5				
57	1-No. 4						100	95-100		25-60		0-10	0-5			
6	¾-⅜							100	90-100	20-55	0-15	0-5				
67	¾-No. 4							100	90-100		20-55	0-10	0-5			
68	¾-No. 8							100	90-100		30-65	5-25	0-10	0-5		
7	½-No. 4								100	90-100	40-70	0-15	0-5			
78	½-No. 8								100	90-100	40-75	5-25	0-10	0-5		
8	⅜-No. 8									100	85-100	10-30	0-10	0-5		
89	⅜-No. 16									100	90-100	20-55	5-30	0-10	0-5	
9	No. 4-No. 16										100	85-100	10-40	0-10	0-5	
10	No. 4-0[4]										100	85-100				10-30
G1[5]	1½-No. 50						100	90-100		50-85		20-40	15-35	5-25	0-10	0-2
G2[5]	1½-No. 8						100	65-100		35-75		10-25	0-10	0-5		
G3[5]	1½-No. 4						100	60-95		25-60		0-15	0-5			

[1] *Coarse Aggregates (Crushed Stone, Gravel and Slag), Simplified Practice Recommendation R 163–48*, Approved June 30, 1948, National Bureau of Standards, U.S. Department of Commerce, p. 2.

[2] In inches, except where otherwise indicated. Numbered sieves are those of the United States Standard Sieve series.

[3] Special sizes for sewage trickling filter media.

[4] Screenings.

[5] The requirements for grading depend upon percentage of crushed particles in gravel. Size G1 is for gravel containing 20 percent or less of crushed particles; G2 is for gravel containing more than 20 percent and not more than 40 percent of crushed particles; G3 is for gravel containing crushed particles in excess of 40 percent. (Designated as railroad ballast, gravel.)

Table 2.—Basic Simplified Practice Recommendations numbering system

Basic SPR designations	Combinations of basic designations	Nominal size: Maximum	Nominal size: Minimum	Size limits: Maximum	Size limits: Minimum
1		3½-in.	1½-in.	4-in.	¾-in.
2		2½-in.	1½-in.	3-in.	¾-in.
3		2-in.	1-in.	2½-in.	½-in.
	357				
4		1½-in.	¾-in.	2-in.	⅜-in.
	467				
5		1-in.	½-in.	1½-in.	⅜-in.
	56				
	57				
6		¾-in.	⅜-in.	1-in.	No. 4.
	67				
	68				
7		½-in.	No. 4	⅝-in.	No. 8.
	78				
8		⅜-in.	No. 8	½-in.	No. 16.
9		No. 4	No. 16	⅜-in.	No. 50.

grading series was developed by combining the basic gradings. These combinations of the basic gradings are identified by corresponding combinations of the single digit numbers. Thus, standard aggregate No. 357, shown in the second column of table 2, which immediately follows No. 3 in the SPR table of gradings (table 1), is a combination of standard sizes Nos. 3, 5, and 7 in such proportions as to conform to the grading-band limits that were assigned to it. Similarly, standard aggregate No. 56, following No. 5, is a combination of standard sizes Nos. 5 and 6 in such proportions as to conform to the grading-band limits assigned to it.

Gradings Nos. 1F, 2F, G1, G2, and G3, listed in table 1, do not apply to highway work and are not included in the abridged version of table 1 that has been published in the AASHO and ASTM Standards. Item 10 (table 1) represents screenings and may be considered more or less a residual material from aggregate crushing and processing. It is not generally subject to close control, as indicated by the wide limits on the amount passing the No. 100 sieve, and is not considered pertinent to this discussion.

3. *Flexibility.*—The SPR gradings permit a high degree of flexibility.

The standard, stock aggregates can be combined to produce any reasonable total grading for roadbuilding purposes when further combined with suitable sands or mineral filler.

Adoption by AASHO and ASTM

The original SPR issuance, R 163-36, was adopted, essentially as promulgated, by the American Society for Testing and Materials in 1937 as Tentative Specification D 448-37T. It was carried as a Tentative Standard, with revisions in 1941 and 1942, until 1947, when it was advanced to Standard. The Standard was revised in 1949 and in 1954 and now appears in ASTM publications as Standard Specification D 448-54.

The simplified practice recommendation, including its numbering system, was adopted to cover standard sizes of coarse aggregate for highway construction by the American Association of State Highway Officials in 1942 and was designated AASHO Specification M 43-42. With some exceptions the SPR gradings were also adopted that year for crushed stone and crushed slag, for various specific purposes as in AASHO Designation M 75-42, base course; M 76-42, bituminous concrete base course and others; and also M 80-42, coarse aggregate for portland cement concrete; but in these individual applications the SPR numbering system was not used by AASHO until 1949. Since that year, all features of the SPR scheme have, with minor deviations,[3] been generally included in AASHO specifications for specific items as well as in the general group specification for coarse aggregates for highway construction. Some slight revisions of M 43-42 were made in 1949 and the designation was changed to M 43-49 which is still carried.

The present SPR system does not provide complete gradings for portland cement concrete or bituminous paving mixtures because it does not cover sands or mineral fillers. For both of these, however, there are AASHO and ASTM standards.

Aggregates for Portland Cement Concrete

The adoption by AASHO and ASTM of the SPR system for coarse aggregates for portland cement concrete has just been described. With regard to sand for portland cement concrete, the need for standardization is now met by AASHO Specification M 6-51 and ASTM Specification C 33-59, which are very similar to each other, as shown in table 3, and both of which have proved satisfactory in use. Both gradings utilize the logarithmic sieve sizes and are therefore compatible with the SPR system.

Aggregates for Bituminous Paving Mixtures

Coarse aggregates

AASHO has two specifications for coarse aggregates for bituminous paving mixtures: one for bituminous concrete base course, M 76-51, and one for bituminous concrete surface course, M 79-51. However, each of these is somewhat lacking in desirable flexibility in that only two SPR aggregate sizes are provided in each case.

[3] These deviations are as follows:

Size designation No. 3 (2 in. to 1 in.): Percentage passing the 2-in. sieve: 95-100 (SPR 163-48); 95-100 (AASHO M 43-49); 90-100 (ASTM D 448-54).

Size designation No. 67 (¾-in. to No. 4): Percentage passing the ¾-in. sieve: 90-100 (SPR 163-48); 90-100 (ASTM D 448-54); 95-100 (AASHO M 80-51); 90-100 (AASHO M 43-49).

Table 3.—AASHO and ASTM sand gradings for portland cement concrete

Sieve size	Percentage passing sieve: AASHO M 6-51	Percentage passing sieve: ASTM C 33-59
⅜-in.	100	100
No. 4	95-100	95-100
No. 8		80-100
No. 16	45-80	[1] 50-85
No. 30		25-60
No. 50	[2] 10-30	[2] 10-30
No. 100	[3] 2-10	[3] 2-10

[1] Prior to 1952 these requirements were 45-80.
[2] These requirements may be changed to 5-30; see referenced specifications.
[3] These requirements may be changed to 0-10; see referenced specifications.

B.2.4 Instrumentation Calibration Verification

Provision should be made to allow for verification of the signal conditioning instrumentation calibration (to account for the effects of zero and gain drifts).

General Requirements for Calibration Signal

The minimum acceptable facility for verification of conditioning instrumentation is a calibration signal subsystem. The calibration signal should be provided from such a source and in such a manner that there is little likelihood of variation in the calibration signal itself. This assurance then permits the operator to make adjustments in the measurement subsystem gain to offset the frequent small deviations which occur due to changes in ambient temperature and other operating parameters.

Force Measurement Calibration Signal

The most straightforward technique for providing a force measurement calibration signal is to make provisions for switching a high quality shunting resistor of known value in parallel with one arm of the force transducer strain gauge bridge. This induces an imbalance in the bridge equivalent to the application of a known force to the transducer. The resultant signal is sufficient to verify, or provide means of adjustment for, all elements of the force measurement system forward of the transducer itself.

Frequency of Use

Instrumentation calibration verification through use of calibration signals should be accomplished at the beginning of each day's operation after equipment warm up, at intervals of no more than 2 hours when the system is in continuous use, and upon the renewal of operation throughout the day after any period during which the signal conditioning equipment has been turned off or the unit has been allowed to stand without use for 30 minutes or more.

B.2.5 Check List

A check list should be available to the crew and should be used prior to the beginning of daily operations and on any occasion during the day when testing is is not, by any means, the only design factor for the grading bands for bituminous paving mixtures, it has had a predominating influence.

The second design step for bituminous paving mixtures consists of either determining or estimating the appropriate amount of bituminous binder to use. Here again practice has been established on the basis of experience and judgment in some cases while well established laboratory procedures, based on laboratory and field research, are used in others. In the latter case, the predominating factor determining suitable bitumen content is related to density or specifically to the void spaces available for binder in the compacted aggregate and the effect of overfilling or underfilling these voids on the stability and weather resistance of the plastic paving mixture.

Portland cement concrete

The situation with regard to portland cement concrete design is quite different. The design controls for concrete in present-day practice are fineness modulus, cement factor, and water-cement ratio with the cement factor and water-cement ratio being the primary variables used in designing for a specific strength range. The cement factor and water-cement ratio may also be varied to some extent to affect workability as measured by the slump test, with plasticizers being used occasionally to improve workability and strength. From the practical standpoint of field control, no one factor so adversely affects the strength and uniformity of the concrete as lack of control of water content. The proportions are set up on the basis of laboratory trial mixtures, utilizing the aggregates for the specific job and taking into consideration such factors as particle shape and surface texture, absorption, and others. Little or no use is made of total grading bands that might be set up on the basis of density or other possible design factors related to overall grading.

The practice of setting up the mixture for each job on the basis of laboratory tests is followed for reasons of practicality even though, for many years, research was conducted to develop the relations between the density of the aggregate, as influenced by the grading, and the quality of concrete.[3]

[3] Reference is made to this research and to the relations so established in *A Treatise on Concrete, Plain and Reinforced*, by F. W. Taylor and S. E. Thompson, 3d edition, 1916.

Table 7.— Composition of asphalt paving mixtures (from table III, ASTM specification for hot-mixed, hot-laid asphalt paving, Designation D 1663–59T)

Sieve size	Nominal maximum size of aggregates: 2-in.	1½-in.	1-in.	¾-in.	½-in.	⅜-in.	No. 4	No. 8
	Asphalt concrete						Sand asphalt	Sheet asphalt
GRADING OF TOTAL AGGREGATE (COARSE PLUS FINE, PLUS FILLER IF REQUIRED): AMOUNTS FINER THAN EACH LABORATORY SIEVE (SQUARE OPENING), PERCENTAGE BY WEIGHT								
2½-in.	100							
2-in.	90-100	100						
1½-in.		90-100	100					
1-in.	60-80		90-100	100				
¾-in.		40-80		90-100	100			
½-in.	35-65		60-80		90-100	100		
⅜-in.				60-80		90-100	100	
No. 4	15-50	20-55	25-60	35-65	45-70	60-80	80-100	100
No. 8	10-40	10-40	15-45	20-50	25-55	35-65	65-100	95-100
No. 16							40-80	85-100
No. 30							20-65	70-95
No. 50	2-15	2-16	3-18	3-20	5-20	6-25	7-40	45-75
No. 100							3-20	20-40
No. 200[2]	0-4	0-5	1-7	2-8	2-9	2-10	2-10	9-20
ASPHALT CEMENT, PERCENTAGE BY WEIGHT OF TOTAL MIXTURE[3]								
	3½-7½	3½-8	4-8½	4-9	4½-9½	5-10	7-12	8½-12
SUGGESTED COARSE AGGREGATES, SPR SIZES								
	3 and 57	4 and 67	5 and 7 or 57.	67 or 68 or 6 and 8.	7 or 78	8		

[1] In considering the total grading characteristics of an asphalt paving mixture the amount passing the No. 8 sieve is a significant and convenient field control point between fine and coarse aggregate. Gradings approaching the maximum amount permitted to pass the No. 8 sieve will result in pavement surfaces having comparatively fine texture, while coarse gradings approaching the minimum amount passing the No. 8 sieve will result in surfaces with comparatively coarse texture.

[2] The material passing the No. 200 sieve may consist of fine particles of the aggregates or mineral filler, or both. It shall be free from organic matter and clay particles and shall be nonplastic when tested by the method of test for liquid limit of soils (ASTM Designation D 423), and the method of test for plastic limit and plasticity index of soils (ASTM Designation D 424).

[3] The quantity of asphalt cement is given in terms of percentage by weight of the total mixture. The wide difference in the specific gravity of various aggregates, as well as a considerable difference in absorption, results in a comparatively wide range in the limiting amount of asphalt cement specified. The amount of asphalt required for a given mixture should be determined by appropriate laboratory testing or on the basis of past experience with similar mixtures, or by a combination of both.

ASTM Grading Bands for Hot-Mix Asphaltic Paving Mixtures

As already indicated, density has been generally discarded as a direct design factor for portland cement concrete but not for bituminous paving mixtures. Concurrently with the work done recently in developing a set of three sand gradings for bituminous work, ASTM has also developed a system of grading bands for combined coarse, fine, and filler aggregates for sand asphalt, sheet asphalt, and asphaltic concrete. These gradings are presented as table III in ASTM Standard Specification D 1663-59T. They are reproduced here in table 7.

The same industry and consumer representatives that

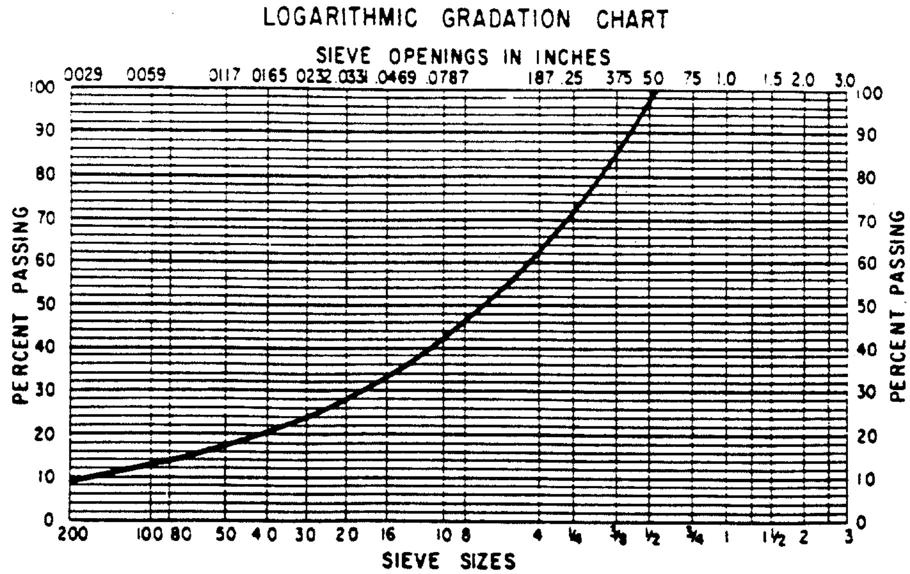

Figure 1.—A dense, stable grading plotted on the logarithmic gradation chart.

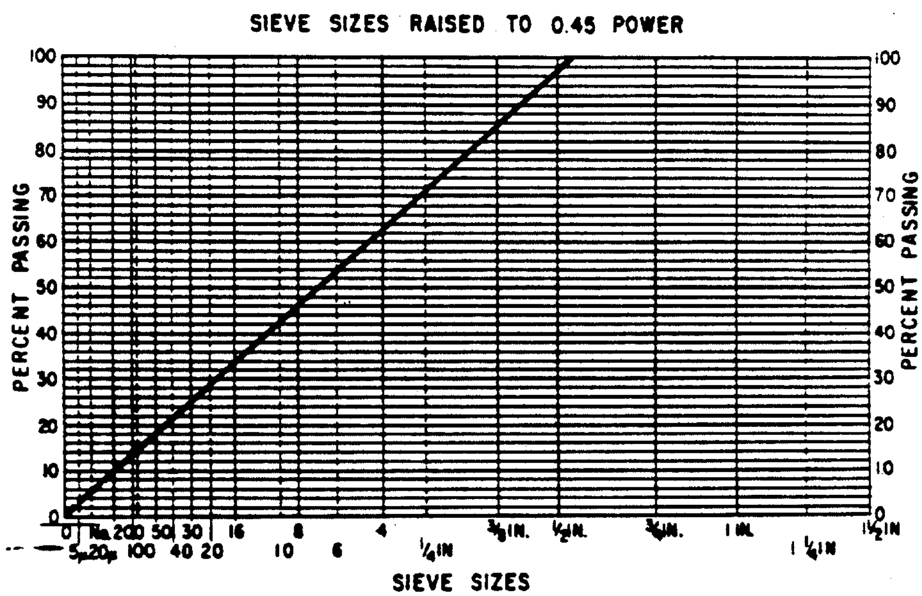

Figure 2.—Grading shown in figure 1 replotted on the 0.45-power gradation chart.

were previously named, also participated in this development. The ASTM composite gradings of table 7 are made up from SPR coarse aggregates and the ASTM sands and filler previously described. They are thus fully compatible with the SPR system. They have existed as ASTM Tentative Standards for only 2 years and were set up with the full realization that they might require some revision in the light of experience.

New gradation chart developed

In presenting the graphical material that is to follow, use is made of a new gradation chart devised by the Bureau of Public Roads, based on relations established by L. W. Nijboer of the Netherlands. Development of the chart is described in detail in the companion article in this bulletin.

In the plotting method now generally used, gradings that have proved to be highly compactible, and hence desirable as conducive to stability and resistance to moisture and weathering in bituminous paving mixtures, have a downward curving shape which is generally agreed to approximate the curve shown in figure 1. Here, the vertical scale is arithmetic and shows total percentage passing the various sieves, while the horizontal scale represents the logarithms of the sieve openings.

The simple expedient of using, for the horizontal scale, the sieve openings (inches or millimeters) raised to the 0.45 power, converts this particular curve to a straight line passing at its lower left extremity through zero percent for an imaginary sieve having zero-size openings, as shown in figure 2. Of course, grading curves having either greater or less curvature could be similarly straightened by using different exponents. It is believed, however, that the curve of figure 1 and its corresponding straight-line equivalent, figure 2, represents very nearly an ideal grading from the standpoint of density. Both research and experience indicate that the maximum particle size of the graded aggregate does not affect the shape of the maximum-density curve so that the straight-line principle using the exponent 0.45, or other basic curves and corresponding exponents, applies regardless of maximum size. The convenience of this device is readily apparent since it relieves those concerned with asphalt technology of the need to remember the exact shape of a specific curved line.

Problem mixtures

In recent years several State highway departments have reported one or more instances of difficulty with bituminous concretes produced under their own current specifications: the mixtures were hard to compact and remained "tender" for some time after rolling—that is, they were slow in developing stability. Others have reported instances of splotchy pavement surfaces where moisture was present in the aggregate. Some of these States have supplied information to the Bureau of Public Roads as to the aggregate gradings that produced these unsatisfactory mixtures.

It has been noted that, in nearly all cases, these gradings were characterized by a rise or hump in the grading curve, when plotted by the new method, because of disproportionately large quantities of finer sand fractions. It was further noted that the unsatisfactory mixtures did not contain what would be considered excessive amounts of filler, the fraction passing the No. 200 sieve.

In 1961, the Bureau of Public Roads conducted a

GRADATION CHART

SIEVE SIZES RAISED TO 0.45 POWER

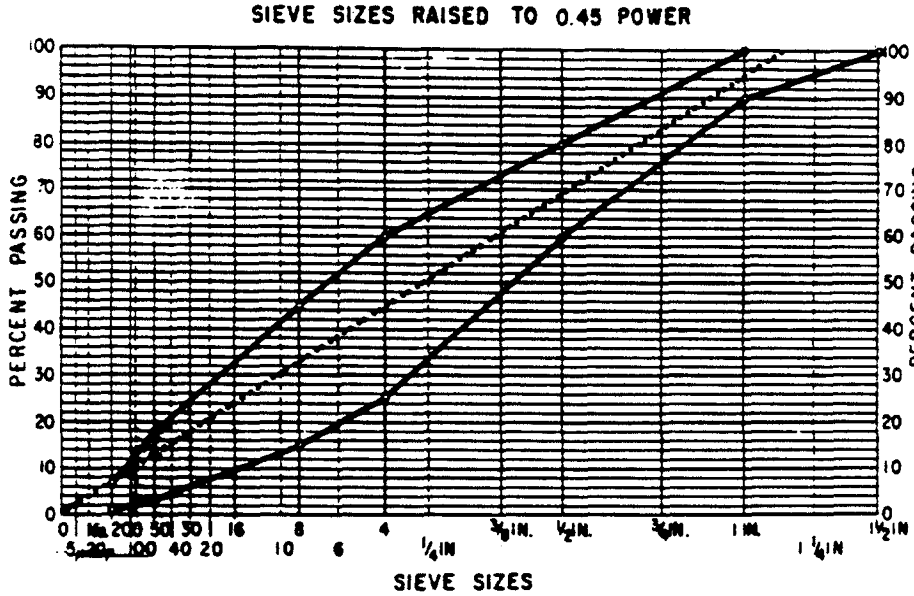

Figure 2.—ASTM limits, 1-inch nominal maximum size, compared with straight-line, maximum density grading.

laboratory study of this specific problem and utilized, for the first time, the new method of plotting gradings to facilitate interpretation of the results. Some of the results of that study are shown graphically here because they bear directly on the problem of grading control as treated in this report. They are fully reported and discussed in the companion article in this bulletin.

Among other things, the study showed that the laboratory test results were consistent with the unsatisfactory experience reported by the States on the problem mixtures described.

ASTM gradings need further study

The ASTM grading band for 1-inch maximum size asphaltic concrete is shown in figure 3 as illustrative of the eight sizes covered by ASTM Specification D 1663-59T and presented in table 7. Also shown in figure 3 is the straight (dotted) line that would represent the maximum-density grading if it can be assumed for this purpose that the maximum size for each grading may be arbitrarily established by passing the straight line midway between the upper and lower band limits for the largest sieve having both values shown.

Figures 4–6 show the aggregate gradings for the problem mixtures previously mentioned and the relation of their gradings to corresponding ASTM grading bands. These mixtures, which proved tender in the field or were splotchy when laid, were found to be low in stability when duplicated and tested in the laboratory. The two mixtures shown in figures 4 and 5 are representative of several cases in which the States reported the mixtures to be tender during construction and for considerable periods after rolling. The mixture shown in figure 6 represents several cases where splotchy pavements have been noted.

Since two of these typically humped gradings fall within the upper band limits of the corresponding ASTM gradings, even in the critical, fine sand zone, there is a strong indication that the upper band limits of the ASTM grading specifications for asphaltic concrete need some downward adjustment, at least at the No. 30 and No. 50 sieves, to further restrict the fine sand. However, a definite recommendation in this specific matter must await further study.

Basic Purpose of SPR System

The line of argument most frequently used by those opposing changes in grading control is that they are familiar and satisfied with what they are using and that they do not need or want new gradings. This points up the need for a clearer understanding of the basic purpose of the SPR scheme and of the ease with which any desired grading curve or band can be converted from one sieve-size system to another. The well established and fully validated graphical conversion method is illustrated in figure 7, which has a logarithmic horizontal scale. The equivalent straight line chart, exponent 0.45, is shown in figure 8.

In these two illustrations, an aggregate gradation band regularly specified by one of the State highway depart-

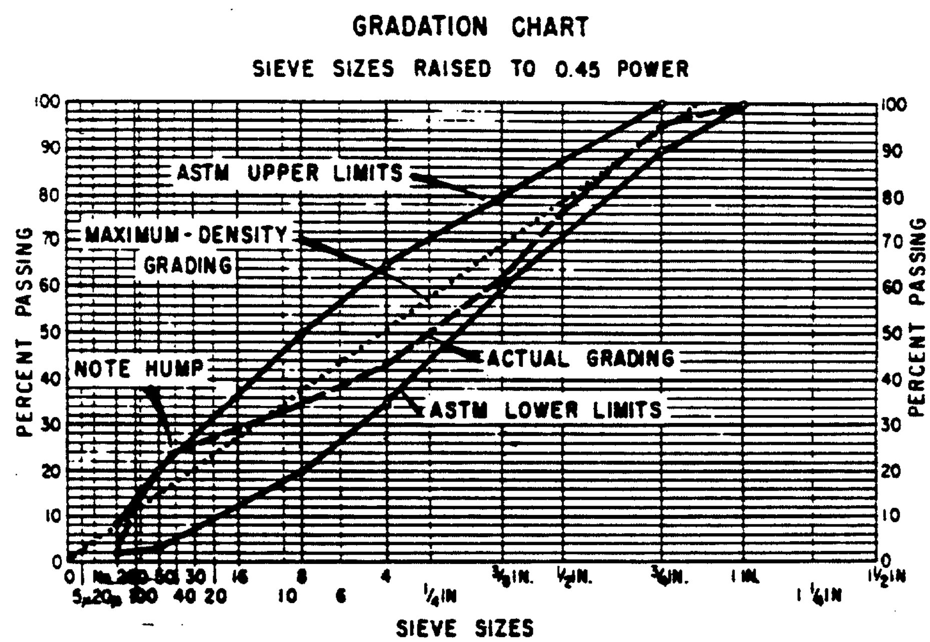

Figure 4.—Aggregate grading for a 3/4-inch nominal maximum size mixture identified as a "tender" mix.

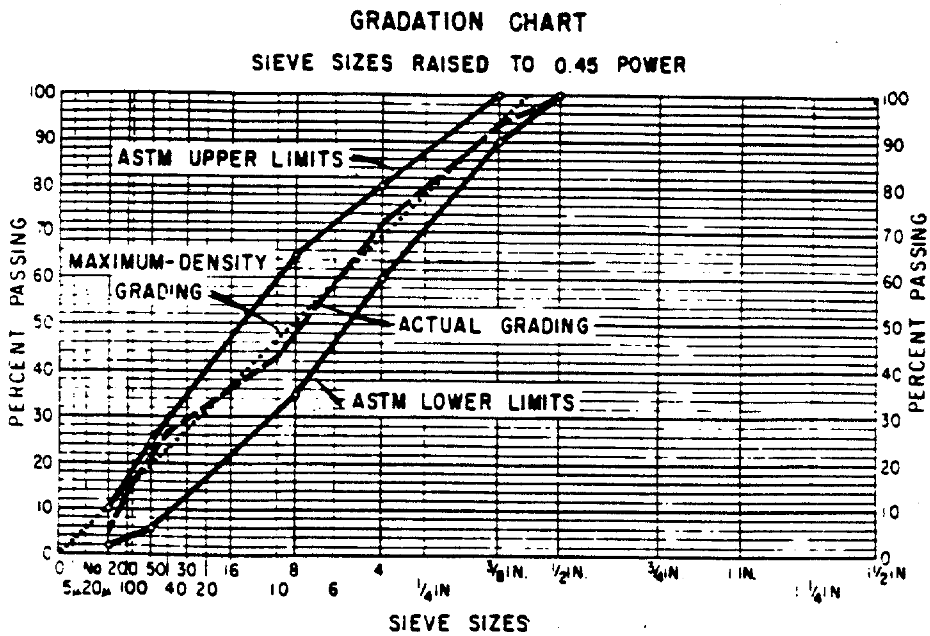

Figure 5.—Aggregate grading for a 3/8-inch nominal maximum size mixture identified as a "tender" mix.

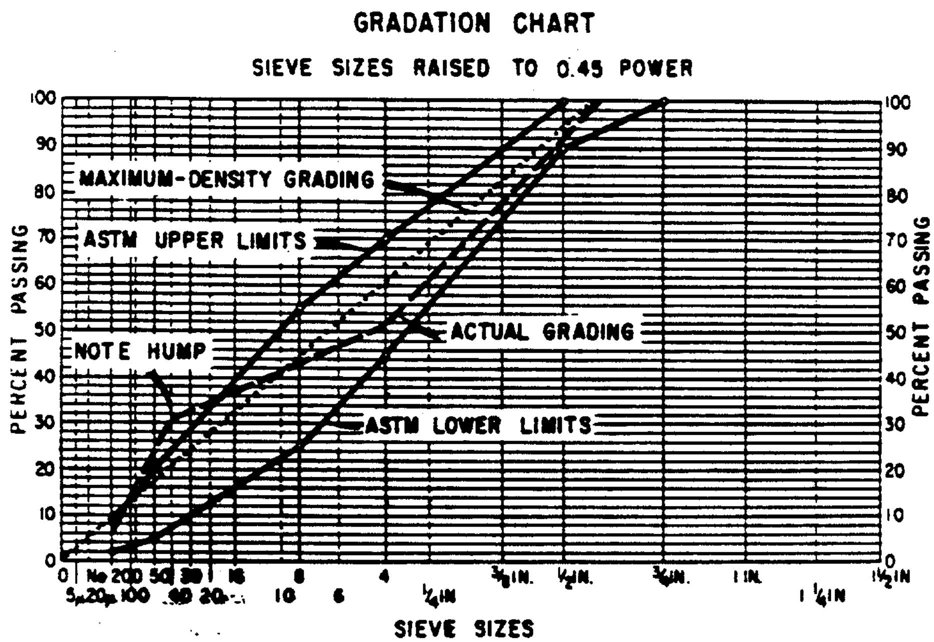

Figure 6.—Typical grading for a 1/2-inch maximum size mixture where a small amount of moisture in the aggregate has resulted in a splotchy pavement surface.

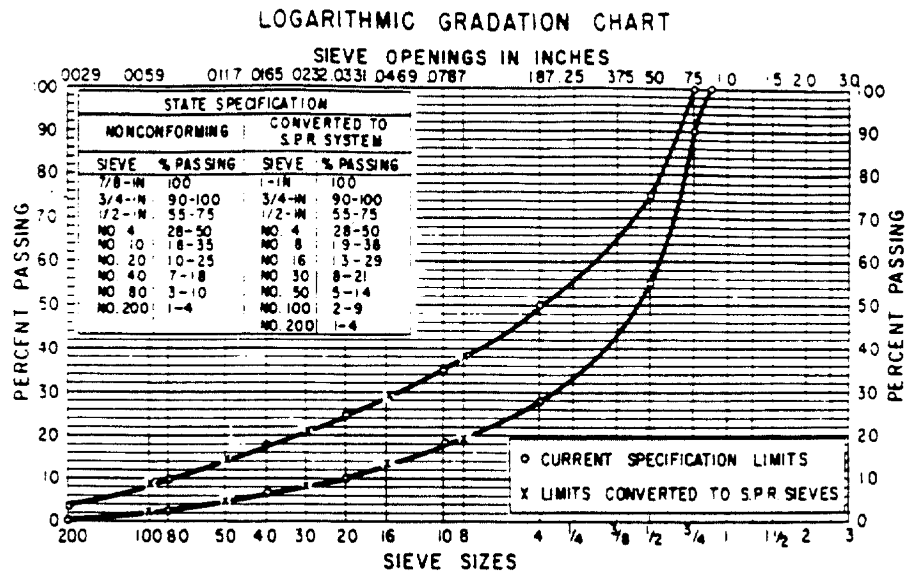

Figure 7.—Conversion of a current State specification to SPR sieve sizes, using the logarithmic gradation chart.

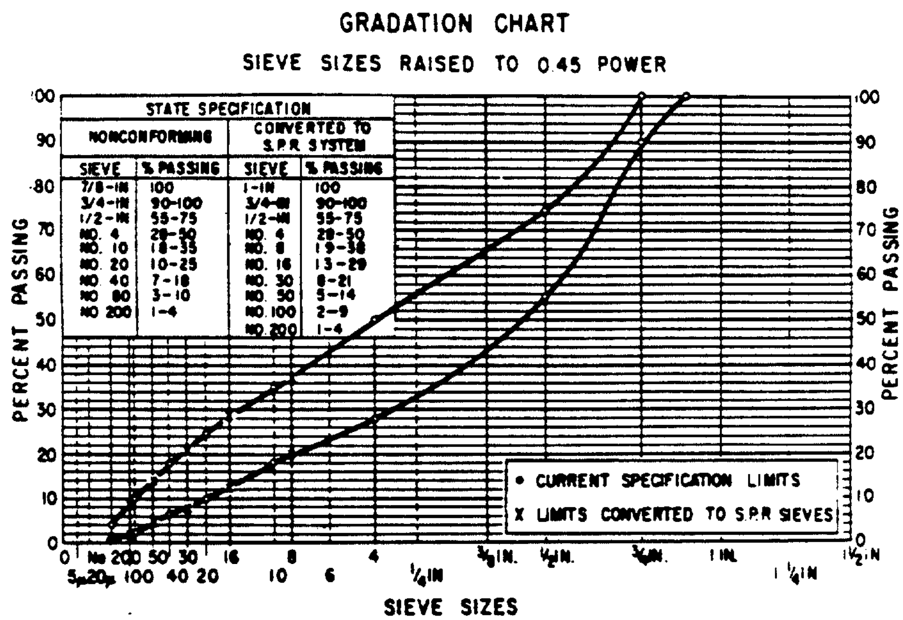

Figure 8.—Conversion of a current State specification to SPR sieve sizes, using the Public Roads gradation chart.

ments is converted from the sieve-size system traditionally used by the State to the SPR sieve-size system. The corresponding tabular gradings are shown on the charts. In making the conversion, no change is introduced in the shape or placement of the band limits and it can be stated with confidence that an aggregate produced to conform with either, will conform to the other.

Not only do these illustrations demonstrate the ease and convenience of converting other grading systems to the SPR system, or common language, but additionally, they demonstrate that the conversion does not involve changing the particle distribution of a specific, designed, or desired aggregate.

It should be pointed out in this connection that the use of the SPR sieve series to express total gradations, as for example, 1½-inch maximum size to No. 200, does not assure that specific desired gradings can always be made up from combinations of standard SPR numbered aggregate fractions with ASTM sand and filler, although in normal practice such situations should be comparatively rare.

Generally, the same freedom to modify grading band control limits to exploit field experience or the findings of research is inherent in the standardized scheme presented here as exists in the multiplicity of State specifications now in use. The need for some degree of freedom in this respect is fully recognized.

However, this philosophy cannot legitimately be used to justify the kind of trivial differences that account for a large proportion of the hundreds of aggregate gradations appearing in State specifications.

Recommended Course of Action

The study which is the subject of this report was undertaken for the purpose of furthering the three objectives mentioned—drastic reduction of "standard" gradations, agreement on sieve sizes, and agreement on a uniform system of identification of standard gradations. Because of the inherent flexibility of the SPR scheme, coupled with compatible sand and filler specifications now available as AASHO and ASTM standards, it is believed that a large proportion of the many special gradings now appearing in State specifications could be eliminated, thereby achieving important economies in highway construction. In many cases, it would only be necessary to convert to the SPR standard sieve sizes, as illustrated in figures 7 and 8, and to use SPR grading designations.

A desirable course of action and one that is strongly recommended for implementation by the American Association of State Highway Officials is essentially as follows:

1. Elimination from individual State specifications of all sieve sizes that are at variance with those officially adopted by AASHO and substitution therefor of conforming sieve sizes. This could be done easily by utilizing the method illustrated in figures 7 and 8. The new grading tables would provide the same gradations as those previously specified.

2. Elimination from individual State specifications of other gradation requirements not conforming to AASHO or related ASTM standards to the maximum practicable extent.

3. Retention for use, as special provisions or supplemental specifications, of such nonconforming gradation requirements as may be justified.

Standards Now Recommended

The following AASHO and ASTM standards are recommended for general use by all highway departments:

1. AASHO M 43–49, standard sizes of coarse aggregate for highway construction.

2. AASHO M 80–51, coarse aggregate for portland cement concrete.

3. AASHO M 6–51, fine aggregate for portland cement concrete.

4. ASTM D 692–59T, coarse aggregate for bituminous paving mixtures.[4]

5. ASTM D 1073–59T, fine aggregate for bituminous paving mixtures.

6. ASTM D 242–57T, mineral filler for sheet asphalt and bituminous concrete pavements.

In addition to the above six standards, the following tentative standard is recommended for study, possible revision, and general use:

7. ASTM D 1663–59T, hot mixed, hot laid asphalt paving mixtures.

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[4] Requires one revision for adoption by AASHO to conform to AASHO M 43–49, namely for aggregate No. 3 the percentages passing the 2-in. sieve would have to be changed from 90–100 (ASTM) to 95–100, as now required in AASHO M 43–49.

The third approach is a compromise in which the pipe is wrapped in a filter fabric and the trench is backfilled with a filter aggregate or coarse sand as shown in the bottom sketch of Figure 6. In this approach, the aggregate acts as a filter keeping the fines from clogging the filter fabric. The coefficient of permeability of the filter aggregate material varies, but it is generally much lower than an open-graded aggregate backfill.

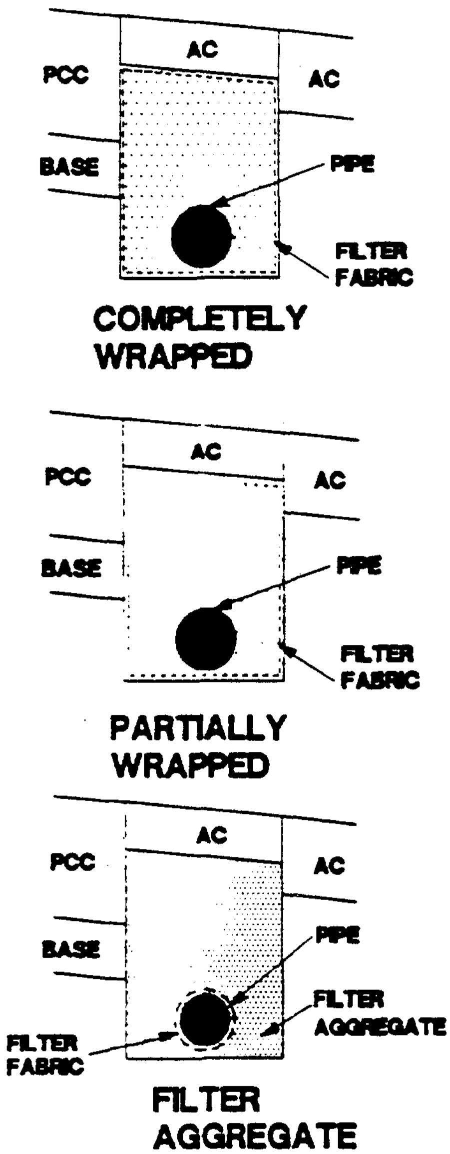


Figure 6. COMPARISON OF EDGEDRAIN DESIGN

It is pointed out that in all of the approaches any erodible fines in the base course will be washed out. The difference in the approaches is the manner in which the fines are handled.

It should be noted that there is no way to prevent a filter adjacent to a material with a high percentage of fines from eventually clogging. If of sieve sizes (see fig. 7 in the preceding article, p. 10). This chart, which will be referred to hereafter as the logarithmic gradation chart, has had wide use for some 30 years and has proven valuable in illustrating individual gradations and determining their position relative to specification limits. This type of chart, however, has one significant disadvantage in that it shows a maximum density gradation as a deeply sagging curve, the shape of which is hard to define.

To provide a better means of relating actual aggregate gradation to maximum density gradation, a new chart has been devised by the Bureau of Public Roads. The horizontal scale for the several sieve sizes of this chart is a power function rather than the logarithm of the sieve opening in microns. The vertical scale is arithmetical, the same as for the logarithmic chart. An important feature of the new chart is that it provides for a zero theoretical sieve size. Thus, for practical purposes, all straight lines plotted from the lower left corner of the chart, at zero percent passing zero theoretical sieve size, upward and toward the right to any specific maximum size, represent maximum density gradations. The exponent of the power function is 0.45, i.e., the horizontal scale represents the various sieve openings in microns raised to the 0.45 power.

Background of development

The selection of the 0.45 exponent was based on research performed by L. W. Nijboer of the Netherlands and first published in 1948.[3] Nijboer used a double logarithmic gradation chart in a study of the influence of aggregate gradation on mineral voids. All gradations used in his study were represented by straight lines, with various slopes, when plotted on his chart; the variation in slope resulting from his use of several different gradations of the same maximum ($\frac{3}{4}$-inch) size. Nijboer made two series of tests on compacted bituminous mixtures, using rounded gravel for the coarse aggregate in one series of tests and an angular crushed stone in the other. Mineral voids were determined for all of the mixtures and were plotted

[3] *Plasticity as a Factor in the Design of Dense Bituminous Road Carpets*, by L. W. Nijboer, Elsevier Publishing Co., 1948.

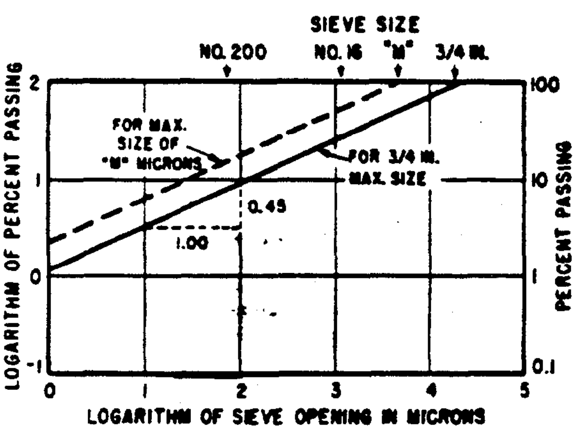


Figure 1.—Maximum density gradation plotted on a double log chart.

against the slopes of the straight line gradation curves. For both types of coarse aggregate, the minimum mineral voids, or maximum aggregate density, occurred for a gradation having a slope of 0.45 on the double log chart.

Figure 1 shows this maximum density gradation for a $\frac{3}{4}$-inch maximum size aggregate plotted on a double log chart. The figure also illustrates a maximum density curve for a gradation with a maximum size designated as M microns, for the following discussion in which it is assumed that all maximum density curves have a slope of 0.45 on the double log chart regardless of maximum size.

In developing the equation for a maximum density curve let:

M = maximum size of aggregate in microns.

S = size of opening for a particular sieve.

P = percentage passing the particular sieve.

$\log B$ = intercept on vertical axis of the chart.

The general equation of the curve is:

$$\log P = \log B - 0.45 \log S \quad \ldots\ldots\ldots\ldots (1)$$

Other equations are:

$$\log 100 - \log B = 0.45 (\log M - \log 1); \text{ or}$$

$$2 - \log B = 0.45 (\log M); \text{ or}$$

$$\log B = 2 - 0.45 \log M \quad \ldots\ldots\ldots\ldots (2)$$

Substituting equation (2) in equation (1) we have:

$$\log P = 2 - 0.45 \log M - 0.45 \log S; \text{ or}$$

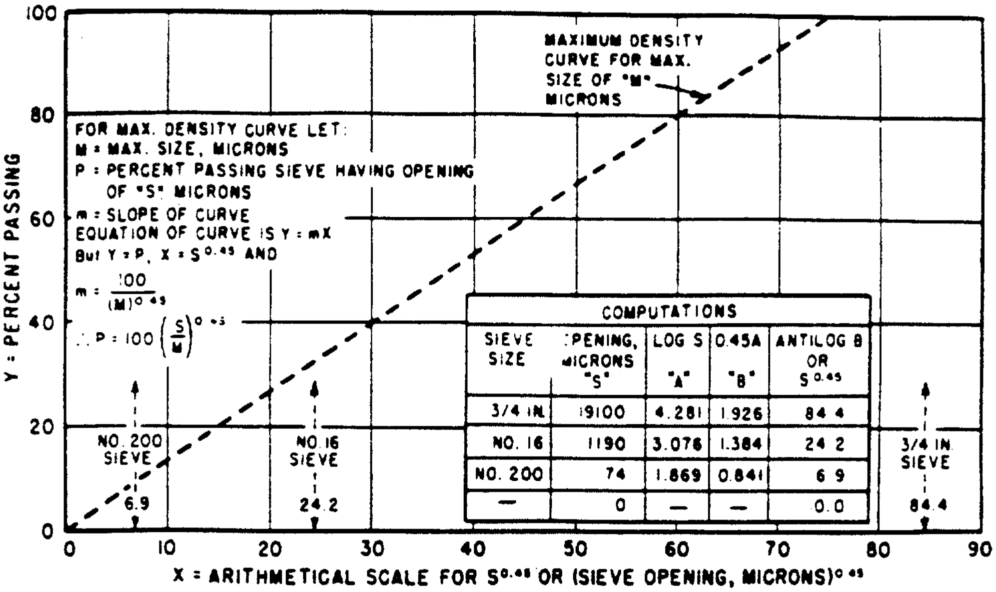
$$\log P = 2 - 0.45 (\log S - \log M); \text{ or}$$

$$P = 100 \left(\frac{S}{M}\right)^{0.45} \quad \ldots\ldots\ldots\ldots (3)$$

The exponent in equation (3) is the one used in designing the new gradation chart. By the use of logarithms, the sizes of sieve openings in microns were raised to the 0.45 power. These values were then employed with a suitable arithmetical scale for establishing the horizontal position of each sieve. The procedure is illustrated for a few of the sieve sizes on figure 2.

Figure 2 also illustrates how maximum density gradation is indicated for a gradation having a maximum size of M microns: simply by plotting a straight line from the origin, at the lower left corner of the chart, to the selected maximum size at the top of the chart. As can be seen from the information on the left side of the chart, the equation for such a line is that shown above as equation (3). Thus, any gradation that will plot as a straight line through the origin of the new chart will also plot as a straight line on the double log chart of Nijboer and will have a slope of 0.45.

The new gradation chart described in this article, and hereafter referred to as the Public Roads gradation chart, is not, strictly speaking, an entirely new type. The National Crushed Stone Association, in its *Crushed Stone Journal*, has been using a square-root gradation chart for several years to illustrate gradations. The only difference between the Association's chart and the new one presented here is that the former is based on an exponent of 0.50 for the power function instead of 0.45. The research of Nijboer and data to be presented later in this article show that 0.45 is a more realistic value for indicating maximum density.



COMPUTATIONS

SIEVE SIZE	OPENING, MICRONS "S"	LOG S "A"	0.45A "B"	ANTILOG B OR $S^{0.45}$
3/4 IN.	19100	4.281	1.926	84.4
NO. 16	1190	3.076	1.384	24.2
NO. 200	74	1.869	0.841	6.9
—	0	—	—	0.0

Figure 2.—Illustration of computations and method of positioning sieve sizes in setting up the Public Roads gradation chart.

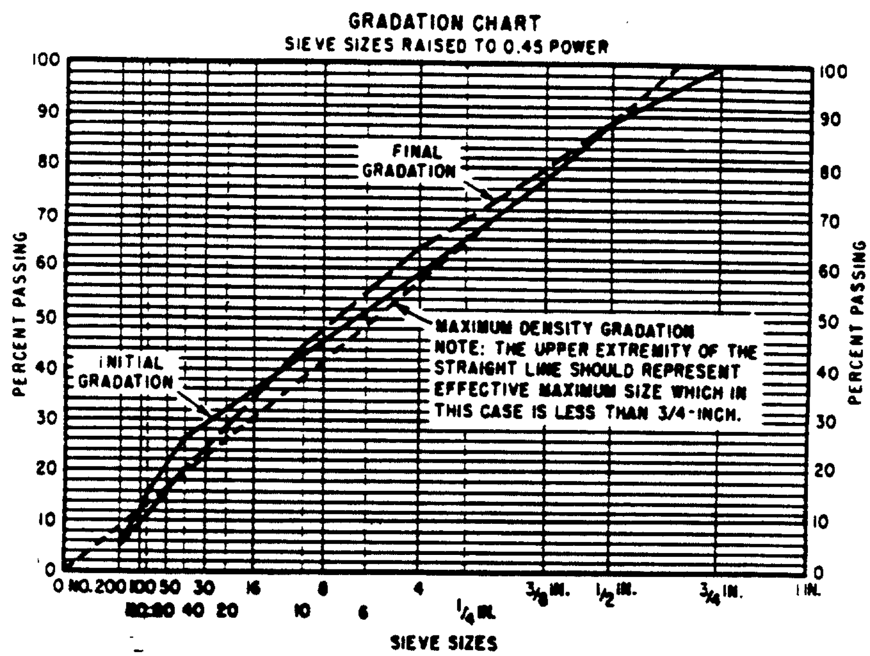


Figure 3.—Gradations of problem mixture (project A) compared with maximum density gradation.

Using Chart in Study of Tender Mixes

Soon after the Public Roads gradation chart was developed it was used to study gradations of aggregate from several bituminous mixtures that had been reported as having unsatisfactory compaction characteristics. During the past 4 or 5 years, engineers have reported several instances of hot asphaltic concrete mixtures that con-formed to their specifications but could not be compacted in the normal manner because they were slow in developing sufficient stability to withstand the weight of rolling equipment. Such mixtures are usually called "tender" mixes.

Those having experience with such mixtures have tended to place most of the blame on the particular asphalt used. Occasionally it was recognized that such factors as high temperatures of the mixture, the air, and the underlying structure, excessively heavy rolling equipment, or the presence of moisture in the mixture might contribute to the unsatisfactory condition. The possibility was very seldom considered that aggregate gradation could be an equally important factor and that the grading requirements used could be contributing to this problem.

To illustrate the type of aggregate gradation that seems to be rather consistently associated with tender mixtures, some specific examples from three different parts of the country are discussed in the paragraphs that follow.

On a 1958 construction project, identified as project A, the engineers were careful to select cold feed materials and proportions for the wearing course mixture that would provide a median gradation within the specification limits. Despite these precautions, the resulting mixture had the characteristics of a tender mix. It was described as a critical mixture which did not compact satisfactorily at any asphalt content within the specification limits. At asphalt contents only slightly below the one that was most nearly satisfactory, the mixture was friable and developed cracks behind the finishing machine. At only slightly higher asphalt contents the mixture was too unstable to compact.

Although the engineers suspected the asphalt was at fault they decided to try a modified gradation, which resulted in a less critical mixture with greatly improved compaction characteristics. The initial and final gradations and the corresponding maximum density gradation are shown plotted on the Public Roads gradation chart in figure 3. Attention is called to the hump in the curve above the maximum density line at the Nos. 50, 40, and 30 sieve sizes for the initial gradation used in the unsatisfactory mixture and to the absence of a hump at these sieve sizes for the final gradation which produced the more satisfactory mixture.

Figure 4 shows gradations used on three other projects, each having a hump above the maximum density line at about the No. 30 sieve when plotted on the Public Roads chart. Two of these, for projects B and D, built in 1958 in a different State than project A, are gradations of mixtures containing gravel and sand that were described

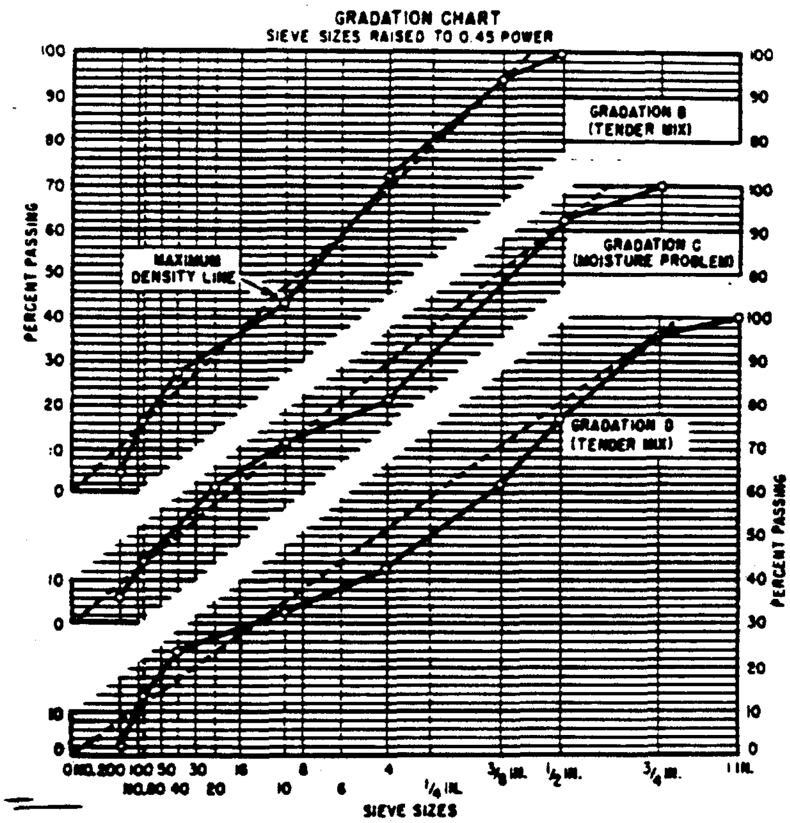


Figure 4.—Gradations of problem mixtures (projects B, C, and D) compared with maximum density gradations.

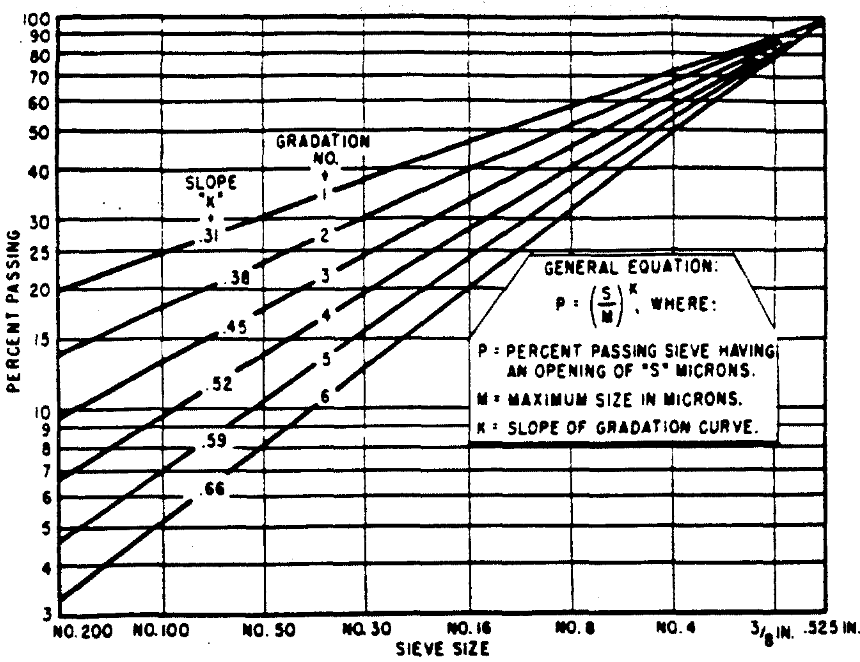
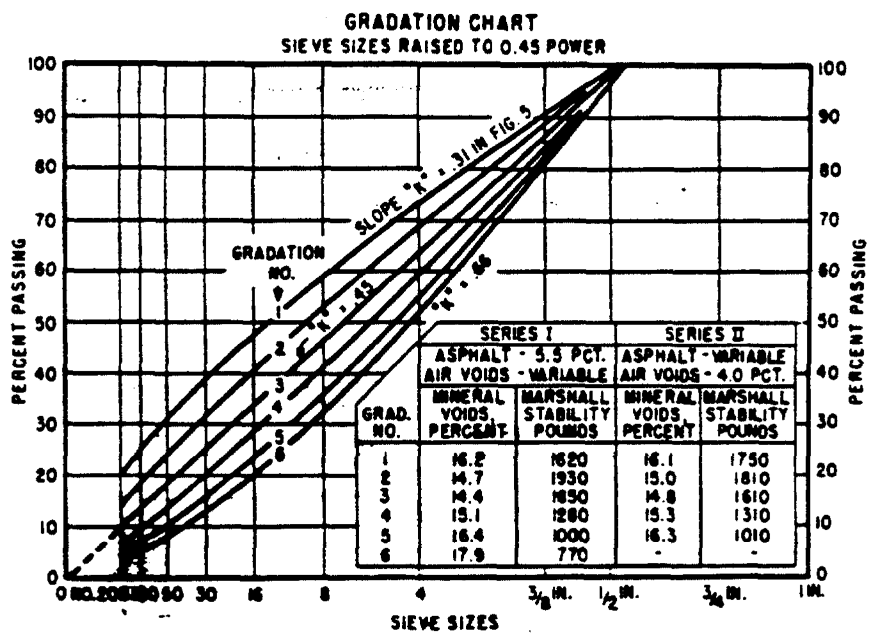


Figure 5.—Gradations Nos. 1-6 plotted on double log chart.



GRAD. NO.	SERIES I: ASPHALT - 5.5 PCT. AIR VOIDS - VARIABLE		SERIES II: ASPHALT - VARIABLE AIR VOIDS - 4.0 PCT.	
	MINERAL VOIDS, PERCENT	MARSHALL STABILITY POUNDS	MINERAL VOIDS, PERCENT	MARSHALL STABILITY POUNDS
1	16.2	1620	16.1	1750
2	14.7	1930	15.0	1810
3	14.4	1850	14.8	1610
4	15.1	1280	15.3	1310
5	16.4	1000	16.3	1010
6	17.9	770	-	-

Figure 6.—Gradations Nos. 1-6 (straight-line gradations in fig. 5) plotted on the Public Roads gradation chart.

as tender mixes. The third gradation, for project C, is typical of those used in a State which has had considerable difficulty with moisture problems in laying bituminous pavements containing certain coarse aggregates. A very small amount of moisture in such mixtures often results in a splotchy pavement surface.

There have been exceptions, but nearly all gradation curves of problem mixtures studied by the research laboratories of the Bureau of Public Roads have been characterized by a hump above the maximum density line at or near the No. 30 sieve. Such mixtures have an excess of fine sand in relation to total sand. This excess not only results in lower compacted densities but tends to float the larger particles and destroy stability that might otherwise result from coarse aggregate interlock. In addition, fine sand is inherently less stable than coarse sand.

Thus, improper aggregate gradation is identified as an important contributing factor to the unsatisfactory behavior of some bituminous mixtures. Other factors, such as asphalt characteristics, high temperatures, and moisture vapor cannot be ruled out; but unsatisfactory grading, particularly oversanding in the fine sizes, must not be overlooked as a possible source of trouble.

Laboratory Evaluation of Gradation Chart

To evaluate further the usefulness of the new Public Roads gradation chart, a laboratory study was undertaken with two main objectives: To substantiate Nijboer's findings, and to determine more precisely the effect of "hump" gradations on mineral voids and stability of compacted asphaltic concrete. The study employed the gyratory method of molding and the Marshall stability test.

The investigation was limited to 24 different gradations of gravel, sand, and limestone dust aggregate having a maximum size of 0.525 inch. These gradations are shown in table 1 of the appendix (p. 24), together with values for effective specific gravity values which were used in computing voids.

Verification of 0.45 exponent

In order to verify Nijboer's findings, the first six gradations were made up so that they would plot as straight lines with varying slopes *K* on the double log chart, as shown in figure 5. When plotted on the New Public Roads

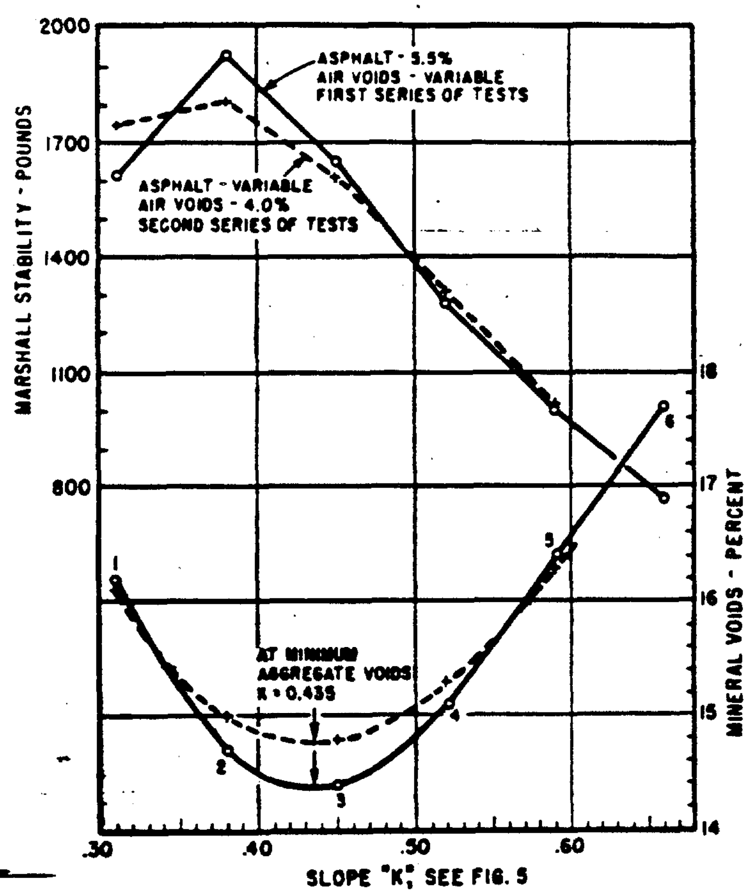


Figure 7.—Mineral voids and Marshall stabilities of gradations Nos. 1–6.

gradation chart, figure 6, five of these gradings plotted as curves because of the variations in the exponent K. Only gradation No. 3, which had a slope (or exponent K) of 0.45 in figure 5, plotted as a straight line in figure 6. Figure 6 also contains, for ready reference, data on mineral voids and Marshall stability extracted from table 4 of the appendix. It will be noted that the aggregates were combined with asphalt in two series of mixtures, one with constant asphalt content of 5.5 percent and the other with variable asphalt content to produce constant air voids of 4.0 percent.

Figure 7 shows the Marshall stability and mineral void values in graphical form. In the upper part of this figure, Marshall stability (see tabulation, fig. 6) is plotted against K or slope from the double log chart (see fig. 5). The solid-line curve represents test results for a constant percentage of asphalt, the first series of tests; the dashed line represents results for a constant percentage of air voids, the second series of tests. Corresponding curves for mineral voids are shown in the lower part of the figure.

It will be noted in figure 7 that minimum aggregate voids, or maximum aggregate densities, occur at the point where K equals 0.435. This is slightly lower than Nijboer's value of 0.45 on which the new Public Roads gradation chart is based, but the slight difference is not considered significant. Figure 7 also shows that the value of K had a pronounced effect on Marshall stability for both series of tests. For the coarsest grained aggregate (grading No. 6, for which $K=0.66$), stability was less than 800 pounds. For the finest grained aggregate of the study (grading No. 1, for which $K=0.31$), stability was between 1,600 and 1,750 pounds for the two series. The maximum values for the two series were between 1,800 and 1,950 pounds.

Study of "hump" gradations

Figures 8–10 use the Public Roads gradation chart to illustrate gradations that plotted with a hump at the No. 30 sieve size and to compare them with a maximum density curve (gradations Nos. 7–11 and 13–21, shown in table 1 of the appendix). Each of these figures also includes a tabulation (extracted from table 4 of the appendix) showing mineral voids and stability for mixtures with constant asphalt content and with a constant volume of air voids.

Figure 8 shows the gradation curves and test results for gradations Nos. 7–11, each of which had 46.0 percent passing the No. 8 sieve, the same as that for the maximum density curve. These gradations are considered optimum in the amount of total sand.

As will be seen in figure 8, the curve for gradation No. 11 plotted as a straight line from the No. 8 sieve to the No. 200 sieve and this portion of the curve is below the maximum density line. The curve for gradation No. 10 is on the maximum density line from maximum size to the No. 30 sieve but then drops below the maximum density line to the No. 200 sieve; it therefore has a slight hump at the No. 30 sieve but the fact that this hump is not above the maximum density line is considered significant since gradation No. 10 had the lowest mineral voids of this group of gradations for both series of tests, and also had the highest stability for the series in which asphalt content was maintained constant. Its stability was only 30 pounds lower than the highest value in the second test series, where air voids were maintained constant.

The humps at the No. 30 sieve size for gradations Nos. 9, 8, and 7 are progressively larger than that for gradation No. 10 and are all above the maximum density line. As the humps become more pronounced the gradations show increasing void contents and decreasing stabilities.

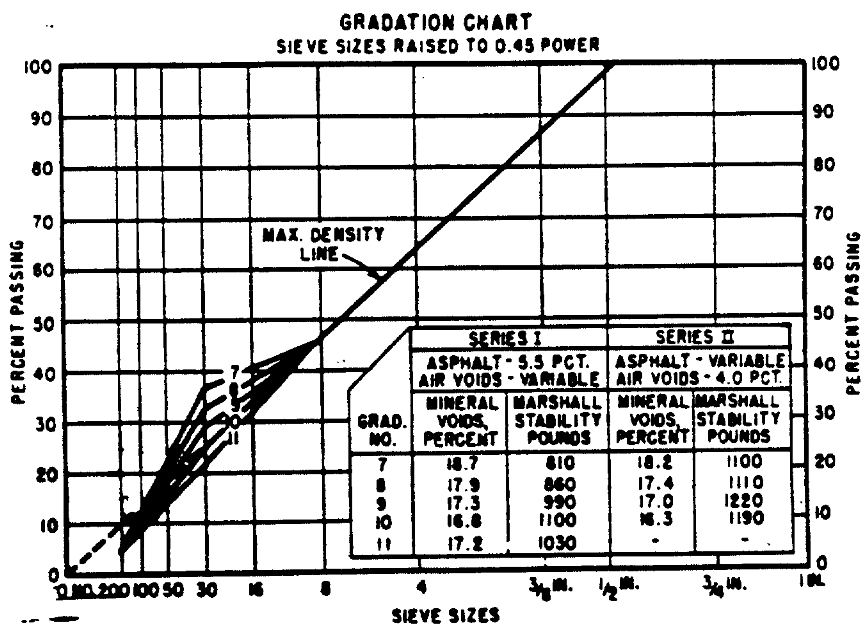


Figure 8.—Hump gradations of gravel mixtures, medium in total sand.

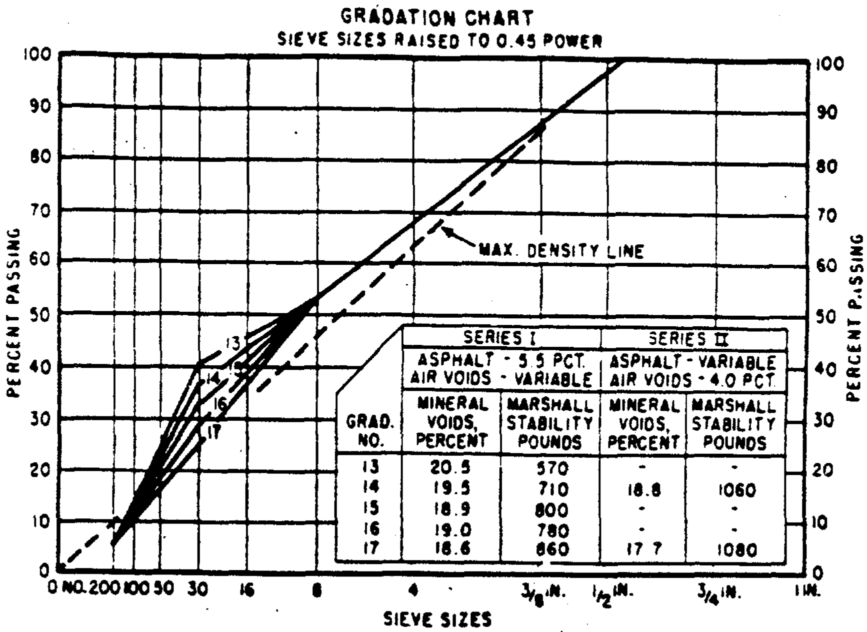


Figure 9.—Hump gradations of gravel mixtures, high in total sand.

Figure 9 shows the gradation curves and test results for gradations Nos. 13–17, all of which had 53.3 percent passing the No. 8 sieve and are considered high in total sand when compared to the gradations shown in figure 8.

The curve for gradation No. 17 does not have a hump at the No. 30 sieve size: it is a straight line from the No. 8 to the No. 200 sieve and intersects the maximum density curve at the No. 30 sieve. This gradation showed the lowest value of mineral voids for the group. The curve for gradation No. 16 has a slight hump above the maximum density curve at the No. 30 sieve size, and gradation curves Nos. 15, 14, and 13 have increasingly larger humps. Allowing for experimental error, it will be noted that, in general, increasing magnitude of the hump corresponded with increasing mineral voids and decreasing stability for the series of tests where the asphalt was maintained constant. Where the air voids were maintained constant, in the two instances shown, there was a slight increase in mineral voids but no significant change in stability.

Figure 10 shows the curves for gradations Nos. 18–21, which had 38.9 percent passing the No. 8 sieve and are considered low in total sand when compared to the gradations shown in figure 8.

The entire curve for gradation No. 21 plotted below the maximum density line and has a very slight hump at the No. 30 sieve size. The curve for gradation No. 20 has a slight hump and touches the maximum density line at the No. 30 sieve size; otherwise it is completely below the maximum density line. This is considered significant since gradation No. 20 had the lowest mineral voids and the highest stability of this group of gradations in both series of tests.

Gradation No. 19 had a considerable hump at the No. 30 sieve size, above the maximum density curve. This gradation had greater mineral voids and less stability than those of gradation No. 20. Gradation No. 18 had the largest hump of the group and it also had the highest percentage of mineral voids and the lowest stabilities.

Conclusions on hump gradations

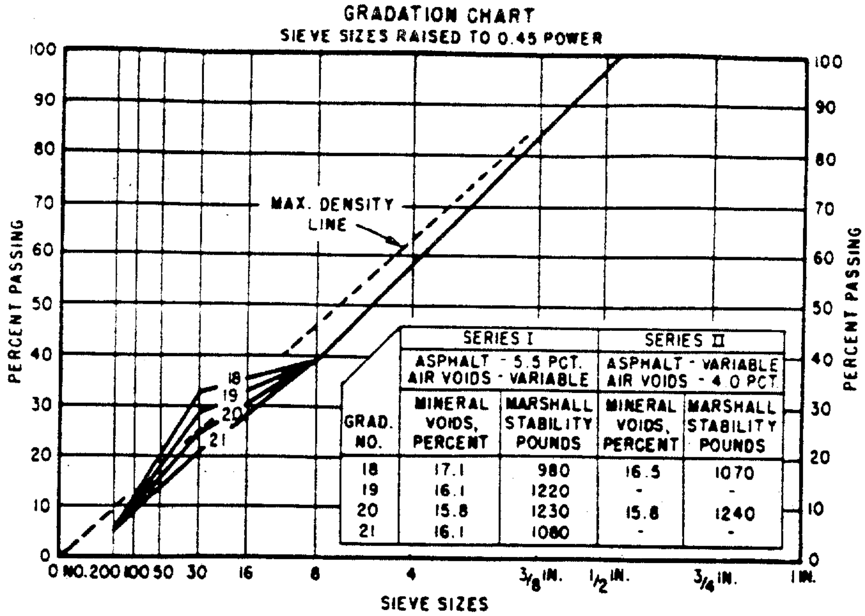
The above discussions, based on figures 9–10, of humps in gradation curves at the No. 30 sieve size, may be summarized as follows:

1. A hump above the maximum density line in all cases was associated with a lower aggregate density (higher mineral voids) than a hump that just touches the maximum density line.

2. In nearly all cases the hump also was associated with a lower Marshall stability value. The reduction in stability was more pronounced for the series of tests in which the asphalt content was maintained constant than for the series in which the asphalt content was varied to provide a constant volume of air voids.

3. The greater the magnitude of the hump above the maximum density line, the lower was the aggregate density (in all cases) and the stability (in nearly all cases).

Thus, based on results of laboratory tests of gravel mixtures, the presence of a hump in the aggregate gradation curve at about the No. 30 sieve and above the maximum density line is indicative of an undesirable gradation. The extent to which differences in laboratory density and stability can be related to field compaction and performance characteristics is not now known. However, the results of these laboratory tests and studies of known field examples discussed earlier do show that "hump" gradations may be a contributing factor toward the unsatisfactory behavior of mixtures. Further verification of their effect should be determined by controlled field studies.



GRAD. NO.	SERIES I — ASPHALT - 5.5 PCT. AIR VOIDS - VARIABLE: MINERAL VOIDS, PERCENT	MARSHALL STABILITY POUNDS	SERIES II — ASPHALT - VARIABLE AIR VOIDS - 4.0 PCT.: MINERAL VOIDS, PERCENT	MARSHALL STABILITY POUNDS
18	17.1	980	16.5	1070
19	16.1	1220	-	-
20	15.8	1230	15.8	1240
21	16.1	1080	-	-

Figure 10.—Hump gradations of gravel mixtures, low in total sand.

Use of chart in improving gradations

One of the advantageous uses of the Public Roads gradation chart is in revising gradations to obtain greater or lesser mineral voids. Often it is desirable to decrease the mineral voids to provide a more stable mixture. At other times it is desirable to increase the mineral voids to allow room for more asphalt in the mixture and thereby improve its durability: for example, McLeod [3] prefers to maintain a minimum of 15-percent mineral voids in the compacted mixture.

Based on this 15-percent voids criterion the maximum density gradation used in these tests, No. 3, would not be satisfactory since it had mineral voids of 14.4 and 14.8 percent, respectively, for the first and second series of tests. Gradation No. 10, which is similar to gradation No. 3 except for a lower dust content, would be satisfactory because its respective mineral voids were 16.8 and 16.3 percent, appreciably greater than the 15-percent criterion. Thus, one effective way of modifying a gradation to provide greater or lesser mineral voids is to change its dust content. However, this may not be practical or it may be more economical to modify the gradation at other sieve sizes.

If the modification is to be made by varying the gradation of the sand portion, figures 8–10 suggest that it might be done by increasing or decreasing the percentage passing the No. 30 sieve for the entire aggregate while maintaining constant the percentages passing the No. 8 and No. 200 sieves. In figure 10, for example, if gradation No. 19 should prove too dense it could be modified to a less dense gradation by increasing the percentage of aggregate passing the No. 30 sieve and thereby moving the gradation curve away from the maximum density line: or it could be made denser by reducing the percentage passing the No. 30 sieve to bring the curve closer to the maximum density line.

If, however, the modification is to be made by adjusting the percentage of sand or by varying the gradation of the coarse aggregate, another factor must be taken into account. An allowance must be made for the fact that skip gradations can promote higher density.

Skip gradations

Figure 11 shows curves and data for three skip gradations, Nos. 22–24. The slope of these curves between the No. 4 and No. 8 sieve sizes is appreciably less than the slopes of the remaining portions. They might be referred to as gradations that plot with a hump at the No. 8 sieve size. Figure 11 also shows curves and data for the maximum density gradation, No. 3, and for gradation No. 12 which plots as a straight line from the maximum size to the same percentage passing the No. 200 sieve as that of the other curves.

Comparing the curves in figure 11 with respect to their positions relative to the maximum density line is complicated by the fact that some of them cross it. For example, gradation No. 12 plotted closer to the maximum density line than gradation No. 22 at the No. 4 and larger sieve sizes, but further from the line at the No. 16 and smaller sieve sizes. On the average, however, gradation No. 12 plotted closer to the maximum density line than gradation No. 22, and it showed the higher density (lower mineral voids).

Similarly, skip gradation No. 22 plotted closer to the maximum density line than skip gradation No. 23 at the

[3] *Relationships between Density, Bitumen Content, and Voids Properties of Compacted Bituminous Paving Mixtures*, by N. W. McLeod, Proceedings of the 35th annual meeting of the Highway Research Board, vol. 35, 1956, pp. 327–404.

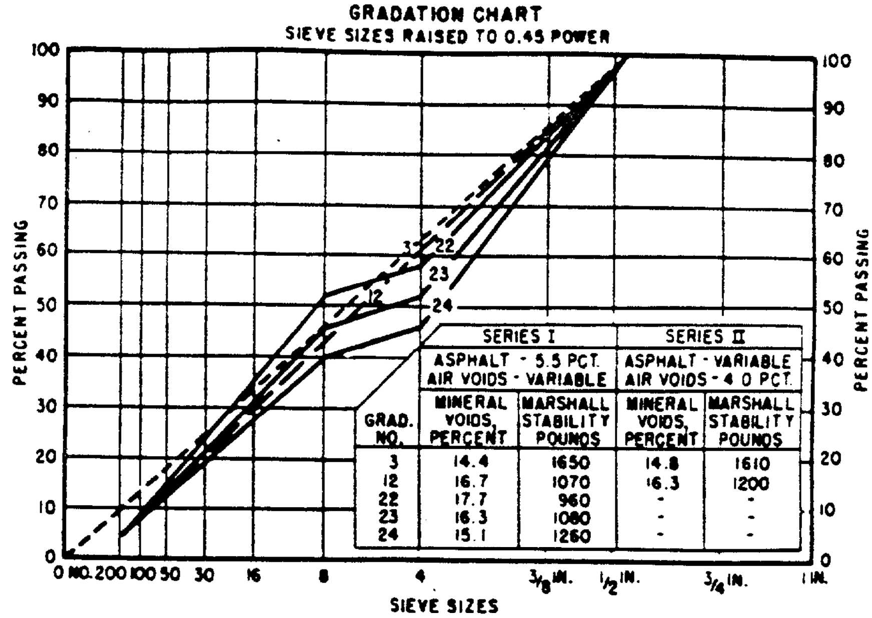


Figure 11.—Skip gradations compared with gradations Nos. 3 and 12.

No. 4 and larger sieve sizes, further from the line at the No. 8 sieve size, and again closer to the line at the No. 30 and smaller sieves. Which gradation plotted closer to the maximum density line on the average is questionable, but gradation No. 23 had the higher density.

There is no doubt that gradation No. 24 plotted the furthest from the maximum density line and it showed the highest density of the three skip gradations. Its density, however, was not as great as that of gradation No. 3, the one that is used to represent maximum density on

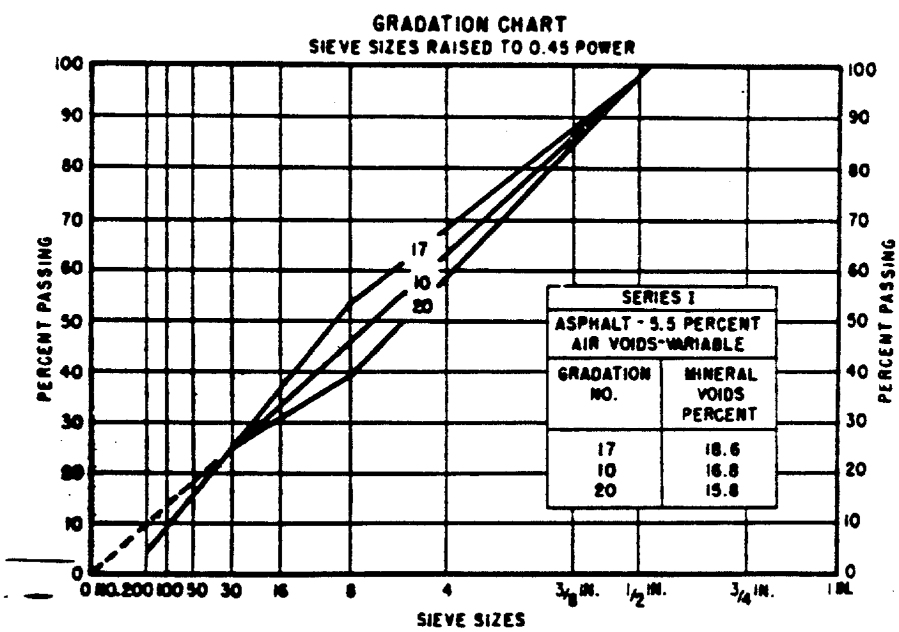


Figure 12.—Gradations varying in percentage passing No. 8 sieve, with medium percentage passing No. 30 sieve.

the gradation chart. But this does not preclude the possibility that there may be other skip gradations of the same maximum size that will exceed the density of gradation No. 3.

Figures 12 and 13 compare data for gradations that vary in the percentage passing the No. 8 sieve. These were selected from previous figures used to illustrate "hump" gradations. They provide the same indications as figure 11. For example, in figure 12, gradation No. 20 plotted further from the maximum density line than gradation No. 10 but had the higher density. The same relationship held for gradations Nos. 18 and 8 in figure 13. Incidentally, gradation No. 20 in figure 12 and gradations Nos. 8 and 18 in figure 13 can be classified as skip gradations as well as "hump" gradations because they plot with slopes flatter between the No. 8 and the No. 30 sieve size than elsewhere.

In reference to the higher density skip gradations in figures 11–13, it is considered important to note that in all cases the right-hand portion of the gradation curve was below the maximum density line. This fact must be taken into account when using the maximum density line as a reference for adjusting skip gradations to provide a lower or a higher density.

Conclusions

The laboratory study covered by this article was limited to data representing 24 different gradations of aggregate of a single maximum size. Only one asphalt and one type of aggregate were used in the mixtures. Based on these limited conditions, the following conclusions are warranted:

1. The new Public Roads gradation chart provides a much more convenient means of studying aggregate gradations than the logarithmic chart now commonly used. The greater convenience results from the fact that maximum density gradations can be represented on the chart by a straight line from a theoretical zero percent passing zero sieve size to 100 percent passing the effective maximum size.

2. This maximum density line constitutes a new design tool, in that it serves as an easily remembered line in comparing different gradations or in adjusting gradations to provide desired voids and stability characteristics.

3. For gradations of the same type of aggregate which plot as smooth curves entirely above or below the maximum density line, those closest to the line will usually represent gradations yielding the lowest voids in the compacted mixture.

4. For gradations of the same type of aggregate which plot as identical curves except for the portion between the No. 8 and the No. 200 sieves, those that show appreciable humps above the maximum density line at about the No. 30 sieve will have higher mineral voids and lower Marshall stabilities than those plotting with lesser humps. Analysis of several problem mixtures from field projects has clearly confirmed this finding and points up the detrimental effect of gradation humps in the finer aggregate sizes.

5. For skip gradations, low mineral voids are associated with curves that stay appreciably below the maximum density line in the right-hand or coarse aggregate zone of the chart.

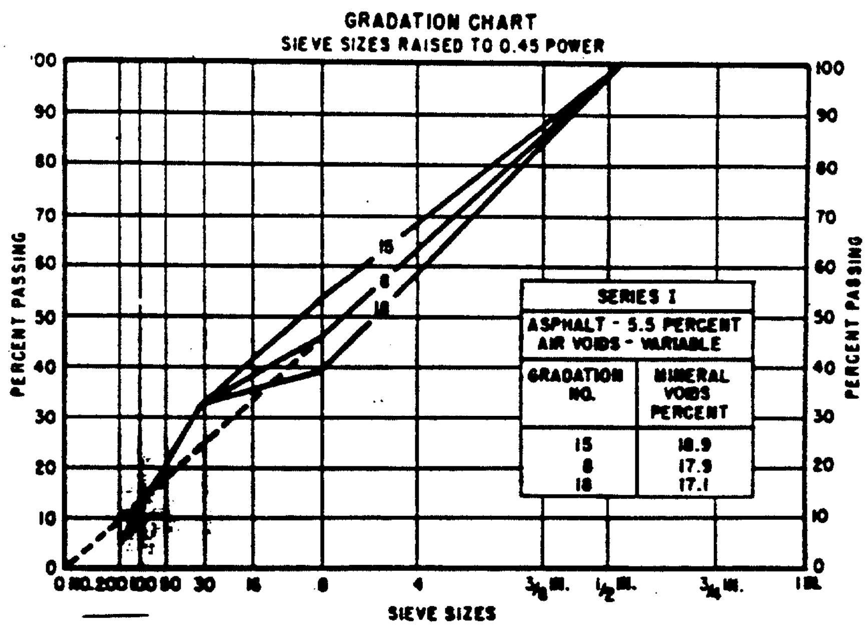


Figure 13.—Gradations varying in percentage passing No. 8 sieve, with high percentage passing No. 30 sieve.

APPENDIX: PROCEDURE AND DETAILS OF PROJECT

Processing aggregate

Table 1 shows the aggregate gradations used in the study and includes values of effective specific gravity which were used in computing voids. The effective specific gravities are rational values determined directly on several of the mixtures by the Rice vacuum saturation procedure.[4]

The aggregate larger than the No. 4 sieve and a portion of that passing the No. 4 sieve and retained on the No. 8 sieve was an uncrushed river gravel. The remainder of the aggregate consisted of sand from the same source and a commercial limestone mineral filler. The amount of mineral filler used varied with the gradation. In all cases 60 percent of the total aggregate passing the No. 200 sieve consisted of limestone dust.

Table 2 gives the apparent and bulk specific gravities of the three stock aggregates. Rational values of apparent and bulk specific gravity of the combined aggregate representing different gradations were not determined.

In preparing the aggregate to be combined to meet the several gradations, the gravel and the sand larger than the No. 8 sieve were accurately separated into 0.525-inch to ¾-inch, ⅜-inch to No. 4, and No. 4 to No. 8 sieve size fractions. Since it is very difficult to obtain clean separations for fine size aggregate in large quantities, no attempt was made to separate the sand into exact sieve size fractions. Instead, it was separated into approximate sizes by a relatively rapid sieving process, and the gradations of the several fractions were then accurately determined and used in computing the correct proportions to provide the desired combined gradations.

[4] *Maximum Specific Gravity of Bituminous Mixtures by Vacuum Saturation Procedure*, by J. M. Rice, in *Symposium on Specific Gravity of Bituminous Coated Aggregates*, Special Technical Publication No. 191, American Society for Testing Materials, June 1956, pp. 43–61.

Preparing mixtures and test specimens

An 85–100 penetration grade asphalt was used in all mixtures. Table 3 gives its test properties.

The mixtures were prepared in a laboratory mixer from aggregate heated to 325° F. and asphalt heated to 300° F. Each batch was just sufficient for one test specimen, which, immediately after being mixed, was compacted in a gyratory mold heated to 200° F. Figure 14 (p. 26) shows the gyratory compactor used in molding the specimens.

The test specimens, 4 inches in diameter and 2½ inches in height, were molded by applying 30 gyrations at a 1-degree angle and under a foot pressure of 100 p.s.i. Previous work by McRae and McDaniel[5] indicated that this procedure produced densities corresponding to those of the 50-blow, hand-compacted Marshall specimen.

Tests performed

The specimens were tested for bulk specific gravity, Marshall stability, and Marshall flow value. Bulk specific gravity was determined by the procedure described in Section 4(a) of AASHO Method T-165. Air and mineral voids, based on effective specific gravity of the aggregate, were computed from the bulk specific gravities.

[5] *Progress Report on the Corps of Engineers' Kneading Compactor for Bituminous Mixtures*, by J. L. McRae and A. R. McDaniel, Proceedings of the Association of Asphalt Paving Technologists, vol. 27, 1958, pp. 357–382.

Table 1.—Gradation and effective specific gravity of aggregate blends

Gradation No.	Percentage passing indicated sieve										Effective specific gravity[1]
	0.525 in.	½-in.	⅜-in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	
1	100	[illegible]	[illegible]	73	58.6	47.3	38.0	30.8	24.9	20.0	2.651
2	100	[illegible]	[illegible]	[illegible]	52.0	39.9	30.6	22.6	18.1	13.9	2.650
3	100	[illegible]	[illegible]	[illegible]	46.0	32.7	24.6	18.0	13.2	9.7	2.649
4	100	[illegible]	94	[illegible]	40.8	28.5	19.8	13.8	9.7	6.7	2.648
5	100	97	[illegible]	56	38.2	24.0	15.9	10.6	7.1	4.7	2.646
6	100	97	[illegible]	51	32.1	20.3	12.8	8.1	5.2	3.2	2.643
7	[illegible]	[illegible]	[illegible]	[illegible]	48.0	40.6	30.6	22.6	12.3	4.7	2.665
8	[illegible]	[illegible]	[illegible]	[illegible]	46.0	38.3	32.6	20.4	11.4	4.7	2.661
9	[illegible]	[illegible]	[illegible]	[illegible]	48.0	36.0	28.6	18.1	10.4	4.7	2.658
10	[illegible]	[illegible]	[illegible]	[illegible]	46.0	33.7	24.6	15.9	9.4	4.7	2.655
11	[illegible]	[illegible]	[illegible]	[illegible]	46.0	32.0	21.6	14.2	8.7	4.7	2.653
12	[illegible]	[illegible]	85	61	43.1	30.1	20.4	13.3	5.4	4.7	2.651
13	[illegible]	[illegible]	[illegible]	[illegible]	53.3	46.0	40.6	24.9	12.3	4.7	2.670
14	100	[illegible]	[illegible]	[illegible]	53.3	42.7	36.6	22.6	12.3	4.7	2.667
15	[illegible]	[illegible]	[illegible]	[illegible]	53.3	41.4	32.6	20.4	11.4	4.7	2.663
16	[illegible]	[illegible]	[illegible]	[illegible]	53.3	[illegible]	28.6	18.1	10.4	4.7	2.660
17	[illegible]	[illegible]	[illegible]	[illegible]	53.3	36.8	24.8	15.9	9.4	4.7	2.657
18	100	[illegible]	84	[illegible]	38.9	36.3	32.6	20.4	11.4	4.7	2.659
19	100	[illegible]	84	[illegible]	38.9	32.0	28.6	18.1	10.4	4.7	2.656
20	100	[illegible]	84	[illegible]	38.9	30.7	24.6	15.9	9.5	4.7	2.653
21	[illegible]	[illegible]	84	[illegible]	38.6	28.4	20.6	13.6	8.5	4.7	2.650
22	100	[illegible]	84	[illegible]	52.0	38.0	24.1	15.6	9.3	4.7	2.657
23	100	97	82	[illegible]	46.0	32.0	21.6	14.2	8.7	4.7	2.653
24	100	97	80	46	40.0	28.1	19.2	12.8	8.1	4.7	2.649

[1] Rational values allowing for gradation and based on the results of several tests by the Rice vacuum saturation procedure.

Table 2.—Physical properties of aggregates [1]

	Gravel		Sand	Limestone mineral filler
	3/4-in. to 3/8-in.	3/8-in. to No. 4		
Apparent specific gravity	2.[illegible]	2.66	2.67	2.71
Bulk specific gravity	2.[illegible]	2.62	2.58	....
Water absorption, percent	1.0	6	1.4	....

[1] AASHO methods T 84 and T 85.

Table 3.—Physical properties of asphalt

Property	Value
Original asphalt:	
Specific gravity, 77°/77° F	1.018
Flash point, C.O.C. ....°F	540
Softening point ....°F	117
Penetration, 77° F, 100 g., 5 sec.	[illegible]
Ductility, 77° F ....cm	220
Bitumen ....percent	99.8
After oven loss test (AASHO T 47)	
Loss ....percent	0.06
Penetration	90
Retained penetration ....percent	98
After thin-film oven test (AASHO T 179)	
Loss ....percent	0.20
Softening point ....°F	132
Penetration	54
Retained penetration ....percent	58
Ductility ....cm	196

Two series of tests were conducted, the results of which are summarized in table 4. The first series was performed on all 24 gradations shown in table 1. All 24 mixtures contained 5.5 percent of asphalt by weight of the aggregate. A total of 72 test specimens, 3 for each of the 24 gradations, was made. The work was done in three rounds, one round of 24 specimens being prepared on each of three different days. The test results for each group of three corresponding specimens from the three rounds were averaged.

The second series of tests was performed on 14 of the 24 gradations. Asphalt contents were computed from the results of the first series of tests to produce air voids in pairs of compacted specimens slightly greater and slightly less than 4 percent so that test results for this second test series could be interpolated for exactly 4-percent air voids. A total of 84 specimens, 3 pairs for each of the 14 gradations, was made. The work was done in 3 rounds, 1 round of 28 specimens for the 14 gradations being prepared on each of 3 different days. The test results for each group of corresponding specimens were averaged.

Table 4.—Physical properties of gyratory compacted gravel mixtures

Gradation No.	1st series of tests: [1] Asphalt, 5.5 percent; [2] air voids, variable					2d series of tests: [1,4] Asphalt, variable; air voids, 4.0 percent [3]				
	Bulk specific gravity	Mineral voids [3]	Air voids [3]	Marshall stability	Marshall flow	Asphalt content [2]	Bulk specific gravity	Mineral voids [3]	Marshall stability	Marshall flow
		Percent	Percent	Pounds		Percent		Percent	Pounds	
1	2.344	16.2	4.2	[illegible]	9	[illegible]	2.347	16.1	1,750	8
2	[illegible]	14.7	2.5	[illegible]	10	[illegible]	[illegible]	15.0	1,310	8
3	[illegible]	14.4	2.1	[illegible]	10	[illegible]	[illegible]	14.8	1,610	8
4	[illegible]	15.1	2.9	[illegible]	9	[illegible]	[illegible]	15.3	1,310	9
5	[illegible]	16.4	4.4	[illegible]	9	[illegible]	[illegible]	16.3	1,010	8
6	[illegible]	17.9	6.2	770	9					
7	[illegible]	18.7	7.0	[illegible]	7	[illegible]	[illegible]	18.2	1,100	7
8	[illegible]	17.9	6.5	[illegible]	8	6.22	[illegible]	17.4	1,110	7
9	[illegible]	17.8	6.4	[illegible]	8	5.97	[illegible]	17.0	[illegible]	8
10	[illegible]	16.8	4.8	[illegible]	9	[illegible]	2.347	16.3	1,190	7
11	2.318	17.2	[illegible]	[illegible]	8					
12	[illegible]	16.7	4.7	1,070	9	[illegible]	2.344	16.8	1,200	9
13	[illegible]	20.5	9.0	[illegible]	7					
14	[illegible]	[illegible]	7.9	710	7	[illegible]	2.317	18.8	[illegible]	8
15	2.277	18.9	7.2	[illegible]	7					
16	2.276	19.0	7.3	[illegible]	8					
17	[illegible]	18.6	6.9	[illegible]	8	[illegible]	[illegible]	17.7	[illegible]	8
18	[illegible]	17.1	5.2	[illegible]	7	5.70	[illegible]	16.5	1,070	7
19	[illegible]	[illegible]	4.1	[illegible]	8					
20	[illegible]	15.8	2.8	[illegible]	9	[illegible]	[illegible]	15.8	1,240	8
21	[illegible]	16.1	4.1	[illegible]	8					
22	[illegible]	17.7	[illegible]	[illegible]	8					
23	[illegible]	16.8	4.3	[illegible]	8					
24	[illegible]	15.1	2.9	[illegible]	8					

[1] Averages of 3 values, 1 per round for 3 rounds of tests.
[2] By weight of aggregate.
[3] Based on effective specific gravity of the aggregate.
[4] Interpolated values from results at 2 asphalt contents.

CHAPTER 5

PAVEMENT DRAINAGE

5.1 Pavement Design Acceptance, Consideration of Drainage, Memorandum, T. D. Larson, February 6, 1992.

- Technical Guide Paper, 90-01, Subsurface Pavement Drainage, 1990.

5.2 Longitudinal Edgedrains, Concrete Pavement Drainage Rehabilitation, State of Practice Report, Experimental Project No. 12, April 1989.

5.3 Permeable Base Design and Construction, January 1989.

5.4 Case Study, Pavement Edgedrain, TA 5040.14, June 8, 1989.

5.5 Subsurface Drainage of Portland Cement Concrete Pavements; Where Are We? December 1991.

5.6 Western States Pavement Subdrainage Conference, August 10, 1994.

5.7 Drainable Pavement Systems, Demonstration Project 87, April 06, 1992.

5.8 Effectiveness of Highway Edgedrains, Concrete Pavement Drainage Rehabilitation, State of Practice Report, Experimental Project No. 12, April 14, 1993.

5.9 Maintenance of Pavement Edgedrain Systems, March 21, 1995.

5.10 Pavement Subsurface Drainage Activities, December 16, 1994.

U.S. Department
of Transportation

Federal Highway
Administration

Memorandum

Subject: Pavement Design Acceptance
Consideration of Drainage

Date: February 6, 1992

From: Administrator

Reply to Attn of: HNG-42

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Consideration of drainage is recognized as one of the important factors in pavement design. However, inadequate subsurface drainage continues to be identified as a major cause of pavement distress. During the last 10 years, significant strides have been made in the development of positive drainage systems for new and reconstructed pavements. In addition, there has been major product development of materials which can be used for retrofit longitudinal edgedrains. The attached Technical Guide Paper 90-01 provides state-of-the-practice guidance on the design, construction, and maintenance of subsurface drainage systems.

The developments in technology for permeable bases and longitudinal edgedrains make the provision of positive drainage of the pavement section possible and affordable. Accordingly, to be acceptable to the Federal Highway Administration, each State's pavement design procedure must include a drainage analysis for each new or reconstructed pavement section. Where the drainage analysis or past performance indicates the potential for reduced service life due to saturated structural layers or pumping, the design must include positive measures to minimize that potential.

Each division office is to evaluate the State's current design procedures to determine if pavement drainage is being adequately addressed. Where deficiencies are noted, the division will work with the State to accomplish needed changes by August 1, 1992.

The Pavement Division is available to provide technical support and guidance to achieve these actions. I have directed the Pavement Division to report to me monthly on progress. This will require a report from each Region to the Pavement Division (HNG-40) on the first of each month, until acceptable design procedures that consider pavement drainage are in operation in each State. I ask that each of you lend your personal support to this important initiative to improve pavements.

T. D. Larson

Attachment

TECHNICAL PAPER 90-01

TECHNICAL GUIDE PAPER

ON

SUBSURFACE PAVEMENT DRAINAGE

FEDERAL HIGHWAY ADMINISTRATION

OFFICE OF ENGINEERING

PAVEMENT DIVISION

OCTOBER 1990

INTRODUCTION

Water in the pavement structure is a recognized cause of pavement distress, particularly in portland cement concrete (PCC) pavements. Many highway agencies are retrofitting drainage on existing pavements and including free draining bases on new or reconstructed pavements.

This paper is based on the observation of many pavement structure drainage installations and a review of current research. It represents the current state-of-the-practice in design practices for draining the pavement structure. Design and construction of permeable bases and retrofit longitudinal edgedrains are discussed.

This paper was originally developed as a Technical Advisory (TA) on subsurface pavement drainage. However, because of the large amount of experimentation and research underway in pavement structure drainage, it was decided to delay issuance of the TA. The purpose of this paper is to provide interim guidance until the TA is issued. If there are any questions concerning this paper, or if you wish to offer any information relating to permeable bases or retrofit longitudinal edgedrains, please send them to the Pavement Division (HNG-40) or contact John Hallin at (202) 366-1323.

Technical Guide Paper October 25, 1990

Subsurface Pavement Drainage

1. PURPOSE. To provide guidance for the design, construction, and maintenance of subsurface drainage systems for the removal of surface water that infiltrates the pavement structure. The procedures and practices outlined below are directed primarily towards high-type portland cement concrete (PCC) pavements; however, the principles and procedures may be applicable to high-type asphalt concrete (AC) pavements as well.

2. DEFINITIONS

a. Permeability - the capacity of a material to conduct or discharge water under a given hydraulic gradient.

b. Coefficient of permeability (K) - a measure of the rate at which water passes through a unit area of material in a given amount of time under a unit hydraulic gradient.

c. Permeable Base - a base that is designed and constructed with the intent to rapidly drain moisture that infiltrates the overlying pavement structure.

3. BACKGROUND

a. The pavement structural section is a costly element of the highway system, and its premature failure is of major concern. Among the reasons cited for pavement failures, inadequate base drainage has been identified as a nationwide problem, particularly for PCC pavements. The *AASHTO Guide for Design of Pavement Structures (1986)* includes drainage as an essential element of pavement design.

b. One of the primary distress mechanisms observed on PCC pavements is pumping. The conditions which cause pumping are free water, voids in the pavement section, repeated heavy wheel loads, and an erodible base. Unfortunately, these four conditions are present on the vast majority of PCC pavements designed and constructed to date.

c. The primary source of free water is infiltration through cracks and joints in the pavement. A major source of infiltrated moisture is the longitudinal pavement/shoulder joint, particularly when AC shoulders are used. Water also enters the pavement section from shallow ditches and medians.

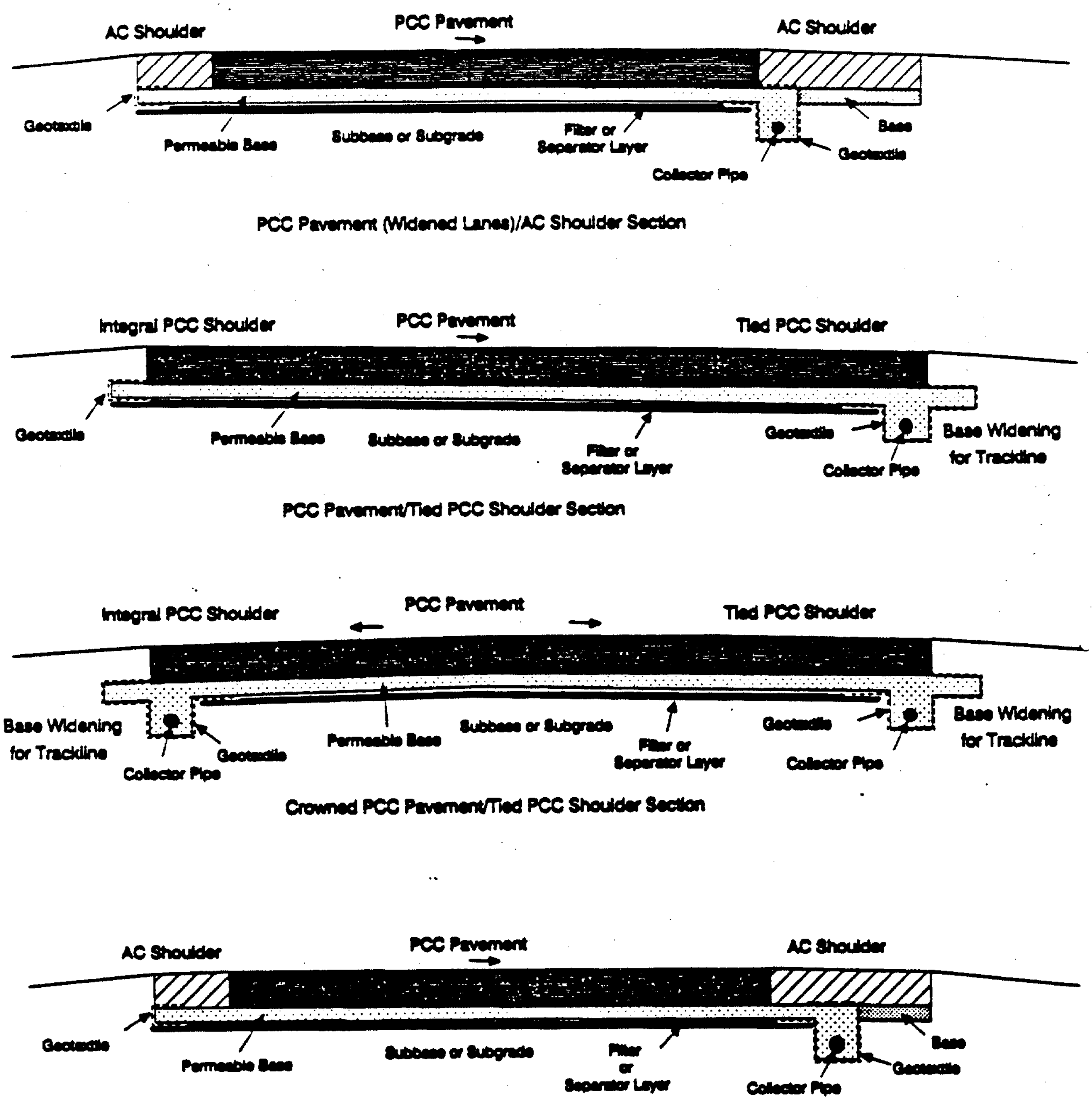
d. To reduce moisture infiltration into the pavement structure, two approaches are recommended. First, all pavement joints and cracks should be sealed to reduce infiltration. While a pavement cannot be completely sealed, properly sealed joints can significantly reduce the amount of water entering the pavement structure. Second, pavement structure drainage systems should be used to remove free water as quickly as possible.

e. Adequate pavement and shoulder cross-slope are important drainage features. In addition, proper joint design (including tiebars and joint sealing) and adequate roadside ditch depth are important. Tiebars help prevent joints from separating and allowing water to infiltrate. The use of tied PCC shoulders provides a tighter and easier to seal joint which can reduce the amount of infiltration.

4. DESIGN OVERVIEW

a. Drainage Policy. The FHPM on *Pavement Management and Design Policy (6-2-4-1)* states FHWA's position on pavement structure drainage. State highway agencies (SHA's) are encouraged to perform a drainage analysis for each new, rehabilitated, or reconstructed pavement design. Designs should include methods to minimize the potential for reduced service life due to saturated structural layers.

b. Positive Drainage for New and Reconstructed Pavements. For new construction and reconstruction of PCC pavements, positive drainage is strongly recommended. Positive drainage consists of three elements: 1) a permeable base to provide rapid drainage of free water that may enter the pavement structure; 2) a longitudinal edgedrain collector system to convey accumulated water from the permeable base; and 3) a filter-separator layer to prevent migration of fines (minus 200 material) into the permeable base from the subgrade, subbase, or shoulder base material. Filter material should not be placed between the pavement and the permeable base, nor between the permeable base and the edgedrain. Unrestricted flow to the permeable base and the edgedrain must be ensured. The filter-separator layer, whether aggregate or geotextile, must be properly designed to prevent migration of fines and possible base contamination. These elements are shown in Figure 1.



PCC Pavement (Widened Lanes)/AC Shoulder Section

PCC Pavement/Tied PCC Shoulder Section

Crowned PCC Pavement/Tied PCC Shoulder Section

PCC Pavement (Widened Lanes)/AC Shoulder Section
(Edgedrain Installed After PCC Paving)

Figure 1. Permeable Base Sections

c. Positive Drainage for Rehabilitated Pavements. Since most existing PCC pavements have been designed and constructed with impermeable bases, rapid lateral drainage of infiltrated water from the base is not practical. However, retrofit longitudinal edgedrains can rapidly drain water that has infiltrated the pavement structure and migrated to the slab/base interface particularly when AC shoulders are used. Edgedrains placed adjacent to the pavement/shoulder joint can intercept this moisture and significantly shorten the time that free water is present in the interface, thereby minimizing the potential for pumping.

d. AASHTO Drainage Coefficient

(1) The *AASHTO Guide for Design of Pavement Structures (1986)* attempts to recognize the effects of drainage on pavement design. The guide uses a drainage coefficient to model the effect of drainage in determining the thickness of PCC pavement. Of all the parameters in pavement thickness design, pavement thickness is most sensitive to changes in the drainage coefficient. **However, it must be emphasized that a thicker pavement will not compensate for poor drainage.**

(2) **A positive drainage system, including a permeable base, a filter layer, and longitudinal edgedrains, should be provided to ensure good drainage. Once adequate drainage has been provided, pavement thickness can be determined using a drainage coefficient of 1.0 or greater.**

e. Drainage Analysis

There are generally two types of pavement subsurface design criteria used in design. They are: 1) criterion for the time of drainage of the base beginning with the saturated condition and continuing to an established acceptable level, and 2) an inflow-outflow criterion, by which drainage occurs at a rate greater than or equal to the inflow rate, thus avoiding saturation. It should be noted that the drainage layer design is based only on the infiltration of water from the surface. Normally, other sources of water to the drainage layer would be minor and normally are not a consideration in the design of the permeable base. Should ground water be present in any substantial quantities, special provisions should be made to intercept and drain the water before it reaches the permeable base. The permeable base is expected to aid in the drainage of water in the subbase and subgrade caused by frost action, but this volume of water is generally not considered in computing the design water inflow.

(1) (a) One method of drainage analysis is to examine the gradation of the base material. Estimates of permeability and filter-separator criteria can be made by analyzing the gradations of the base and subgrade material. By comparing the gradation of the sample material to the gradation of a material whose permeability has been determined, the permeability of the sample material can be estimated.

(b) Material permeability can also be determined in the laboratory by the constant head permeability test or the falling head permeability test. The tests should be performed in accordance with *AASHTO T 215, Permeability of Granular Soils (Constant Head)* and the *U.S. Army Corps of Engineers, Engineer Manual (EM 1110-2-1906), Laboratory Soils Testing, Appendix VII, Permeability Tests (Falling Head).*

(c) A method of determining the in-situ permeability of a base material is to use the field permeability testing device (FPTD) as described in the report, *Determination of the In Situ Permeability of Base and Subbase Courses*. This device determines the in-situ permeability of a material by measuring the velocity of flow between two points. The FPTD's upper and lower limits are 28,000 feet per day (10 centimeters per second) and 0.28 feet per day (10^{-4} centimeters per second), respectively. Average coefficients of permeability determined in field testing of the FPTD have shown good correlation with average laboratory permeabilities.

(d) Field percolation tests are another method for evaluating the ability of the existing base material to drain. In a percolation test, a hole is cored down to the base and filled with water. Observation of the water level in the hole over time will give an indication of the base material's ability to drain. Caution must be exercised with this method to ensure that percolating moisture is confined to the particular layer being tested. If moisture is allowed to escape along an interface, through voids, or through an adjacent material, the percolation test can give a false indication. In addition, it is important to ensure that the top of base is not clogged during coring.

(2) Edgedrain Hydraulic Capacity. In any drainage analysis, the hydraulic capacity of the edgedrain should be determined to establish the outlet pipe spacing.

(a) Permeable Base Edgedrain. The hydraulic capacity of a longitudinal edgedrain to drain a permeable base should be based on draining free water within the pavement structure within 2 hours of rain cessation. In most cases, a conventional partially geotextile wrapped trench with a 4-inch diameter pipe and backfilled with permeable material will provide excess hydraulic capacity.

(b) Non-Drainable Base and Retrofit Edgedrain. Determining the hydraulic capacity of the edgedrain is not as critical with longitudinal edgedrains on pavements with non-draining or very slow draining bases. Drains should be sized to remove the volume of water occupying the voids in the pavement section once rain has stopped. The purpose of a longitudinal edgedrain in these cases should be to drain free water in the slab/base interface within 2 hours of rain cessation. The capacity should be calculated to satisfy this criteria and flow rates across geotextiles should permit this. Because of the potential for blinding (soil particles blocking the geotextile openings) or clogging (soil particles are trapped within the pore openings, thus reducing the permeability of the geotextile) it is extremely important to properly size the geotextile for the particular soil type and percentage of fines.

f. Outflow Design. To ensure rapid drainage of accumulations of water within a permeable base structural section and to protect the component parts of a drainage system, the outflow capacities of the system should increase in the direction of flow, starting at points of entry and progressing through the base drainage layer, collector pipes, and outflow pipes. In essence, when progressing along possible paths of flow in drainage systems, the water removing capabilities should increase, never decrease, in the direction of flow. This is particularly important with respect to pipe drains and the backfill surrounding them.

g. Filter Design.

(1) The function of any filter is to provide both drainage and filtration. The filter must allow water to pass (drainage) with minimal head loss while retaining soil particles (filtration). It must also enable the creation of a natural filter in the neighboring soil to prevent piping (loss of finer soil particles through the filter leaving larger soil voids behind). For a geotextile to effectively perform as a

filter in a geotextile drainage system, it must remain free-draining by having opening characteristics compatible with the surrounding soil. In some cases, the geotextile is required to prevent migration of fine grained soils without clogging. In complete clogging, the fabric's permeability is reduced to less than that of the soil. In other cases, some fine-grained soils may be required to pass through the geotextile to prevent blinding. In blinding, particles coat the surface of the geotextile such that the permeability is substantially reduced. In any case, some loss of soil particles through the filter during its early life takes place. As fine soil moves through the geotextile, larger particles may combine to bridge the appertures of the geotextile. Immediately behind this bridging zone is another zone (soil filter zone) consisting of soil particles whose permeability decreases with distance from the geotextile. Thus, the choice of a correct geotextile is critical to formation of a stable and effective soil filter. Geotextiles, like graded filters, require engineering design. Unless proper fabric piping resistance, clogging resistance, and constructability strength requirements are specified, it is doubtful that the desired results will be obtained. Construction installation and monitoring must also be provided to ensure that the materials have been installed correctly.

(2) The major criteria considered for a geotextile drainage/filtration application include: 1) soil retention (piping resistance), 2) permeability, 3) clogging potential, 4) chemical composition requirements/considerations, and 5) constructability and survivability requirements.

(3) As with other elements of highway design, geotextiles must be engineered. The geotextile should have a permeability at least several times greater than the aggregate base/subbase so that water can drain freely from it. Geotextiles must also retain the upstream soil. The apparent opening size (AOS) (or equivalent opening size (EOS)) -- AOS and EOS are equivalent terms -- is defined as the U.S. standard sieve number that has openings closest in size to the openings in the geotextile. If given as the equivalent sieve size opening in millimeters, it is referred to as the 95 percent opening size or O_{95}. The AOS of the geotextile should be selected to prevent fines from piping through the filter and clogging the permeable material and leaving voids behind. The appropriate geotextile AOS can be determined by the following criteria adopted by Task Force 25 (refer to Appendix B, Table 1).

1. For a soil with 50 percent or less particles by weight passing the No. 200 sieve, the AOS of the geotextile should be equal to or greater than the No. 30 sieve (i.e., $O_{95} \leq 0.60$ mm).

2. For a soil with more than 50 percent particles by weight passing the No. 200 sieve, the AOS of the geotextile should be equal to or greater than the No. 50 sieve (i.e., $O_{95} \leq 0.30$ mm).

(4) It should be noted that there is no way to prevent a filter adjacent to a material with a high percentage of fines from eventually clogging. If there are no voids or if the voids are small, the filter won't clog up as rapidly and the filter will function for a longer period of time. If, however, voids are present between the material to be drained and the filter, soil particles are provided an opportunity to go into suspension and will eventually clog the filter. Likewise, geotextiles need intimate contact with the material to be drained. A filter placed along a pavement with voids between the slab and the base or between the geotextile and the pavement base would be comparable to this situation.

(5) Generally, nonwoven needle-punched geotextiles are better for pavement drainage applications than heat-bonded geotextiles. Woven or slit-film geotextiles should not be used.

5. PERMEABLE BASES

a. Permeable Base Design. Most existing design methods have relied on the practice of building pavements strong enough to resist the combined effects of load and water. However, they do not always account for the potential destructive effects of water within the pavement structure. As a result, increased emphasis is needed to exclude water from the pavement and provide rapid drainage of any moisture that infiltrates the pavement surface. Permeable bases provide rapid drainage of this moisture. In theory, a properly designed and constructed permeable base will rapidly drain water that infiltrates the pavement surface and not allow destructive high pressures to build up beneath the pavement.

(1) To overcome moisture related distresses in PCC pavements, many SHA's are now using permeable bases. There are two types of permeable bases; unstabilized and stabilized.

(2) The combination of base thickness and permeability should be capable of rapidly draining the design flows and preventing saturation of the base. The time period that free water is present within the pavement structure should

be as short as possible, desirably less than 2 hours following the cessation of precipitation.

(3) A longitudinal edgedrain collector system with outlet pipes should be provided to ensure positive drainage. The outlets must be discharged into gutters or drainage ditches or connected to culverts or drainage structures. Daylighting the permeable base layer is not effective in draining the base since it is subject to clogging from roadway debris and vegetation. In addition, daylighted layers may allow silty material or storm water from ditches to enter the pavement structure.

b. Base Material. Both unstabilized and stabilized permeable base material should consist of a hard, durable, crushed, angular aggregate with essentially no fines (minus No. 200 sieve material). A permeable base consisting of crushed aggregate meeting the gradation requirements noted in this Technical Guide Paper will provide sufficient stability on which construction equipment such as dump trucks, transit trucks, and tracked pavers can operate, as well as provide good slab support. The permeable base material gradation should have good aggregate interlock. To prevent the aggregate from degrading and generating fines during construction, the material for the permeable base should also be hard and durable. Also, consideration should be given to construction of a test section to ensure the material will be stable under construction traffic. Recommended gradations of the base material vary depending on whether the material is stabilized or unstabilized. A coefficient of permeability greater than 1000 feet per day is recommended.

(1) Unstabilized Permeable Base

(a) Unstabilized permeable bases utilize an open-graded aggregate material. Most SHA's that use unstabilized permeable bases have developed a gradation that represents a careful trade-off of constructability, stability, and permeability. Unstabilized permeable base materials contain more smaller size aggregate to provide stability through aggregate interlock. The use of more smaller sized aggregate results in lower permeability. To provide good stability for paving equipment, unstabilized permeable base aggregate should be composed of 100 percent crushed stone. Where 100 percent crushed stone with an LA abrasion index of 30 or less is not available, consideration should be given to stabilizing the aggregate with asphalt cement or portland cement. If a material other than a crushed stone is used, other gradations and/or stabilization will need to be investigated.

(b) Below is a gradation for unstabilized permeable material which provides satisfactory permeability (greater than 1000 feet per day) and excellent stability to carry construction equipment. The following is an example of a gradation that has worked:

Sieve Size	Percentage Passing
1 1/2"	100
1"	95-100
1/2"	60-80
No. 4	40-55
No. 8	5-25
No. 16	0-8
No. 50	0-5

(Note: Wet-washed, dry-sieved)

(2) Stabilized Permeable Base

(a) Stabilized permeable bases utilize open-graded aggregate that has been stabilized with asphalt cement or portland cement. Many SHA's require 90 to 100 percent two-crushed faces with a maximum LA Abrasion wear of 40 to 45 percent. Material passing the No. 8 sieve has been virtually eliminated, and the resulting coefficient of permeability is usually much greater than 3,000 feet per day. Stabilizing the permeable base provides a stable working platform without appreciably affecting the permeability of the material. Stabilization is accomplished by using only enough asphalt or cement paste to coat the aggregate. Therefore, its the gradation of the permeable base material that will determine how much stabilizer to use. Its very important that the voids are not filled by excess stabilizer.

1. The stabilization material predominantly used is asphalt cement (AC-20) at 2 to 2 1/2 percent (by weight) for the very open-graded materials such as the AASHTO No. 57 stone. Higher asphalt cement percentages are required when a less open-graded material is used. For example, New Jersey's asphalt cement stabilized permeable base gradation shown below requires 3 percent asphalt cement to coat the aggregates. For additional asphalt stabilized permeable base stability, a stiffer asphalt cement, such as an AC-40, should be used. It should be noted that if AC-40 is used the aggregate should be heated to 275 to 325 degrees Fahrenheit to stiffen the asphalt cement.

2. Portland cement at 1 1/2 to 3 bags per cubic yard has also been used. As with asphalt cement stabilized permeable base, the amount of portland cement per cubic yard will depend on the voids and surface area of the aggregate in the permeable material. For example, California uses not less than 282 pounds of portland cement per cubic yard with a water-cement ratio of 0.37. The permeability of this material is approximately 4,000 feet per day. Whereas Wisconsin with a more open material (permeability approximately 10,000 feet per day) has found that 200 pounds of portland cement per cubic yard and a water-cement ratio of 0.37 provides adequate strength, durability, and stability.

(b) Several SHA's use the AASHTO No. 57 gradation for their stabilized permeable base. This gradation and four other stabilized permeable gradations are as follows:

	Percentage Passing				
	No. 57	California		Wis.	New Jersey
Sieve Size	AC/PC Stab.	AC Stab.	PC Stab.	PC Stab.	AC Stab.
1 1/2"	100	-	100	-	-
1"	95-100	100	86-100	-	100
3/4"	-	90-100	X±22	90-100	95-100
1/2"	25-60	35-65	-	-	85-100
3/8"	-	20-45	X±22	20-55	60-90
No. 4	0-10	0-10	0-18	0-10	15-25
No. 8	0-5	0-5	0-7	0-5	2-10
No. 10	-	-	-	0-5	-
No. 16	-	-	-	-	2-5
No. 200	0-2	0-2	-	-	*
Est. "K" (feet per day)	20,000	15,000	4,000	10,000	1,000

("X" is the gradation which the contractor proposes to furnish for the specific sieve size).

(* Add 2 percent (by weight of total mix) mineral filler).

Its important to note that California uses different gradations for their stabilized permeable bases. The AC stabilized gradation is more open (30 percent voids) and has a high crushed content requirement, whereas the PC stabilized gradation is less open (14 percent voids) and has no crushed content requirement.

c. <u>Base Thickness and Width</u>. A minimum permeable base thickness of 4 inches is suggested when the above gradations are used. This thickness should be adequate to overcome any construction variances and provide an adequate hydraulic conduit to transmit the water to the edgedrain collector system. The permeable base should be placed 1 to 3 feet outside the edge of the pavement to provide a stable trackline for the paver (see Figure 1).

d. <u>Filter-Separator Layer</u>

(1) A filter-separator layer must be provided between the permeable base and the subbase/subgrade to prevent subgrade fines from infiltrating and contaminating the permeable base, to provide a working platform for construction equipment, and to provide support for the permeable base and pavement. Generally, a minimum of 4 inches of dense-graded aggregate base is used. Because very little upward flow of water is expected from the subgrade, the permeability criteria for filter layer design does not apply. Either aggregate or a geotextile can be used. However, a filter-separator layer over stabilized subbases/subgrades may not be needed provided the stabilized material is not subject to saturation or high pressures for an extended period of time. An asphalt prime coat placed on the stabilized subbase/subgrade would provide additional protection. Although, a geotextile is generally more costly than 4 inches of dense-graded aggregate base, there may be instances where sufficient aggregate is not available and a geotextile may be cost-effective.

(2) The following are recommended criteria for the design gradation of the filter-separator layer. Both the filter-separator layer/subgrade and the permeable base/filter-separator layer interfaces must be considered. The gradation of the filter-separator must meet the requirements for the filter-separator layer/subgrade interface as listed below:

<u>EQ. 1</u> D_{15} (Filter-Separator) $\leq$ 5 D_{85} (Subgrade)
[Separation requirement]

<u>EQ. 2</u> D_{50} (Filter-Separator) $\leq$ 25 D_{50} (Subgrade)
[Uniformity criteria for piping resistance]

where the D_x is the size at which "X" percent of the particles, by weight, are smaller than that size.

Similarly, the filter-separator layer must meet the requirements for the permeable base/filter-separator layer interface as listed below:

<u>EQ. 3</u> D_{15} (Base) $\leq$ 5 D_{85} (Filter-Separator)
[Separation requirement]

<u>EQ. 4</u> D_{50} (Base) $\leq$ 25 D_{50} (Filter-Separator)
[Uniformity criteria for piping resistance]

Plotting the results of these equations on a gradation chart eases the determination of the gradation of the filter-separator layer. An example problem illustrating the design is provided in Appendix A.

Also, it is recommended that the filter-separator layer have a maximum of 12 percent passing the No. 200 sieve to ensure a dense-graded material without excess fines increasing the potential for loss of support or contamination of the permeable base.

In addition, to ensure that the filter-separator layer is stable the following requirement is also recommended:

$$20 \leq \text{Coefficient of Uniformity} \leq 40$$

$$\text{where Coefficient of Uniformity} = \frac{D_{60}\ \text{(Filter)}}{D_{10}\ \text{(Filter)}}$$

The term coefficient of uniformity (CU) is an indication of the grading of a material. For example, a uniform (one-size) material will have a small CU because the the size of the D_{60} material is very similar in size to that of the D_{10}. Because it consists primarily of one-size material, it contains insufficient fines to fill the voids between the larger particles and consequently it will have an open, more porous structure despite compaction. As a result it will be more easily displaced under load and have less supporting power. The most uniform granular material commonly encountered in engineering is standard Ottawa sand, which has a CU of approximately 1.1. Conversely, a well-graded material will have a large CU because the D_{60} will be much larger than the D_{10}. A well-graded dense aggregate base material plotted on the maximum density line will have CU of between 50 and 60. A well-graded material is relatively stable, can readily be compacted to a very dense condition, and will develop high shear resistance and bearing capacity.

In most cases, a 4-inch dense-graded aggregate subbase will meet the filter-separator layer requirements for both the filter-separator layer/subgrade and the permeable base/filter-separator layer interfaces. In addition, 4 inches of dense-graded aggregate subbase meets the CU criteria for stability providing an excellent working platform for construction of the permeable base.

(3) Although not generally recommended, some SHA's use a geotextile instead of an aggregate filter-separator layer. The principal advantage of the geotextile is uniform installation. The geotextile should have enough strength to survive the construction phase. Care should be used in placing the geotextile so that it is not damaged during construction. Base course materials must be placed so that the geotextile is not damaged. Slit-film or most woven geotextiles should not be used as they do not prevent fines from pumping through the geotextile. Geotextiles should meet the material requirements of the AASHTO-AGC-ARTBA Task Force 25 Specification shown in Appendix B.

e. <u>Construction Considerations</u>

(1) Construction of unstabilized permeable bases requires care since these bases are subject to displacement by construction traffic. Unstabilized permeable bases are also subject to segregation of the material during placement. The addition of 2 to 3 percent water by weight of aggregate reduces the potential for segregation during hauling and placement. Care must also be exercised during construction operations to prevent contamination of the permeable base.

(2) Stabilized permeable bases have sufficient stability for paving equipment and construction traffic. However, because the material is open and must remain so to function properly, it is extremely important to prevent contamination of the permeable base from fine-grained materials. Also, the grade of the stabilized permeable base is more difficult to modify once it has been placed and compacted/consolidated.

(3) SHA's should be encouraged to restrict construction traffic from the permeable base. If the working area is restricted and construction equipment must travel on the permeable base, a stabilized permeable base should be considered.

f. <u>Compaction of Permeable Base</u>

(1) <u>General</u>. Compaction or consolidation of the permeable base material is important. The conventional approach of requiring a fixed percent of a standard or target density may not be applicable. The purpose of compacting a permeable base is to seat the aggregate. A level of consolidation should be specified which results in no appreciable displacement of the base following compaction.

(2) Unstabilized and Asphalt Stabilized. Most SHA's specify one to three passes of a 4 to 10 ton steel-wheeled roller. Over rolling can cause degradation of the material and a subsequent loss of permeability. Caution should be exercised when using vibratory rollers to compact permeable bases, as they can cause degradation, over densification, and a subsequent loss of permeability.

(3) Portland Cement Stabilized. Two methods of compacting or consolidating portland cement stabilized permeable base have been commonly used; 1) rolling consisting of 1 to 3 passes of a 4 to 10 ton steel-wheeled roller (non-vibratory) and 2) vibration using vibrating screeds or vibrating plates.

g. Curing of Portland Cement Stabilized Permeable Base. Curing is another aspect that is of concern with portland cement stabilized permeable bases. Covering the permeable base with polyethylene sheeting for 3 to 5 days is one method used by a few SHA's. A fine water mist cure applied to the portland cement stabilized permeable base several times the day after placement has been used by a few SHA's as well. The method that provides the desired strength and durability to allow for paving on the portland cement stabilized permeable base should be used. A SHA may want to construct a test strip of portland cement stabilized permeable base to determine which curing method to employ as well as which method of compaction/consolidation to use.

6. LONGITUDINAL EDGEDRAINS

a. Edgedrain Design

(1) General. Design considerations will vary for longitudinal edgedrains depending on whether they are used in a new or reconstructed case (for draining permeable base pavements) or in a retrofit case (for draining non-permeable base pavements). The amount of moisture to be drained and the presence or lack of fines and the condition of the base/subbase are important considerations in edgedrain design.

(2) Edgedrain for Permeable Bases. When a permeable base is used, all runoff that enters the pavement section should quickly drain to the edgedrain. The trench backfill material and edgedrain pipe must have adequate capacity to handle the flows. Erosion of fines should not be a problem since the base should contain very little erodible fine material. A longitudinal edgedrain collector system that is open to the permeable base should be used. A geocomposite fin drain is not recommended to drain a permeable base.

(3) Edgedrain for New Non-Permeable Base Pavement. Edgedrains installed on a new non-permeable base should function longer

than retrofit longitudinal edgedrains and are more likely to improve pavement performance. This is because the pavement and base are in excellent condition and erosion of fines should be minimal as a result of small/few voids.

(4) Retrofit Longitudinal Edgedrains

(a) For retrofit longitudinal edgedrains, a field survey should be performed on the existing pavement to determine its condition and drainage features. It is imperative that the existing pavement structure be no more than moderately distressed (i.e., less than 5 percent of the right lane requiring full depth replacement). Studies have shown that if the pavement is severely cracked or has broken slabs, retrofit edgedrains may not be an appropriate rehabilitation technique unless combined with a technique which also increases the structural capacity of the pavement such as an overlay.

(b) In any design analysis of retrofit longitudinal edgedrains, there are two steps that must be followed to determine if the proposed design will accomplish its goal of pavement drainage; 1) identify the source of moisture, and 2) evaluate the erodibility of base material.

1. The first step is to identify the source of moisture that the edgedrains will drain. Retrofit longitudinal edgedrains will drain water that enters the pavement/shoulder joint and any water that infiltrates the PCC pavement slab and collects in voids along the slab/base interface or the base/subgrade interface. This is free water that follows the path of least resistance and is strongly influenced by the effects of gravity. Any water that enters and ultimately saturates the dense graded base may take days or weeks to be drained by the retrofit longitudinal edgedrain.

2. The second step is to evaluate the erodibility of the base material. If the base tends to have 15 to 20 percent or more fines (minus 200 sieve material), it will probably be highly erodible. A geotextile around the drain will not prevent fines from being eroded from the base material. The geotextile controls what happens to the fines after they migrate to the edgedrain. The AOS of the geotextile determines the size of the soil particles that will be retained and those that will pass through the geotextile. The

selection of the AOS for soils with a high percentage of fines becomes a trade-off between allowing the fines to pass through the geotextile and clogging the drain and preventing the fines from passing and clogging and/or blinding the geotextile. If an excessive amount of fines are eroding from the base, retrofit longitudinal edgedrains will not be effective in extending the pavement life and may actually be detrimental by carrying eroded fines away.

(5) Adequate Relief. For both the permeable base and retrofit cases, the cross section of the highway surface must have sufficient relief to provide positive drainage to the roadside ditches. Subsurface and surface drainage must be coordinated. If sufficient relief does not exist, lateral outlet pipes carried out to the ditch may not be feasible and an enclosed drain pipe system may have to be constructed. In addition, shallow ditches result in the water being closer to the pavement structure than with deep ditches.

(6) Transition from Edgedrain to Outlet. The transition from the edgedrain pipe to the lateral outlet pipe should be gradual to facilitate cleaning. Radii of 2 to 3 feet for pipe bends should be used. The radii should permit the use of jet rodding or cleaning equipment. Tee's should not be used on conventional trench/pipe edgedrains. Some SHA's incorporate cleanouts and/or vents into their edgedrain system to improve flow and to facilitate cleaning.

b. Longitudinal Edgedrain Types

(1) Pipe Edgedrain. Conventional pipe edgedrains have a relatively high hydraulic capacity and can be maintained. Retrofit pipe edgedrains should be used with caution when the existing base has more than 20 percent minus 200 sieve material. The edgedrain should be large enough to allow placement of and compaction around a 3 to 6 inch pipe laid in the bottom of the trench which has been partially wrapped with a geotextile and backfilled with a permeable coarse aggregate material. Figure 2 shows the suggested edgedrain configuration. An aggregate trench without a pipe conduit is not recommended because of the much smaller hydraulic capacity and inability to be cleaned. Because the geotextile serves as a filter layer, the permeability of a geotextile must meet the requirements for filter layers noted in section 6.f.(4).

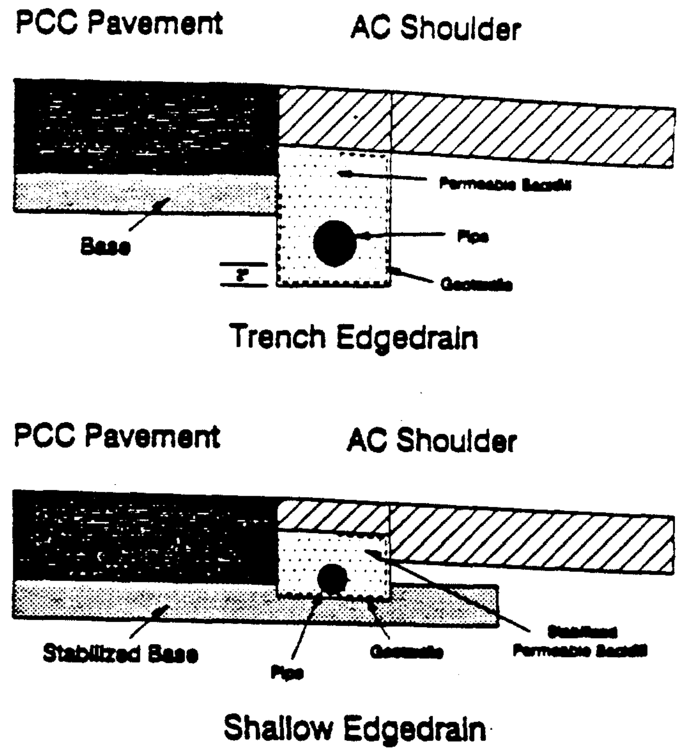


Figure 2. Retrofit Pipe Edgedrains

(2) <u>Geocomposite Fin Drains</u>

(a) A geocomposite fin drain consists of a plastic core, usually rectangular shaped, surrounded by a geotextile. The geotextile retains the soil particles while allowing the water to drain into the core. The plastic core provides the structural capacity and acts as a conduit for the water. Many different types of proprietary geocomposite fin drains are commercially available.

(b) The primary advantage of geocomposites is the ease of installation. Since the trench width is usually only 4 to 5 inches and excavated material is used to backfill the trench, installation costs can be reduced. However, the long-term performance of geocomposites is under evaluation. A typical geocomposite fin drain installation is shown in Figure 3. Geocomposite fin drains should be used with caution when the existing base has more than 15 percent minus 200 sieve material. There is a greater potential for plugging of the core under this condition.

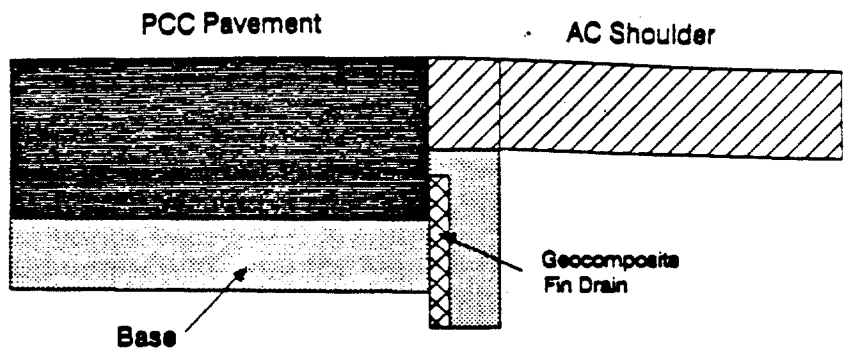


Figure 3. Retrofit Geocomposite Fin Drain

c. <u>Edgedrain Location</u>. For the retrofit case, the edgedrain should be located adjacent to the pavement under the shoulder so that water entering the pavement/shoulder joint can drain rapidly. In the retrofit case, the edgedrain should be placed primarily to intercept flow from the slab/base interface. Dense-graded impermeable bases, subbases, and subgrades cannot effectively be drained. With the retrofit case where tied PCC shoulders exist, the edgedrain should generally be located along the outside edge of the shoulder. For the edgedrain location on a new or reconstructed pavement with a permeable base refer to Figure 1.

d. <u>Geotextile Design</u>

(1) As voids develop at the slab/base interface, free water under pressure from moving heavy wheel loads will erode fines in the base material. These fines will migrate to the edgedrain. If the edgedrain is completely wrapped in a geotextile, eroded fines may collect on the surface and blind the geotextile or get trapped within the matrix and clog the geotextile. Once the geotextile has been blinded or clogged, there is no path for the water to escape and the entire pavement section will become saturated. This condition will reduce subgrade strength, accelerating pavement deterioration.

(2) Most geotextiles used for pavement drainage and filtration applications have AOS's in the 40 to 70 range. It is important that the permeability of the geotextile be greater than that of the adjacent base material. This ensures rapid removal of water that migrates to the slab/base interface, and to a much lesser extent, allows water to drain from the base while retaining the base material. The recommended permeability of a geotextile should be within a range of 4 to 10 times the permeability of the adjacent base. Most

of the geotextiles used by SHA's in pavement drainage/filtration applications have a permeability in the range of 100 to 500 feet per day. While these rates are much greater than that of most existing dense-graded base materials, they may be much less than the permeability of most permeable bases.

(3) The greater the percentage of fines in the base material, and the more free water present in the base; the more aggravated the potential clogging problem will be. Regardless of the geotextile placement, fines will be eroded from the base. The geotextile only controls what happens to the fines after erosion (i.e., retain or allow to pass through).

(4) It is recommended that the trench only be partially wrapped with a geotextile as shown in Figure 2. By eliminating the geotextile at the slab/base interface, free water entering at the pavement/shoulder joint and water flowing at the slab/base interface will be drained. This will drastically reduce the time water is available to saturate the base. Partially wrapping the trench creates the best hydraulic conditions for draining the free water present.

(5) The trench for the longitudinal edgedrain collector system for a permeable base is generally lined with a geotextile. However, the top of the trench is left open to the permeable base to allow water a direct path into the collector system. See Figure 1.

e. Collector Pipe. Most SHA's use flexible, corrugated polyethylene (CPE) or smooth rigid polyvinyl chloride (PVC) pipe. Pipe should conform to the appropriate State or AASHTO Specification. For CPE pipe, AASHTO specification M 252 Corrugated Polyethylene Drainage Tubing is suggested, while for PVC pipe, AASHTO Specification M 278, Class PC 50 Polyvinyl Chloride (PVC) Pipe, is recommended. If the pipe will be installed in trenches that are to be backfilled with asphalt stabilized permeable material (ASPM), the pipe must be capable of withstanding the temperature of the ASPM. PVC 90 degree centigrade electric plastic conduct, EPC-40 or EPC-80 conforming to the requirements of National Electrical Manufacturers Association (NEMA) Specification TC-2 is suggested when ASPM is used as a trench backfill.

f. Trench Backfill

(1) The edgedrain trench should be backfilled with a permeable material to rapidly convey water to the drainage pipe. Many SHA's use the AASHTO No. 57 stone for trench backfill. This material can be unstabilized or stabilized. Unless the unstabilized permeable backfill material is properly compacted, settlement over the edgedrain may occur. A

solution to the settlement problem is to use a stabilized permeable backfill material. Gradations similar to stabilized permeable base as discussed in paragraph 6.b. can be used for backfill. If asphalt cement stabilized backfill is used, geotextiles and pipes which will withstand the temperatures of the material must be specified.

(2) For geocomposites, the trench is usually backfilled with the previously excavated material. Care must be taken in the backfilling so that the geocomposite is not damaged. Proper compaction of the backfill is necessary to keep the geocomposite aligned, held tight against the pavement, and to prevent settlement.

g. Trench Cap. The edgedrain trench should be capped with a layer of like shoulder material. The longitudinal pavement/shoulder joint should be sealed to reduce the infiltration of surface water into the pavement structure.

h. Lateral Outlet Pipe. The installation of the outlet pipe is critical to the edgedrain system. It is recommended that a metal or rigid solid-walled pipe be used for the lateral outlet pipe to ensure the proper grade. Also they are less susceptible to crushing by mowing operations or emergency stops by heavy vehicles than flexible pipe. A 3 percent slope to the ditch as shown in Figure 4 is recommended. This will ensure that the pipe will drain if there is a slight variance of the pipe grade. A collector pipe system may have to be installed if ditches or medians are too flat to outlet the pipe. The invert of the outlet pipe should be at least 6 inches above the 10-year design flow in the ditch. Outlet pipes should be connected to existing storm drains or inlets, if possible, to provide better gradient and to reduce outlet maintenance. The trench for the outlet pipe must be backfilled with a material of low permeability, or provided with a cut-off wall or diaphragm, to prevent piping. Also, subsurface drainage design should be coordinated with surface drainage.

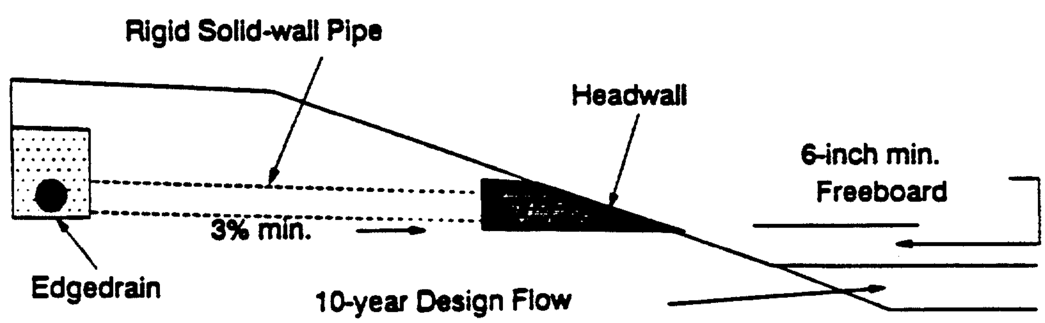


Figure 4. Outlet Pipe Design

i. <u>Outlet Spacing</u>. The purpose of subsurface drainage is to remove water from the pavement structure as quickly as possible; therefore, outlet spacing should be limited to 250 to 300 feet. The edgedrain should be segmented so that each section drains independently.

j. <u>Headwalls</u>. Headwalls are recommended because they provide the following functions: 1) protect outlet pipe from damage, 2) prevent slope erosion, and 3) facilitate the location of outlet pipes. Headwalls should be placed flush with the slope so that mowing operations are not impaired. Positive grades should be provided so that the headwall apron will drain. Both cast-in-place and precast concrete headwalls can be used. The important consideration is maintaining the outlet pipe grade. Some SHA's have used a metal pipe sleeve around plastic outlet pipes that extend 4 to 5 feet into the fill to protect the outlet pipe. A recommended design is shown in Figure 5.

k. <u>Rodent Screens</u>. Rodent screens are recommended as rodents have been reported to damage geocomposite fin drains and build nests in pipe edgedrains. The opening size of the rodent screen should be between 1/4 and 3/8-inch square. Erosion of base fines can build up on rodent screens and restrict the outflow. Rodent screens should be easily removable so that the screens and the outlet pipes can be cleaned (see Figure 5).

l. <u>Reference Markers</u>. Reference markers are recommended because they facilitate locating edgedrain outlets for maintenance or observation. Some SHA's use a simple flexible delineator post to mark the outlet, while others use a painted arrow or other marking on the shoulder.

m. <u>Horizontal Cross Drain</u>. In some cases, a horizontal cross drain may be required as part of a permeable base. A cross drain must be provided at the low-end terminal of permeable base projects (i.e., abutting impermeable base pavement, a bridge approach slab, a sleeper slab, a pavement end anchor or a pressure relief joint). In such cases, a rectangular trench lined with geotextile containing a collector pipe and backfilled with permeable material should be used. The trench should be a minimum of 1-foot deep, 2 feet long, and running the full width of the pavement (see Figure 6). The use of horizontal cross drains on steep grades is generally not necessary. Theoretically, these drains will only collect a small quantity of water. However, in areas such as sag vertical curves or in horizontal curve transition areas horizontal cross drains should be considered. Coordination of the cross drains with the longitudinal structural section drainage systems is important.

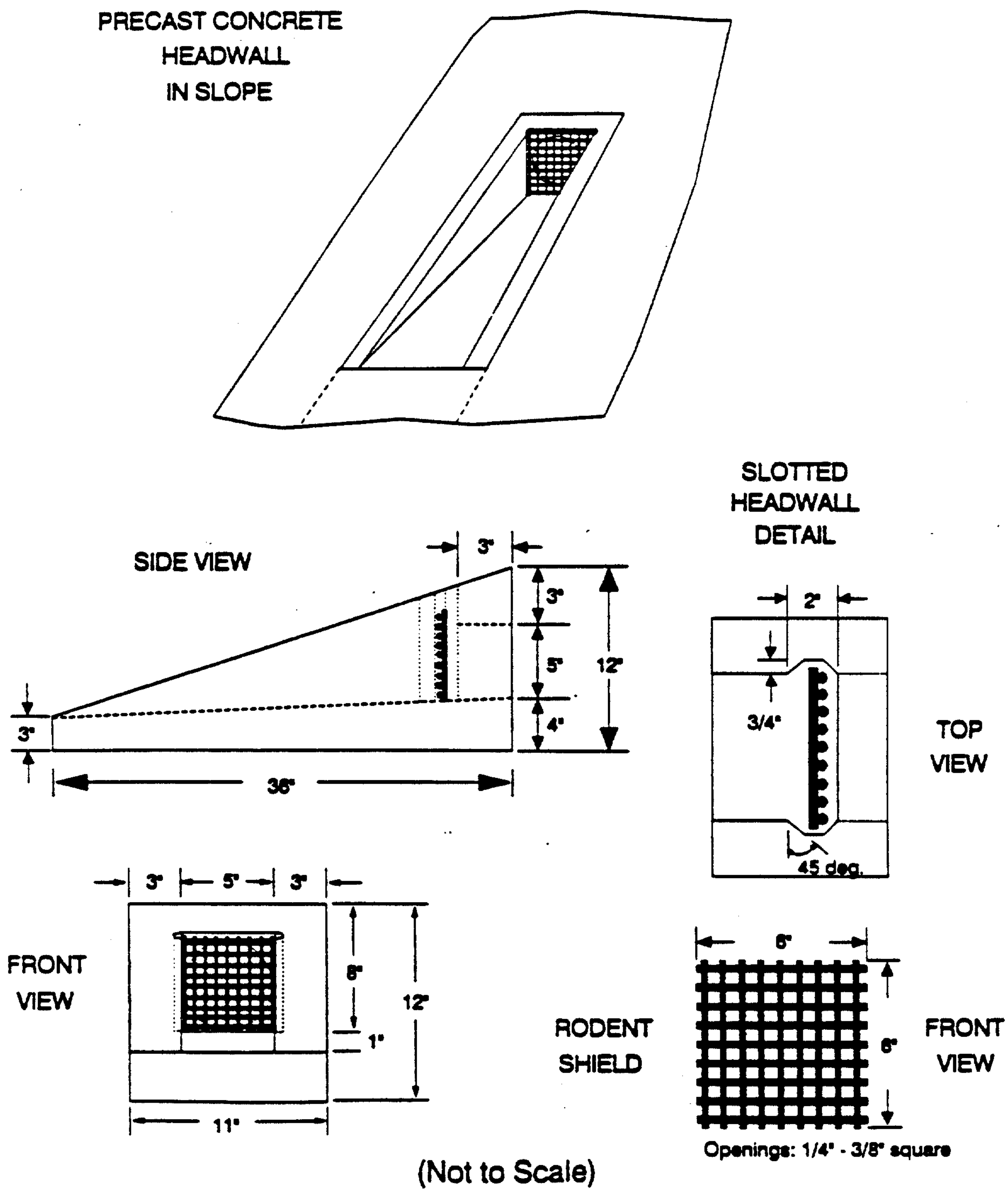


Figure 6. Precast Concrete Headwall with Removable Rodent Screen

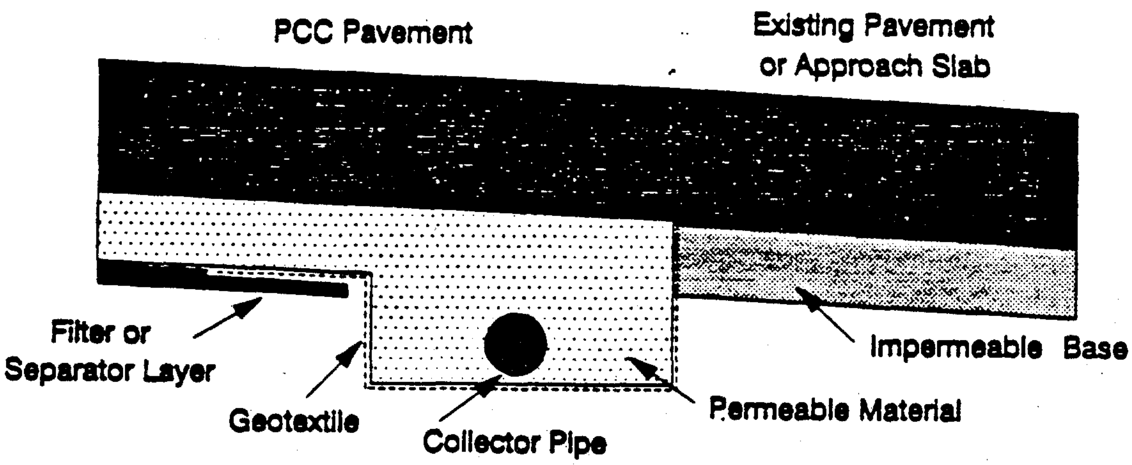


Figure 6. Horizontal Cross Drain

n. Construction Considerations

(1) Attention to details when constructing the longitudinal edgedrain collector system is critical to proper performance of the edgedrain, whether in a retrofit case or as part of a permeable base. As with any other drainage facility, correct line and grade are critical to the hydraulic function of the edgedrains. The placement of the lateral outlet pipe in the trench is very important. High or low spots in the trench must be avoided. The slope of the lateral outlet pipe should be equal to or greater than that of the longitudinal edgedrain.

(2) To prevent water entrapment, it is critical that the exposed end of the pipe is not turned upward or otherwise elevated due to poor construction procedures. There have been some problems noted where the slope of the embankment has prevented a good fit of the lateral pipe into the slope. In a few States, headwall aprons were observed with a reverse grade. Because of improper construction, placement, or settlement, the headwall apron sloped back towards the pipe. Another problem observed was the curling up of the last few feet of flexible outlet pipe resulting in a non-draining outlet. This increases the potential for pavement problems by not allowing accumulated free water adjacent to the pavement structure to drain as rapidly. The pipe curling problem was not observed in those States where rigid lateral outlet pipes were used.

(3) Proper joint seal construction can significantly reduce the amount of moisture entering the pavement.

(4) If undersealing is needed, it should precede the installation of an edgedrain system because of the potential for this operation to contaminate the geotextile and/or aggregate backfill materials.

o. <u>Maintenance</u>

(1) Maintenance is critical to the continued success of any longitudinal edgedrain system. Inadequate maintenance is a universal problem. The combination of vegetative growth, roadside slope debris, and fines discharging from the edgedrains will eventually plug the outlet pipe. Often, outlets can not be found because they are completely covered with vegetative growth and/or roadside slope debris. When outlets that could be found were unplugged, water surged from the pipes.

(2) It is obvious that if maintenance personnel cannot find the outlets no maintenance can be performed. SHA's that used concrete headwalls and/or reference markers had better success at finding outlets. The outlets could be found and maintenance provided.

(3) SHA's should be encouraged to mow around the outlets and clean the outlet pipes a minimum of twice each year.

(4) Periodic flushing or jet rodding of the edgedrain system is important to the continued performance. Therefore, it is important to have the pipe aligned with the proper radii to facilitate this maintenance operation. It is suggested that plan sheets showing alignment of drains and outlets and details on curved connectors.

(5) Maintenance policies should recognize the benefits and necessity of maintaining the joint sealant and thus preventing water from infiltrating into the base layer.

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APPENDIX A

"Filter-Separator Layer Design" Example Problem

A typical subgrade gradation and the unstabilized permeable base gradation from page 10 were selected for this problem. The first step is to plot the gradation of both the permeable base and the subgrade on a gradation chart (shown by the solid lines on Figure A-1).

Then using Figure A-1, determine the D_{85}, D_{50}, and D_{15} particle sizes from the permeable base and subgrade gradation curves:

	Permeable Base (mm)	Subgrade (mm)
D_{85}	17.0	0.65
D_{50}	6.0	0.13
D_{15}	1.85	0.038

where the D_x equals the grain size that "X" percent of the particles, by weight, are smaller.

The next step is to apply the design equations (from page 12) to the filter-separator/subgrade interface and plot the points on a gradation chart (Figure A-1):

EQ. 1 D_{15} (Filter-Separator) $\leq 5\ D_{85}$ (Subgrade)

D_{15} (Filter-Separator) $\leq 5 \times 0.65$

D_{15} (Filter-Separator) ≤ 3.25 mm

EQ. 2 D_{50} (Filter-Separator) $\leq 25 D_{50}$ (Subgrade)

D_{50} (Filter-Separator) $\leq 25 \times 0.13$

D_{50} (Filter-Separator) < 3.25 mm

The equation 1 and 2 criteria are superimposed on the gradation curves as shown by the triangular points on Figure A-1.

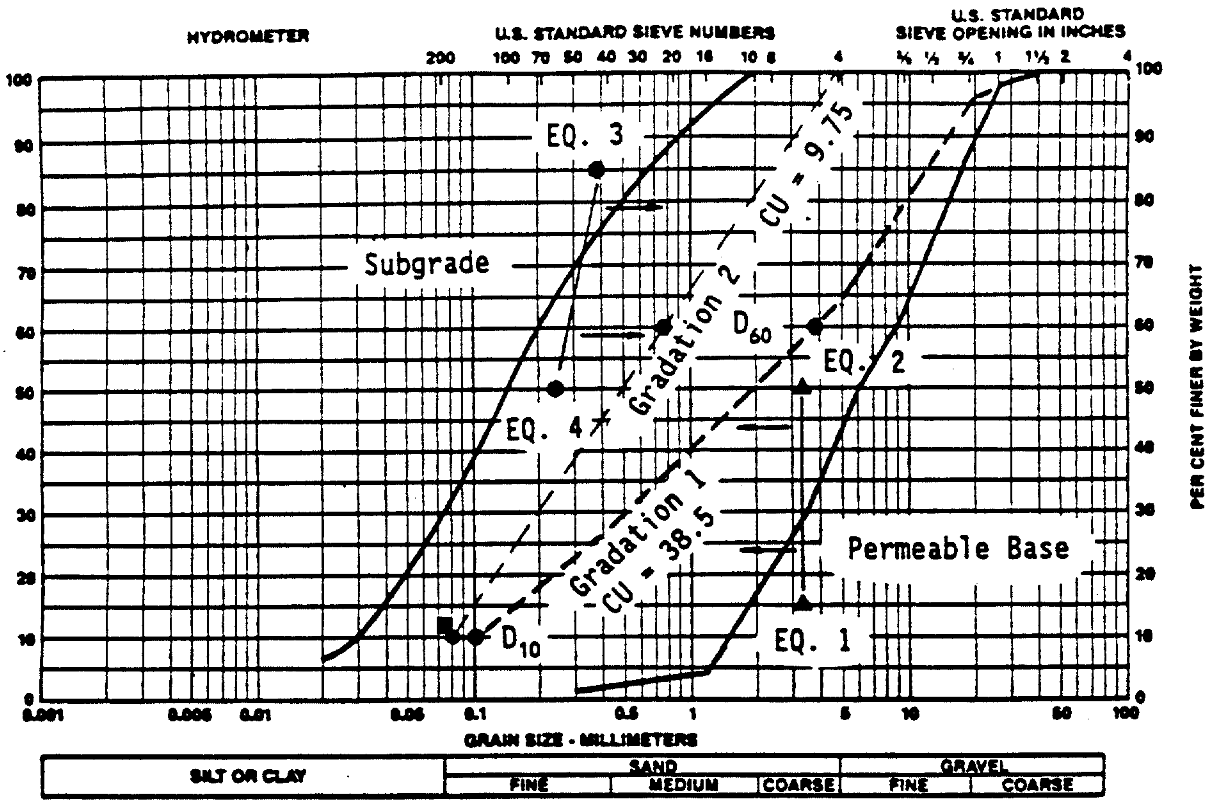

Figure A-2

Once the mid-points of the gradation band are plotted the CU (D_{60}/D_{10}) can be determined. It is recommended that the gradation meet the requirement that the CU be $\geq$ 20 and $\leq$ 40 (requirement from page 12) to ensure that the gradation is well-graded and stable. For example, when plotted on Figure A-2, Gradation No. 1 indicates that the gradation meets the filter-separator criteria and the maximum 12 percent fines criteria.

The final step is to pick out the D_{60} and D_{10} on the dashed line (circular points) and calculate the CU. The CU for this gradation is 38.5 (D_{60}/D_{10} = 3.85 mm/0.1 mm) which falls within the recommended criteria indicating a well-graded and stable gradation.

	Percentage Passing	
Sieve Size	Gradation No. 1	Gradation No. 2
1 1/2 inch	100	-
3/4 inch	85-100	-
No. 4	50-80	100
No. 16	-	60-75
No. 40	20-35	35-50
No. 100	-	15-30
No. 200	5-12	5-12

Gradation No. 2 (on Figure A-2) is a coarse sand gradation which also meets the filter-separator criteria and the maximum 12 percent fines criteria. However, it has a CU of 9.75 (D_{60}/D_{10} = 0.78 mm/0.8 mm) indicating a more uniform, less stable gradation which doesn't meet the recommended criteria.

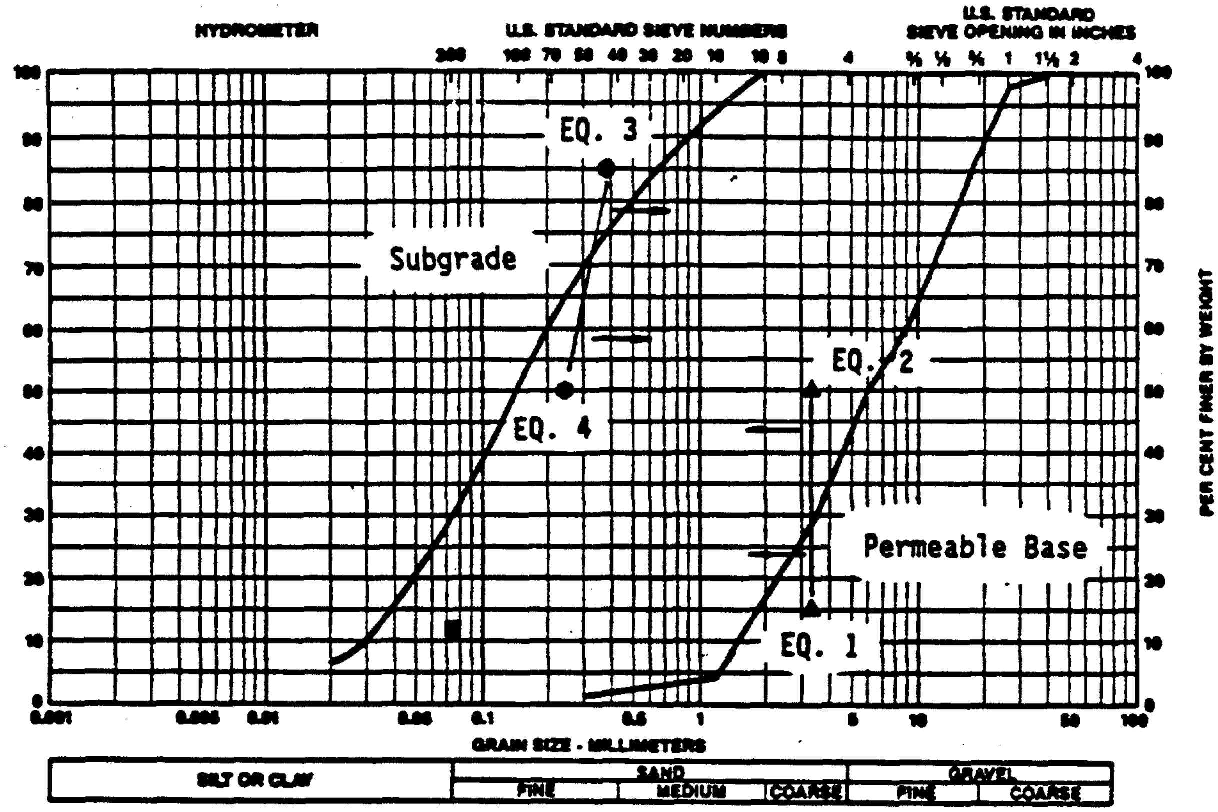

Figure A-1

The next step is to apply the design equations (from page 12) to the permeable base/filter-separator interface and plot the points on the gradation chart:

<u>EQ. 3</u> D_{15} (Base) $\leq$ 5 D_{85} (Filter-Separator)

1.85 $\leq$ 5 D_{85} (Filter-Separator)

D_{85} (Filter-Separator) $\geq$ .37 mm

<u>EQ. 4</u> D_{50} (Base) $\leq$ 25 D_{50} (Filter-Separator)

6.0 $\leq$ 25 D_{50} (Filter-Separator)

D_{50} (Filter-Separator) $\geq$ .24 mm

The equation 3 and 4 criteria are superimposed on the gradation curve as shown by the hexagonal points on Figure A-1.

The mid-point of the filter-separator layer gradation band must fall within the lines joining the two triangular and two hexagonal points determined by the previous equations to meet the criteria. In addition, it is recommended that the gradation have 12 percent or less of the material passing the No. 200 sieve (square point on Figure A-1).

APPENDIX B

TABLE 1
PHYSICAL REQUIREMENTS [1,2]
FOR DRAINAGE GEOTEXTILES

From AASHTO-AGC-ARTBA Task Force 25

Property	Drainage [3] Class A[4]	Drainage [3] Class B[5]	Test Method
Grab Strength (lbs.)	180	80	ASTM D-4632
Elongation (%)	N/A	N/A	ASTM D-4632
Seam Strength[6] (lbs.)	160	70	ASTM D-4632
Puncture Strength (lbs.)	80	25	ASTM D-4833 (Mod.)
Burst Strength (psi)	290	130	ASTM D-3786
Tear Strength (lbs.) (Trapezoidal Tear)	50	25	ASTM D-4533
Apparent Opening Size US Std. Sieve	1. Soil with 50 percent or less particles by weight passing US No. 200 Sieve, AOS less than 0.6 mm (greater than No. 30 US Std. Sieve)		ASTM D-4751
	2. Soil with more than 50 percent particles by weight passing US No. 200 Sieve, AOS less than 0.3 mm (greater than No. 50 US Std. Sieve)		ASTM D-4751
Permeability[7] (cm/sec)	k geotextile > k soil for all classes		ASTM D4491
Ultraviolet Degradation at 150 hours	70 percent Strength retained for all classes		ASTM D4355

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[1] Acceptance of geotextile material shall be based on Task Force 25 acceptance/rejection guidelines.

[2] Contracting agency may require a letter from the supplier certifying that its geotextile meets specification requirements.

[3] Minimum - Use value in weaker principal direction. Numerical values represent minimum average roll value (i.e., [average] test results from any sampled roll in a lot shall meet or exceed the minimum values in the Table). Stated values are for non-critical, non-severe applications. Lots sampled according to ASTM D4354.

[4] Class A Drainage applications for geotextiles are where installation stresses are more severe than Class B applications, i.e., very coarse sharp angular aggregate is used, a heavy degree of compaction (95 percent or greater AASHTO T99) is specified or depth of trench is greater than 10 feet.

[5] Class B Drainage applications are those where geotextile is used with smooth graded surfaces having no sharp angular projections, no sharp angular aggregate is used; no compaction requirements are light, (less than 95 percent AASHTO T99), and trenches are less than 10 feet in width.

[6] Values apply to both field and manufactured seams.

[7] A nominal coefficient of permeability may be determined by multiplying permittivity value by nominal thickness. The k value of the geotextile should be greater then the k value of the soil.

APPENDIX B

TABLE 2
PHYSICAL REQUIREMENTS
FOR SEPARATION APPLICATIONS[1]
(From AASHTO-AGC-ARTBA Task Force 25)

< 50 PERCENT ELONGATION / > 50 PERCENT ELONGATION[2,3]

SURVIVABILITY LEVEL	GRAB STRENGTH ASTM-D 4632 (LBS)	PUNCTURE RESISTANCE ASTM D 4833 (LBS)	TRAPEZOIDAL TEAR STRENGTH ASTM D 4533 (LBS)
HIGH	270/180	100/75	100/75
MEDIUM	180/115	70/40	70/40

ADDITIONAL REQUIREMENTS	TEST METHODS
APPARENT OPENING SIZE (AOS)	ASTM D 4751
1. Less than 50% soil passing a Std. US No. 200 sieve, AOS < 0.6 mm.	
2. More than 50% soil passing a Std. US No. 200 sieve, AOS < 0.3 mm.	
PERMEABILITY	ASTM D 4491
1. k of the geotextile > k of the soil (permittivity times the nominal geotextile thickness).	
ULTRAVIOLET DEGRADATION	ASTM D 4355
1. At 150 hours exposure, 70% strength retained for all cases.	
GEOTEXTILE ACCEPTANCE	ASTM D 4759

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[1] Values shown are minimal roll average values.
Strength values are in the weaker principle direction.

[2] Elongation as determined by ASTM D 4632.

[3] The values of geotextile elongation do not imply the allowable consolidation properties of the subgrade soil. These must be determined by a separate investigation.

U S Department of Transportation
Federal Highway Administration

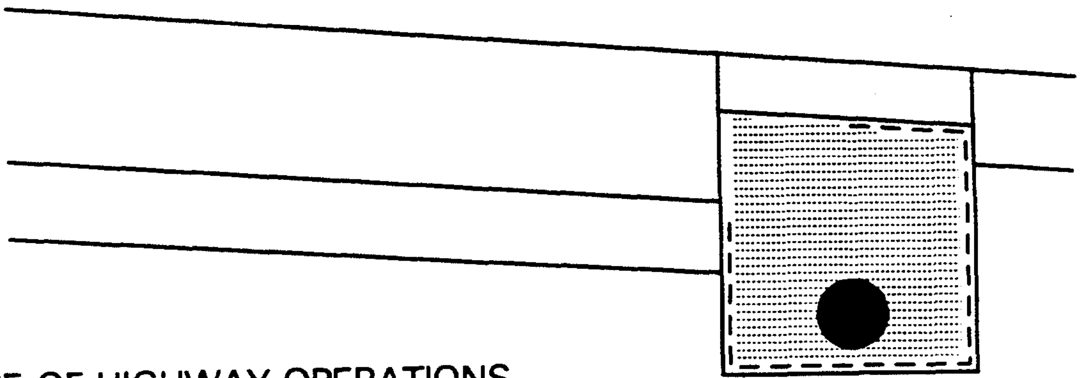
Experimental Projects Program

EXPERIMENTAL PROJECT 12

Concrete Pavement Drainage Rehabilitation

Technology Transfer

ep



OFFICE OF HIGHWAY OPERATIONS
DEMONSTRATION PROJECTS DIVISION
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Experimental Project No. 12

Concrete Pavement Drainage Rehabilitation

State of the Practice Report

By

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U.S. Department of Transportation
Office of Highway Operations
Pavement Division
and
Demonstration Projects Division

April 1989

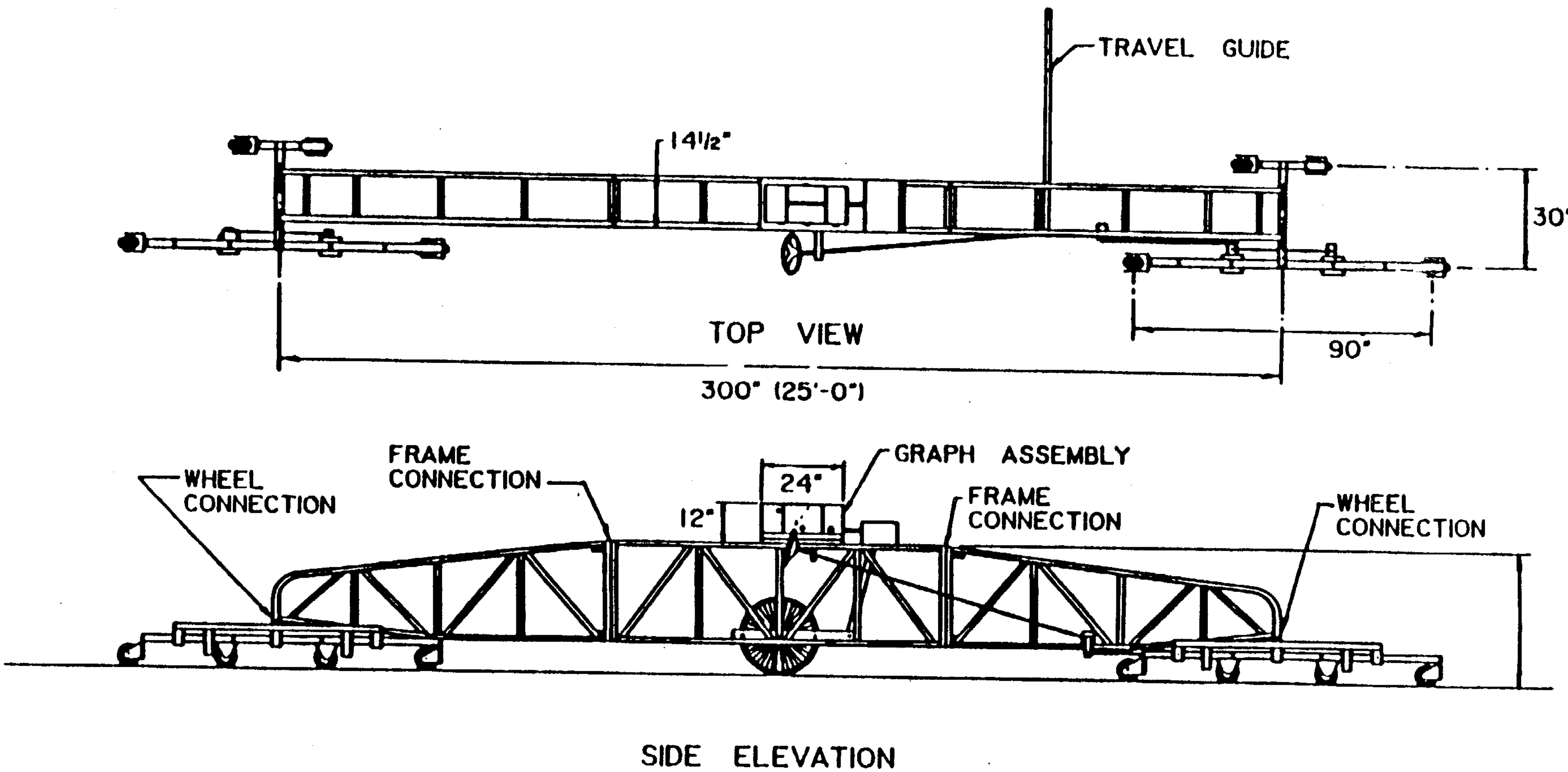

Figure 1. Schematic of California type profilograph.

to develop a state-of-the-practice report on edgedrain design by the States reviewed.

In the third phase, In-depth Analysis, the pavement at the test sites will be instrumented and data will be collected over a 1-year period. Rainfall and edgedrain outlet discharge rates and patterns will be recorded. Soil moisture and pressure transducer gauges will be installed in an attempt to identify moisture conditions under the pavement. Dye will be injected into the pavement structure in an effort to identify subsurface flow patterns.

Nondestructive testing will be accomplished by viewing the edgedrain pipe with a borescope. Faulting and deflection measurements may be taken to determine the condition of pavements.

Test pits will be dug, and the edgedrain trench excavated. Visual observations will be made of the pipe, filter fabric, backfill material, base material and slab/base interface. Permeability tests will be run on the filter fabric and backfill material.

The fourth phase, Analysis and Evaluation, will analyze the data that has been gathered and attempt to evaluate the performance of longitudinal edgedrains. A final report outlining the findings of the study will be prepared.

The Water Resources Division of the U.S. Geological Survey (USGS) has been retained to instrument the pavements, analyze the data, and prepare the final report. Since the USGS has a District office in each State, they will have easy access to the test site. USGS's experience in water data collection and testing should enhance the quality of the project.

Individual SHA's will provide the necessary traffic control, core drilling, saw cuts, and trench excavation.

1.3 Project Selection Criteria and State Selection

It was necessary to develop project selection criteria for selecting the State and projects to be included in the review. The first criteria was that the States selected should have a geographic spread so that the study would represent nationwide conditions.

The most important criteria was that the retrofit longitudinal edgedrains were installed 3 to 10 years prior on PCC pavements showing only a moderate amount of distress. It is believed that this condition will best represent the merits of providing retrofit longitudinal edgedrains. Another criteria that works in concert with this one is the need for a control section. By identifying a similar pavement that was not retrofitted with edgedrains, the rates of deterioration can be compared. If a control section was not available, consideration will be given to plugging of the drain on the selected section to simulate an undrained condition.

A PCC pavement not having received an asphalt concrete (AC) overlay was also a project criteria. The need to have the pavement directly subject to rainfall was recognized. A lesser criteria was that the project be located relatively near the State Capital so that it would receive the necessary attention during the instrumentation phase.

Submissions describing the projects available in the individual States were forwarded to FHWA for review. After an in-depth review the following States were selected; Alabama, Arkansas, California, Illinois, Minnesota, New York, North Carolina, Oregon, West Virginia, and Wyoming.

2.0 SUMMARY OF STATES' PHILOSOPHY ON RETROFIT EDGEDRAIN DESIGN

The basic approach to edgedrain design varied among the States reviewed. Each State believes that its particular design best meets the needs of the State. The following is a discussion of each State's basic approach to edgedrain design.

2.1 Alabama

Rehabilitation of high-type (Interstate) PCC pavements in Alabama includes installation of longitudinal edgedrains (an aggregate trench drain). New PCC pavements also are constructed with the same longitudinal edgedrain design. Since water is being drained the State feels that edgedrains are beneficial. The State is pleased with its edgedrain design and believes edgedrains extend the service life of its PCC pavements. Alabama's standard PCC pavement section is a dowelled jointed plain concrete pavement (JPCP) consisting of 9 inches of PCC over 6 inches of soil subbase which has been stabilized with 7 percent cement. Beneath this is a 6-inch layer of soil subbase on top of 12 inches of improved roadbed. The soil subbase contains up to 40 percent minus No. 200 sieve material.

2.2 Arkansas

Arkansas has been installing longitudinal edgedrains on PCC pavements since 1975/76. Approximately 150-200 lane miles of edgedrains have been installed in that time with the basic edgedrain design remaining the same. Arkansas' edgedrains are designed to rapidly drain the water that migrates to the slab/base interface and to permit the draining of infiltrated moisture trapped in the poor draining base (the majority of PCC pavements in Arkansas were constructed on a crushed stone or gravel base with very low permeability).

Arkansas' PCC pavements are generally 10-inch jointed reinforced concrete pavements (JRCP) with dowelled contraction joints at 45-foot spacing. Warping joints are also constructed at 15-foot intervals in the slab. Rehabilitation of PCC pavements in Arkansas generally consists of installing edgedrains in conjunction with concrete pavement restoration (CPR). It is hoped that rehabilitation will give an additional 10 years of service life to the pavement. Arkansas does not have any quantitative criteria for when to install retrofit longitudinal edgedrains. Installation is based on visual observations of moisture related distress.

2.3 California

Retrofit longitudinal edgedrains were California's first attempt at pavement drainage. They have been installing retrofit longitudinal edgedrains (on PCC pavements only) on a routine basis since 1978. Over 500 lane miles of edgedrains have been installed since then. Most of California's PCC pavements are plain jointed undowelled with short joint spacing (15 feet) constructed over a cement treated base (CTB) or lean concrete base (LCB) placed over a minimum 24 inches of aggregate subbase with an R-value of 50. Their edgedrains are designed to rapidly drain the water that migrates to the slab/base interface. Generally, no other work is performed on the pavement at the time retrofit longitudinal edgedrains are installed. It is hoped that edgedrains will give an additional 10-15 years of service life to the pavement. Edgedrains are installed along the outside lane only, except in superelevated sections where they are installed along the inside lane as well.

California was the only State that evaluated the effect retrofit longitudinal drains have on PCC pavement performance. Based on this evaluation, the following criteria were developed for installing retrofit longitudinal edgedrains on PCC pavement:

PCC pavement:

1) with no more than 10 percent first stage cracking (one crack per panel) and/or 1 percent third stage cracking (fragmentation of the slab as evidenced by three or more interconnecting cracks);
2) that is no more than 10 years old; and
3) with less than 13 million accumulated ESAL's (equivalent single axle loads).

2.4 Illinois

Illinois has been installing longitudinal edgedrains (on PCC pavements primarily) on a routine basis since 1971. From 1976 to 1985 an average of 1.9 million feet of edgedrain was installed. Illinois' edgedrains are designed to rapidly drain the water that migrates to the slab/base interface, to permit the draining of infiltrated moisture trapped in the poor draining base (the majority of PCC pavements in Illinois were constructed on a dense graded aggregate base (DGAB) or bituminous aggregate material (BAM)), and to drain the subgrade. Rehabilitation of PCC pavements (both JRCP and continuously reinforced concrete pavement (CRCP)) in Illinois generally consists of installing edgedrains prior to shoulder reconstruction or overlaying with AC. Approximately one-half of new high-type pavements are constructed of CRCP and one-half are constructed of JRCP.

Illinois believes that any drainage is better than no drainage. There is no expectation of additional service life with retrofit longitudinal edgedrains although it is believed that drainage does increase the life of the pavement.

The cost of edgedrains has remained in the $2-$3 per linear foot range since 1977. Edgedrains are installed along the outside lane and where feasible, are installed along the inside lane (in the median) as well.

2.5 Minnesota

Minnesota has been installing longitudinal edgedrains (on PCC pavements only) on a routine basis since 1979/80. Over 1100 lane miles of edgedrains have been installed since then. Minnesota's edgedrains are designed to rapidly drain the water that migrates to the slab/base interface, to permit the draining of infiltrated moisture trapped in the poor draining base (the majority of PCC pavement in Minnesota were constructed on a DGAB), and to prevent the stripping in the AC overlay, when used. Rehabilitation of PCC pavement in Minnesota generally consists of installing edgedrains prior to overlaying with AC. It is hoped that rehabilitation will give an additional 10 years of service life to the pavement.

Minnesota has not been able to conclusively prove edgedrains are cost-effective. The State feels that the drains are so inexpensive ($1.00 $1.25 per linear foot) that they can't afford not to put them in. Retrofit longitudinal edgedrains are looked upon as cheap insurance. The State feels that if retrofit longitudinal edgedrains give only an additional 2-3 years of service life to the pavement the edgedrains will have paid for themselves. Edgedrains are installed along the outside lane and where feasible, are installed along the inside lane (in the median) as well.

2.6 New York

New York has been installing longitudinal edgedrains on PCC and AC pavements since 1977. Approximately 600 miles of new and retrofit edgedrains have been installed since then. New York's edgedrains on PCC pavements are designed primarily to rapidly drain infiltrated water that migrates to the slab/base interface and secondarily to permit the draining of infiltrated moisture trapped in the poor draining base (the majority of PCC pavements in New York were constructed on a granular base daylighted to the ditch). Rehabilitation of PCC pavements in New York generally consists of installing edgedrains prior to overlaying with AC.

Installation of longitudinal edgedrains varies from state region to state region. Edgedrain installation is based on field inspection of perceived need. Edgedrains are relatively expensive to install in New York; therefore, good engineering requires discriminate application. Cost is estimated from $10-$12 per linear foot. Cost of edgedrains in New York is believe to be much higher because of increased labor costs. Edgedrains are installed along the outside lane and where feasible, along the inside lane (in the median) as well.

2.7 North Carolina

North Carolina has been installing longitudinal edgedrains on PCC pavements on a routine basis since 1979/80. North Carolina's edgedrains are designed to rapidly drain the water that migrates to the slab/base interface and to permit the draining of infiltrated moisture trapped in the poor draining base (the majority of PCC pavements in North Carolina were constructed on a DGAB). Rehabilitation of PCC pavements in North Carolina generally consists of installing edgedrains as part of CPR. It is hoped that rehabilitation will give an additional 10 years of service life to the pavement. All new construction receive edgedrains on the low side of the pavement. They are looked upon as cheap insurance.

2.8 Oregon

Oregon has been installing longitudinal edgedrains on PCC pavements on a routine basis since 1978/79. Oregon's edgedrains are designed to rapidly drain the water that migrates to the slab/base interface and to permit the draining of infiltrated moisture trapped in the poor draining base (the majority of PCC pavements in Oregon were constructed on a DGAB). Edgedrains are also used to control groundwater. New or reconstructed PCC pavements are generally continuously reinforced placed over LCB. Edgedrains are installed on new and reconstructed PCC pavements if moisture related distress is anticipated or has been a problem in the past. Rehabilitation of PCC pavements in Oregon generally consists of installing edgedrains prior to overlaying with AC. It is hoped that rehabilitation will give an additional 10 years of service life to the pavement. There are no quantitative criteria for the installation of retrofit longitudinal edgedrains. Installation is based on perceived need (i.e., pumping or some other moisture related distress). Edgedrains are considered on all Interstate rehabilitation projects on a case by case basis. Most of the edgedrain projects have been in the I-5 corridor because of the higher precipitation experienced on the western side of the Cascade Mountain Range. Edgedrains have not been used extensively on other road systems.

Oregon has not developed data proving edgedrains are cost-effective. However, they are inexpensive (approximately $2.50 per linear foot). Edgedrains are installed along the outside lane primarily and along the inside lane (in the median) on superelevated sections.

2.9 West Virginia

West Virginia has been installing longitudinal edgedrains on cracked and seated (C&S) PCC pavements only since 1981/82. West Virginia's edgedrains are designed to drain surface water that infiltrates through the pavement and water that migrates up through the underlying layers. The edgedrain also drains water that is trapped in the poor draining base (the majority of PCC pavements in West Virginia are constructed on 6 inches of DGAB). Most PCC pavements are 9-inch jointed reinforced and dowelled with 61.5-foot joint spacing. Rehabilitation of PCC pavements in West Virginia generally consists of installing edgedrains prior to cracking the PCC into 12- to 18-inch pieces and overlaying with 3 to 4 inches of AC. The age of the PCC pavements at rehabilitation is generally 18 years. It is hoped that this rehabilitation will give an additional 10-15 years of service life to the pavement. At present, the State does not install edgedrains on rehabilitated AC pavements. The State does not have any quantitative criteria for installing retrofit longitudinal edgedrains. Evidence of pumping and/or other moisture related distress is the determining factor on whether edgedrains are to be installed. Edgedrains are installed along the outside lane primarily and along the inside lane (in the median) on superelevated sections.

2.10 Wyoming

Late in 1987, Wyoming began installing longitudinal edgedrains on PCC pavements only. Wyoming's edgedrains are designed to rapidly drain water that migrates to the slab/base interface and to permit the draining of infiltrated moisture trapped in the poor draining base (the majority of PCC pavements in Wyoming were constructed on a 6-inch DGAB). Most PCC pavements constructed in Wyoming are 8-inch JPCP with skewed joints (2 feet in 12 feet) spaced at 18, 19, 13, and 12 feet. Rehabilitation of PCC pavements in Wyoming is also just beginning and, to date, consists of some CPR techniques (i.e., patching and slab replacement) and the installation of retrofit longitudinal edgedrains. The State hopes that edgedrains will help reduce the faulting (1/4- to 1/2-inch) that is occurring on their JPCP's. There is not much evidence of pumping on their pavements. Wyoming's edgedrains are installed along the outside lane primarily and along the inside lane in superelevated sections. It is hoped that rehabilitation will give an additional 10 years of service life to the pavement.

A comparison of pavement types and criteria for edgedrain installation is provided in Table 1.

Table 1. Listing of Pavement Types and Installation Criteria

	Pavement Type	Subbase	Edgedrain Installation Criteria
Alabama	JPCP	CTS	Observed Need
Arkansas	JRCP	DGAB	Observed Need
California	JPCP	CTB/LCB	All PCC Meeting State Criteria (1)
Illinois	CRCP	DGAB	All PCC Rehabilitation
Minnesota	JRCP	DGAB	All PCC Rehabilitation
New York	JRCP	DGAB	Observed Need
North Carolina	JPCP	DGAB	All PCC Rehabilitation
Oregon	JPCP	DGAB	Observed Need
West Virginia	JRCP	DGAB	Observed Need
Wyoming	JPCP	DGAB	All Current PCC Rehabilitation

(1) Project must meet State criteria as discussed in Section 2.3.

3.0 REVIEW OF CURRENT EDGEDRAIN PRACTICE

One of the objectives of the Field Review Phase was to determine the current edgedrain practices in the States selected for study. Drainage elements such as edgedrain backfill material, edgedrain location, filter fabric, headwalls, etc., were investigated. This information is presented in the following discussions of each design element.

3.1 Type of Edgedrain

Alabama, North Carolina, and West Virginia are the only States that use a stone filled trench without a continuous drain pipe. The trench is wrapped with a filter fabric and backfilled with a open-graded aggregate. A drainage pipe is installed in the last 200 feet of trench in North Carolina and the last 10 feet of trench in the other States before being outletted. This design is a "french drain" approach. Figure 1 shows the aggregate trench type of edgedrain used by these States.

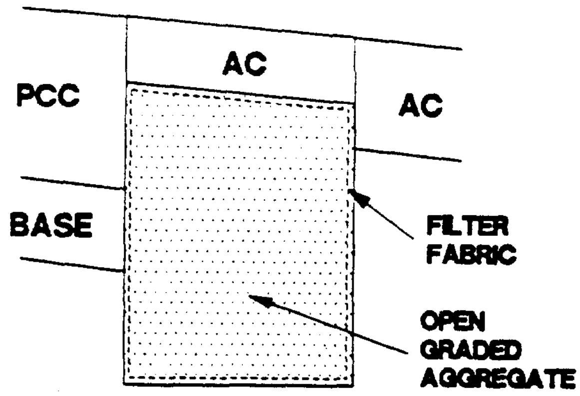

Figure 1. AGGREGATE TRENCH EDGEDRAIN

Illinois, Minnesota, and New York do not wrap the trench with filter fabric but rather use a filter aggregate or coarse sand backfill around a perforated pipe. In this design approach, although generally much slower draining, the drainage aggregate is believed to act as the filtering media to prevent eroded fines from plugging the edgedrain system. Illinois and Minnesota wrap the pipe with filter fabric to prevent the backfill material from entering the pipe. This approach is shown in Figure 2.

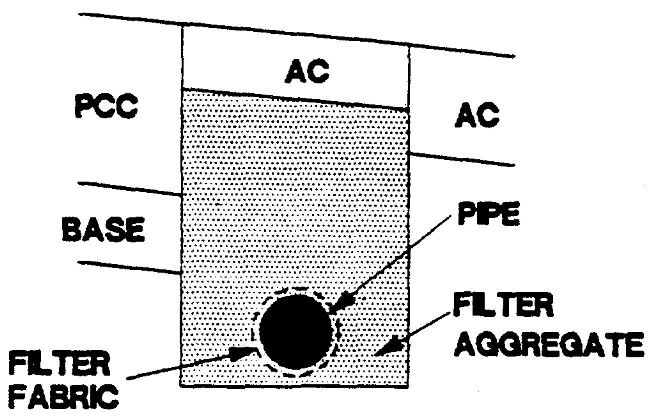

Figure 2. FILTER AGGREGATE EDGEDRAIN

California's edgedrain design consists of a 3-inch slotted rigid PVC pipe which is placed at the bottom of a relatively shallow, partially filter fabric lined trench (12 inches wide and 10 inches deep) excavated slightly into the cement treated base and backfilled with a treated permeable material (TPM) (either asphalt treated at approximately 2 1/2 percent or cement treated at 2 to 4 bags per cubic yard). The purpose of the filter fabric is to prevent aggregate base and subgrade fines from contaminating the edgedrain system. The filter fabric is omitted in the slab/base interface to allow infiltrated water and eroded fines that have migrated to the interface to jet directly into the drain. California's design is unique in that the trench is very shallow. The purpose of the edgedrain system is to drain water which collects at the slab/subbase interface and water entering the pavement/shoulder joint. The invert of the drainage pipe is approximately 1-inch below the slab/subbase interface. California's design approach is shown in Figure 3.

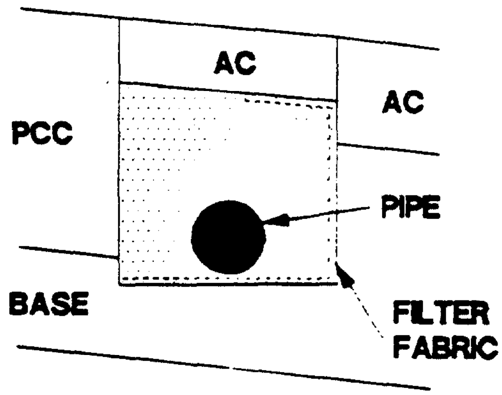

Figure 3. CALIFORNIA EDGEDRAIN DESIGN

All of the remaining States (Arkansas, Oregon and Wyoming) use basically the same design; that is, the trench is completely wrapped with a filter fabric. A 3-4 inch drainage pipe is placed in the bottom of the trench. The trench is then backfilled with an open graded aggregate. A conventional perforated pipe edgedrain is shown in Figure 4.

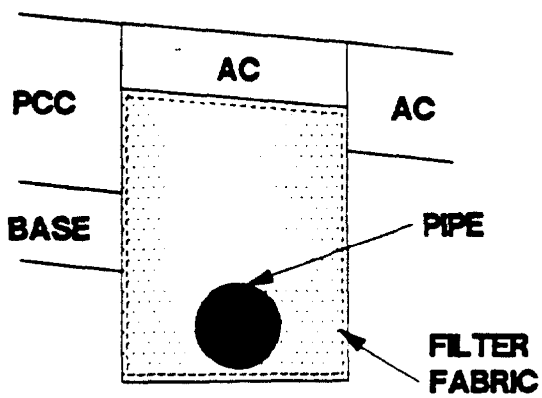

Figure 4. PERFORATED PIPE EDGEDRAIN

Geocomposite fin drains have been used by many States reviewed (Alabama, Arkansas, Illinois, Minnesota, North Carolina, West Virginia, and Wyoming) primarily on an experimental basis. Figure 5 shows a typical geocomposite fin drain. Table 2 provides a comparison of edgedrain types used.

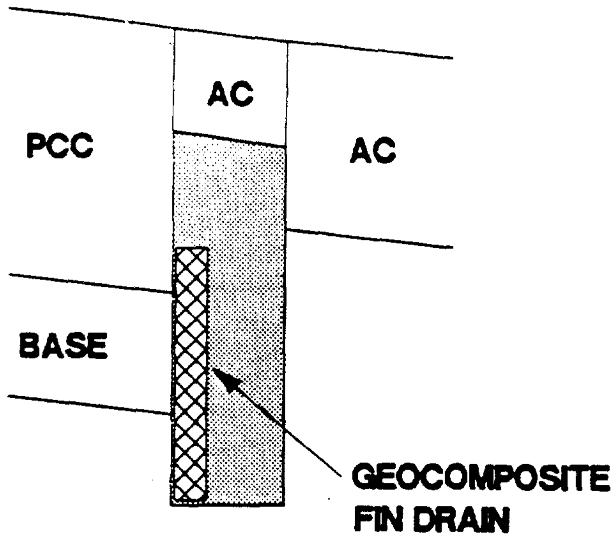

Figure 5. GEOCOMPOSITE EDGEDRAIN

Table 2. Comparison of Edgedrain Types.

	Type of Edgedrain	Geocomposite Fin Drain
Alabama	Aggregate Trench	Experimental
Arkansas	Conventional Pipe	Experimental
California	Shallow Trench	Not Allowed
Illinois	Sand Backfill	Allowed as Alternate
Minnesota	Sand Backfill	Experimental
New York	Filter Aggregate	Experimental
North Carolina	Aggregate Trench	Experimental
Oregon	Conventional Pipe	Allowed as Alternate
West Virginia	Aggregate Trench	Experimental
Wyoming	Conventional Pipe	Experimental

3.2 Edgedrain Location

All of the States reviewed place the edgedrain under the shoulder immediately adjacent to the pavement/shoulder joint.

3.3 Trench Backfill

Both Illinois and Minnesota use a coarse sand backfill. The aggregate gradations are given in Tables 3 and 4, respectively. Illinois and Minnesota anticipate coefficient of permeabilities of 50 to 100 feet per day with these backfill materials. Use of the Illinois gradations is based on local availability of materials.

Table 3. Illinois' Sand Gradations.

	Percent Passing	
Sieve Size	FA 1	FA 2
3/8-inch	100	100
No. 4	94-100	94-100
No. 16	45-85	45-85
No. 50	3-29	10-30
No. 100	0-10	0-10
No. 200	0-3	0-3

Table 4. Minnesota's Sand Gradation.

Sieve Size	Percent Passing
3/8-inch	100
No. 4	90-100
No. 10	45-90
No. 40	15-45
No. 200	0-3

California uses either asphalt treated permeable material (ATPM) at approximately 2 1/2 percent or cement treated permeable material (CTPM) at 2 to 4 bags per cubic yard. Coefficient of permeabilities are approximately 4,000 feet per day for the CTPM and 15,000 feet per day for the ATPM. The gradations for ATPM and CTPM are given in Tables 5 and 6, respectively.

Table 5. California's ATPM Aggregate Gradation.

Sieve Size	Percent Passing
1-inch	100
3/4-inch	90-100
1/2-inch	35-65
3/8-inch	20-45
No. 4	0-10
No. 8	0-5
No. 200	0-2

Table 6. California's CTPM Aggregate Gradation.

Sieve Size	Percent Passing
1 1/2-inch	100
1-inch	86-100
3/4-inch	X ± 22
3/8-inch	X ± 22
No. 4	0-18
No. 8	0-7

Where "X" is the gradation which the contractor proposes to furnish for the specific sieve size.

New York uses a filter aggregate consisting of a crushed stone, sand gravel, or screened gravel with varying degrees of permeability. The filter material gradations are given in Table 7. Gradation type is selected by the State regional soils engineer based on the amount of fines in the native soil. The Type I gradation is used approximately 75 percent of the time. Type III is used if silt is encountered.

Table 7. New York's Aggregate Gradations.

	Percent Passing		
Sieve Size	Type I	Type II	Type III
1-inch	100	-	-
1/2-inch	30-100	100	-
3/8-inch	-	-	100
1/4-inch	0-30	20-100	90-100
No. 8	-	-	75-100
No. 10	0-10	0-15	-
No. 16	-	-	50-85
No. 20	0-5	0-5	-
No. 30	-	-	25-60
No. 50	-	-	10-30
No. 100	-	-	1-10
No. 200 (wet)	-	-	0-3

Oregon uses a gap graded (permeable) aggregate with coefficients of permeability greater than 3000 feet per day. The three gradations used by Oregon are given in Table 8. The type of gradation used is determined by the engineer.

Table 8. Oregon's Aggregate Gradations.

	Percent Passing		
Sieve Size	1 1/2-3/4" size	1 1/4-3/4" size	3/4-1/2" size
2-inch	100	-	-
1 1/2-inch	95-100	100	-
1 1/4-inch	-	90-100	-
1-inch	-	-	100
3/4-inch	0-15	0-15	90-100
1/2-inch	0-2	0-2	0-15
1/4-inch	-	-	0-3

Alabama and North Carolina both use the AASHTO No. 57 gradation as backfill material while West Virginia allows any AASHTO gradation between the No. 2 and No. 57 to be used. Wyoming's gradation is the same as the gradations used by California. Table 9 provides a comparisons of the backfill material used by the States that were reviewed.

Table 9. Comparison of Backfill Material

	Backfill Material	Gradation	Estimated Coefficient of Permeability
Alabama	Aggregate	AASHTO No. 57	3,000 +
Arkansas	Pea Gravel	3/8-inch	200
California	ATPM/CTPM	California Standard	4,000 CTPM 15,000 ATPM
Illinois	Coarse Sand	Illinois Standard	50
Minnesota	Coarse Sand	Minnesota Standard	50-100
New York	Filter Aggregate	New York Standard	1,000 Type I 100 Type III
North Carolina	Aggregate	AASHTO No. 57	3,000 +
Oregon	Pea Gravel	Oregon Standard	3,000 +
West Virginia	Aggregate	AASHTO No. 2 to No. 57	3,000 +
Wyoming	ATPM/CTPM	California Standard	4,000 CTPM 15,000 ATPM

3.4 Pipe Material and Size

Seven of the 10 States reviewed (Arkansas, California, Illinois, Minnesota, New York, Oregon, and Wyoming) used perforated or slotted drainage pipe in the entire length of edgedrain trench to convey the accumulated water from the pavement structure. Two of the States (California and Wyoming) used smooth, rigid polyvinyl chloride (PVC) pipe. The other five States (Arkansas, Illinois, Minnesota, New York, and Oregon) specified corrugated polyethylene (CPE) pipe. The three remaining States (Alabama, North Carolina, and West Virginia) did not use pipe in the entire length of edgedrain trench. Pipe sizes were 3 or 4 inches as shown in Table 10.

Table 10. Pipe Material and Size.

	Pipe Material	Pipe Size (inches)
Arkansas	CPE	4
California	PVC	3
Illinois	CPE	4
Minnesota	CPE	3
New York	CPE	4
Oregon	CPE	4
Wyoming	PVC	3-4

In California and Wyoming, if the pipe is to be installed in trenches that are to be backfilled with asphalt treated permeable material, the pipe shall be PVC 90 degrees C electric plastic conduit, EPC-40 or EPC-80 conforming to the requirements of NEMA Specification TC-2.

3.5 Trench Widths and Depths

Table 11 provides a tabulation of the trench widths and depths encountered in the field reviews.

Table 11. Trench Widths and Depths.

	Trench Width (inches)	Trench Depth(1) (inches)
Alabama	12	27
Arkansas	12	18 (2)
California	8 Min	10 (3)
Illinois	10	30
Minnesota	6-10	15 Min (4)
New York	12	30
North Carolina	12	18 Min (5)
Oregon	8	18 Min
West Virginia	6-10	12 Min (6)
Wyoming	6 Min	18

(1) Measured from the pavement surface.
(2) Invert of pipe is located 12 inches below slab\subbase interface.
(3) Invert of pipe is just below slab/subbase interface.
(4) Invert of pipe is located 3 inches below lowest layer to be drained.
(5) Bottom of edgedrain trench is located 4 inches below the subbase/subgrade interface.
(6) Top of edgedrain is located 3 inches below the top of the slab.

3.6 Filter Fabric Placement

Filter fabric placement is perhaps the most difficult and controversial item in edgedrain design. There are three distinct design approaches to filter fabric placement.

In the first approach, the trench is wrapped in filter fabric to prevent fines from entering the trench backfill as shown in the top sketch of Figure 6. Fines that are eroded from the base course may migrate to and clog the filter fabric.

The second approach leaves the slab/base interface open so that any eroded fines are not retained. Therefore, they will not clog the filter fabric. This approach would have the shortest time to drain and thus less time of saturation. This design is shown in the middle drawing of Figure 6.

The third approach is a compromise in which the pipe is wrapped in a filter fabric and the trench is backfilled with a filter aggregate or coarse sand as shown in the bottom sketch of Figure 6. In this approach, the aggregate acts as a filter keeping the fines from clogging the filter fabric. The coefficient of permeability of the filter aggregate material varies, but it is generally much lower than an open-graded aggregate backfill.

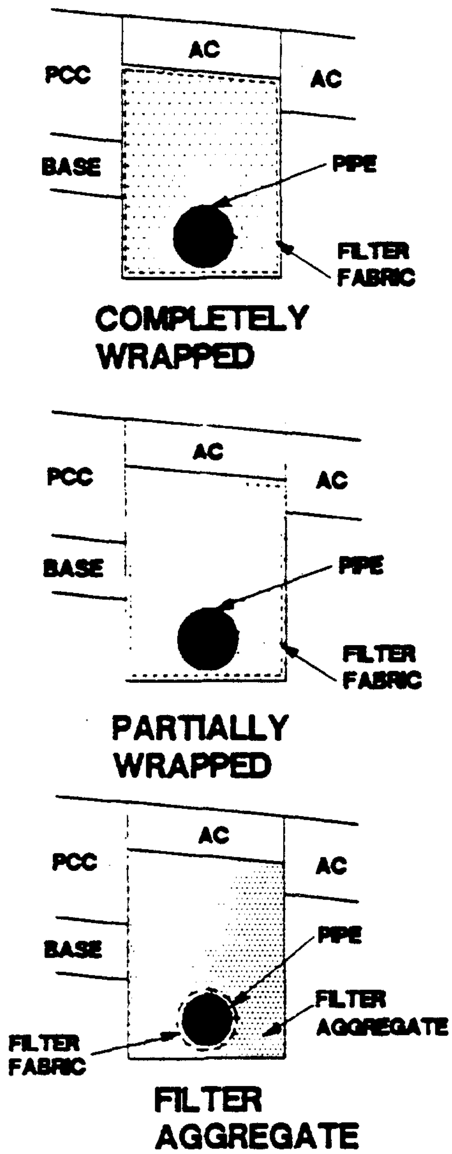

Figure 6. COMPARISON OF EDGEDRAIN DESIGN

It is pointed out that in all of the approaches any erodible fines in the base course will be washed out. The difference in the approaches is the manner in which the fines are handled.

It should be noted that there is no way to prevent a filter adjacent to a material with a high percentage of fines from eventually clogging. If there are no voids or if the voids are small, the filter won't clog up as rapidly, and the filter will function for a longer period of time. If, however, voids are present between the material to be drained and the filter, soil particles are provided an opportunity to go into suspension and will eventually clog the filter. Likewise, filter fabrics need intimate contact with the material to be drained. A filter placed along a pavement with voids between the slab and base would be comparable to the above noted situation.

A study of the three approaches reveals that each approach has advantages and disadvantages. This study indicates that fabric placement is one of the most important elements of edgedrain design, and perhaps, the most unresolved. Each State must be careful to wrap the trench in a fashion that best meets the pavement conditions encountered.

Illinois and Minnesota wrap the drainage pipe with filter fabric using the trench backfill to help filter out fines; however, New York does not use any filter fabric.

California partially wraps the trench leaving the interface with the base course open to prevent the fabric from clogging. A TPM (either ATPM or CTPM) is used as the trench backfill.

All of the remaining States (Alabama, Arkansas, North Carolina, Oregon, West Virginia, and Wyoming) completely wrap the trench.

Table 12 provides a comparison of filter fabric placement.

Table 12. Filter Fabric Placement

	Filter Fabric Placement
Alabama	Wrapped Trench
Arkansas	Wrapped Trench
California	Partially Wrapped Trench
Illinois	Wrapped Pipe
Minnesota	Wrapped Pipe
New York	None
North Carolina	Wrapped Trench
Oregon	Wrapped Trench
West Virginia	Wrapped Trench
Wyoming	Wrapped Trench

3.7 Outlet Spacing

Outlet spacing varied considerably among the States reviewed. Table 13 lists the outlet spacing. Since the purpose of the edgedrain is to remove water from the pavement structure, outlet spacing should not be excessive.

Table 13. Outlet Spacing.

	Outlet Spacing (feet)
Alabama	200-1000
Arkansas	300
California	200-300
Illinois	500
Minnesota	500
New York	250
North Carolina	500
Oregon	400
West Virginia	500
Wyoming	300

3.8 Headwalls

Headwalls are used to protect the outlet pipe from damage, to prevent slope erosion, and to ease the locating of the outlet pipes. Table 14 provides a tabulation of the headwall types encountered in the field reviews.

There was a large variety in the types of headwall used. Alabama provides a large cast-in-place concrete headwall that is flush with the slope so that there is no damage from mowing operations. California's design is a simple precast concrete splash pad that allows the discharge to spread out thus preventing slope erosion. Minnesota and Illinois use a flush, precast concrete headwall with a removable rodent screen. Minnesota's precast concrete headwall design is shown in Figure 7.

3.9 Rodent Screens

Many States believe that rodent screens are necessary to protect the edgedrain system. Table 14 lists the States that used rodent screens in the review.

Some States not included in the review have experienced considerable damage to geocomposite fin drains from field mice.

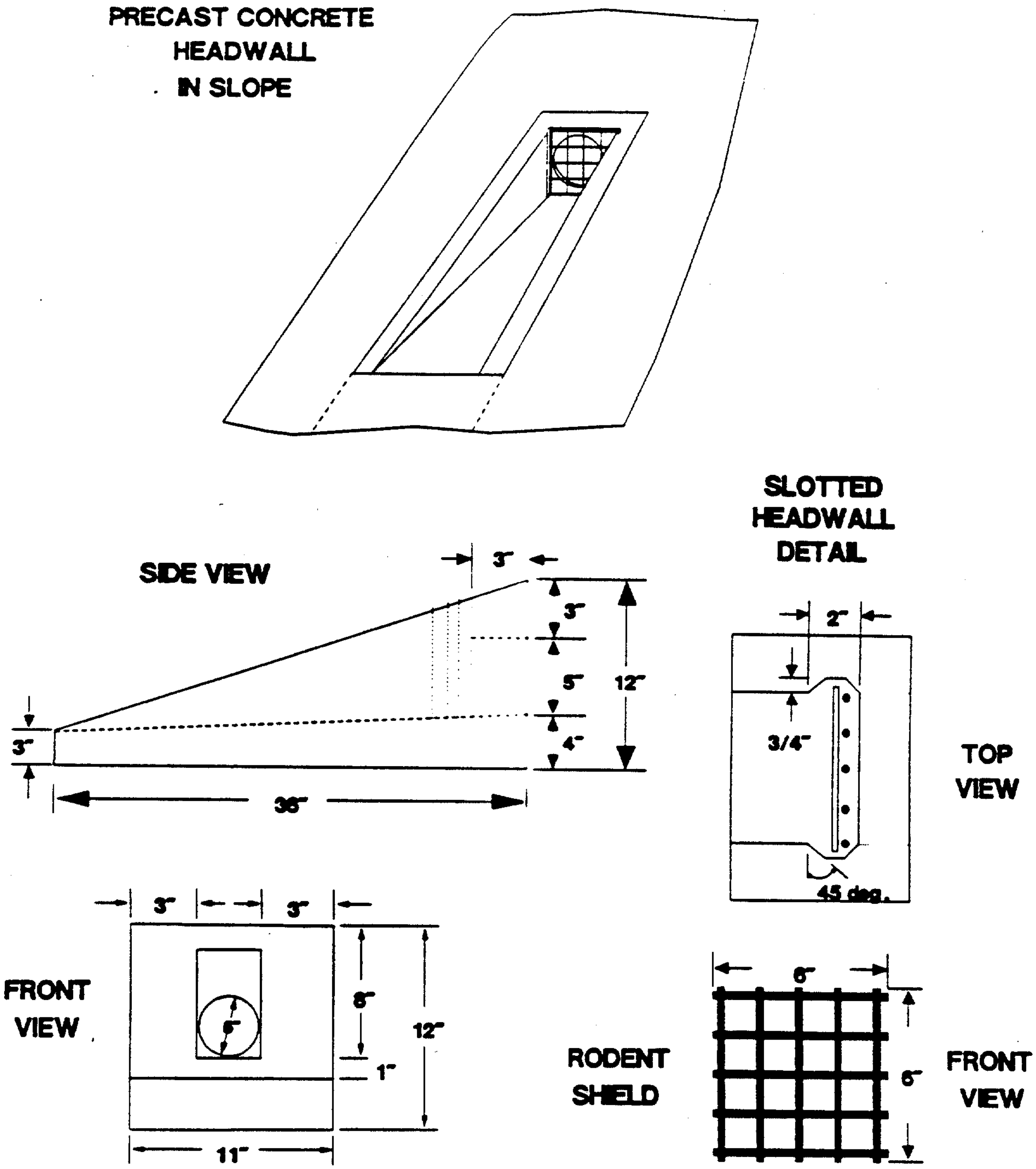

Figure 7. Minnesota's Precast Concrete Headwall

3.10 Reference Markers

Reference markers are used to locate the outlets for maintenance or observation purposes. Reference markers are extremely important in directing maintenance personnel to the pipe outlet. Table 14 indicates which States use reference markers.

California places a raised ceramic pavement marker on the shoulder edge adjacent to the outlet pipe while Minnesota paints a small arrow or stripe on the edge of the shoulder adjacent to the outlet pipe. California and Oregon use a small sign on a metal post to mark the pipe outlet.

Table 14. Headwalls, Rodent Screens, and Reference Markers.

	Headwall	Rodent Screen	Reference Marker
Alabama	Concrete	No	No
Arkansas	Concrete	Yes	No
California	Splash Pad	Yes	Yes
Illinois	Concrete	Yes	No
Minnesota	Precast Concrete	Yes	Yes
New York	None (1)	No	No
North Carolina	Concrete	No	No
Oregon	Concrete	Yes	Yes
West Virginia	Concrete	Yes	No
Wyoming	Splash Pad	Yes	No

(1) A 6-foot section of 6-inch corrugated metal pipe is used to protect the plastic outlet pipe.

3.11 Maintenance

Maintenance is critical to the continued success of any longitudinal edgedrain system. Inadequate maintenance was an universal problem in the States reviewed. The combination of vegetative growth, roadside slope debris, and fines discharging from the edgedrain plugged a number of outlet pipes. At one outlet, a 3-foot long mass of bermuda grass runners and eroded fines was pulled from the outlet pipe. It was impossible for the edgedrain system to discharge any water from this outlet until the mass of material was removed.

On one project, where pumping stains were noted on the right shoulder, it was found that this pumping was occurring on pavement sections where the outlets were plugged. On adjacent sections, where edgedrains outlets were open, there were no signs of pumping. At another outlet, the pipe was completely covered and plugged with vegetative growth. When the pipe was unplugged water drained from the pipe. Many outlets could not be found because of dense vegetative growth. It is obvious that if maintenance crews cannot find the outlet, no maintenance of the edgedrain system can occur.

Based on the observations made during this review , increased emphasis should be placed on maintenance of longitudinal edgedrains, especially the outlets.

3.12 Construction Related Problems Observed

Since all of the edgedrain projects reviewed were previously constructed, it was not possible to identify any construction problems of the longitudinal edgedrain collector system. However, some problems were observed with the lateral outlets. In a few States, headwalls were observed with a reverse grade. Because of improper construction, placement, or settlement, the headwall apron sloped back towards the pipe. Although the outlet would drain when sufficient water had accumulated, sedimentation at the outlet will occur restricting the flow and eventually plugging the pipe. Another problem observed in several States was the curling up of the last few feet of flexible outlet pipe resulting in a nondraining outlet. This may not be a big concern where the edgedrain trench was continuous and where subsequent outlets down grade would allow the water to drain. However, restricted flow from the edgedrain system would increase the time the pavement structure is subject to moisture. This has the potential for increased pavement problems by not allowing accumulated free water adjacent to the pavement structure to drain as rapidly. The pipe curling problem was not observed in those States that used a rigid lateral outlet pipe.

Based on the observations made during this review, increased emphasis should be placed on construction inspection of longitudinal edgedrain systems, especially the outlets. It was apparent that more attention needs to be focused on maintaining the grade of the outlet trench, ensuring the proper placement of the pipe in the trench, and the construction or placement of the outlet headwall. Proper construction is essential for the edgedrain system to perform as intended.

4.0 SUMMARY AND CONCLUSIONS

4.1 Design Philosophy

Retrofit longitudinal edgedrains are an important technique in CPR. Most likely other CPR techniques such as full-depth slab repair, slab stabilization, grinding or joint and crack resealing would be used in concert with retrofit longitudinal edgedrains to provide complete upgrading of the pavement. The engineer must coordinate the construction schedule so that the retrofit longitudinal edgedrains will dovetail with other CPR techniques.

Regardless of which type of retrofit edgedrain is selected, it is a good practice to seal all joints and cracks so that the amount of water infiltrating into the pavement structure is kept to a minimum.

4.2 Design Analysis

In any design analysis of existing concrete pavement rehabilitation, there are three steps that must be analyzed to determine if the proposed design will accomplish its goal of pavement drainage.

The first step in the design analysis of retrofit longitudinal edgedrains is to identify the water that is to be drained. Retrofit longitudinal edgedrains will drain water that enters the pavement/shoulder joint and any water that infiltrates the concrete pavement slabs and collects along the slab/base interface. This is free water that follows the path of least resistance and is strongly influenced by the affects of gravity. Any water that enters and ultimately saturates an impermeable dense graded aggregate base course will not be drained by a retrofit longitudinal edgedrain in a reasonable amount of time.

The second step is to evaluate the erodibility of the subbase material. The best guide for evaluating is past experience with the particular subbase material. If the subbase contains a high percentage of material passing the No. 200 sieve, the subbase will probably be highly erodible. As noted previously, a filter fabric does not prevent fines from being eroded from the subbase material, it only controls what happens to the fines after they migrate to the trench area. If an excessive amount of fines are eroded from the base course, any retrofit edgedrain will probably not be effective in extending the pavement life.

The third step is to determine if there is enough relief provided by the cross-section of the highway surface to provide positive drainage to the roadside ditches. Subsurface and surface drainage must be coordinated.

4.3 Unresolved Issues of Drainage Design

Currently, there are two unresolved issues of drainage design; filter fabric placement and trench backfill permeability. Filter fabric placement was previously discussed in Section 3.6, The three design approaches for filter fabric placement are; completely wrapped trench, partially wrapped trench, and wrapped pipe with sand backfill. There are advantages and disadvantages to each approach as discussed in section 3.6. Any trench backfill must be permeable enough to transmit the accumulated water to the drainage pipe. The backfill must also be stable enough to resist the loads applied to it. In the wrapped pipe with sand backfill approach, the sand backfill will filter the eroded fines preventing the filter fabric around the pipe from clogging. Unfortunately, it is believed that most of the sand backfill currently used does not have enough permeability to rapidly drain the section and significantly reduce the time of saturation. A coarse aggregate backfill will have the necessary permeability to drain the pavement section keeping saturation time to a minimum. Use of asphalt or cement treated backfill will increase stability with little decrease in permeability.

It is hoped that the findings of Experimental Project No. 12 will provide positive guidance to help resolve these issues.

4.4 Design Details

Listed below is a consensus that was developed on the design elements for retrofit longitudinal edgedrains based on this review:

- The edgedrain should be located under the shoulder immediately adjacent to the pavement/shoulder joint.

- Remembering that the filter fabric does not prevent erosion of fines from under the pavement slab, based on our observations, it is believed that the second approach, the partially wrapped trench, is the most promising compromise of design factors. By eliminating the filter fabric at the subbase/edgedrain interface, eroded fines can not clog the filter fabric. This approach will maximize the drainage of the pavement section keeping saturation time to a minimum.

- Trench backfill should be permeable enough to transmit water to the longitudinal edgedrain pipe and it must be stable enough to withstand traffic loads. Asphalt or cement treated backfill increases stability with little or no loss of permeability.

- The most commonly used trench width was 12 inches. The trench depth is determined by the vertical location of the pipe. Locating the top of the pipe at the bottom of the layer to be drained is recommended. This ensures that the flow zone of the pipe is below the layer to be drained.

- Since the purpose of the edgedrain system is to rapidly remove free water from the pavement structure, the outlet spacing should not exceed 500 feet, in most cases. The length of cleaning equipment available may dictate the outlet spacing. Shorter spacing eases maintenance of the edgedrain system, however, more outlets are the result. Conversely, greater spacing lengthens the time to drain. Additional outlets should be provided at the bottom of sag vertical curves.

- Because of the tendency of flexible corrugated plastic pipe to curl, use of rigid PVC pipe is recommended for outlet laterals. Rigid PVC pipe helps to maintain the proper outlet pipe grade and provides more protection from crushing. A few States included in this review have since modified their outlet design specifying the use of rigid PVC.

- Headwalls protect the outlet pipe from damage, prevent slope erosion, and ease in the locating of the outlet pipe. Because, these factors are so important in edgedrain design, the use of headwalls is recommended.

- Use of removable rodent screens is recommended. Removable screens ease cleaning of the screen itself as well as the edgedrain system.

- Since vegetative growth can quickly obscure the outlet pipe, reference markers are also recommended.

5.0 FUTURE ACTIVITIES

The study of pavement drainage is an on-going activity. The first step is the completion of Experimental Project No. 12. After the effectiveness of retrofit longitudinal edgedrains has been determined, FHWA will be in a good position to provide guidance to the field. When pavement design and rehabilitation reviews are conducted in a State, the pavement drainage designs can also be reviewed so that a nationwide assessment can be developed. Most likely a combination drainage demonstration project and training package will be developed. This will allow FHWA to provide needed technology transfer for this important pavement engineering item.

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PERMEABLE BASE

DESIGN AND CONSTRUCTION

BY

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A paper prepared for presentation at the "Fourth International Conference on Concrete Pavement Design and Rehabilitation," April 18-20, 1989 at Purdue University.

INTRODUCTION

This paper will present the state-of-the-practice in pavement drainage (i.e.. permeable bases) for new or reconstructed asphalt concrete (AC) and Portland cement concrete (PCC) pavements.

Rather than using impermeable dense-graded materials many States have gone to using open-graded or "permeable" bases to allow infiltrated moisture to rapidly drain through the base and out from beneath the pavement structure.

Because of the relative unfamiliarity with permeable base pavement structures and with the varying designs in use, this paper synthesizes permeable base pavement systems being used in this country.

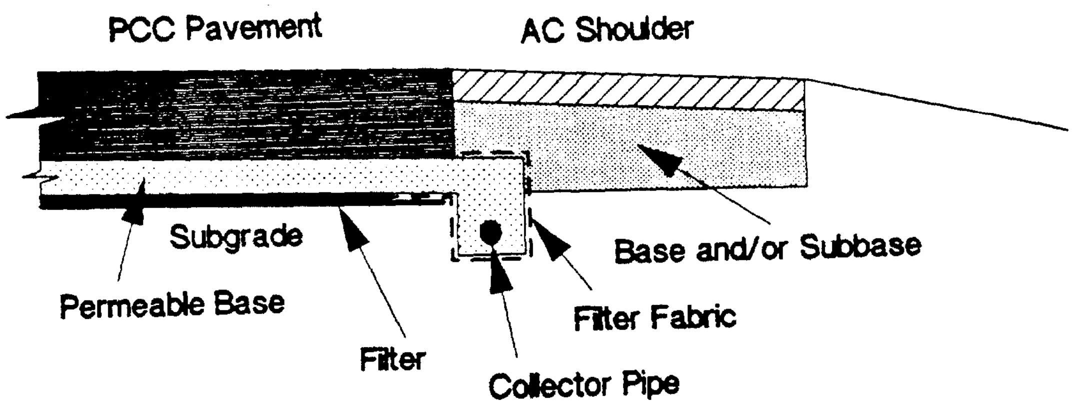
BACKGROUND

The pavement structure is the most costly element of the highway system and its premature failure is of major concern. Among the reasons cited for pavement failures, inadequate drainage of the pavement structure has been identified as a primary cause of pavement distress. The newly published AASHTO Guide for the Design of Pavement Structures (1986) addresses this as a problem by including drainage as an essential element of pavement design. Also, the Federal Highway Administration's (FHWA) pavement management and design policy encourages performing a drainage analysis for each new, rehabilitation, and reconstruction pavement design.

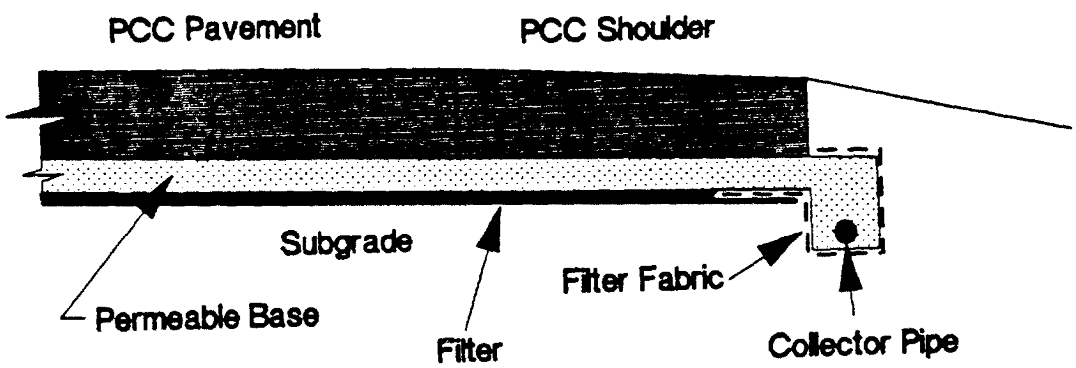
In designing pavement sections in the past, the primary function of the base was to provide uniform support. However, with increasing traffic loads, erosion and pumping of the underlying material resulted. This led to construction of what were thought to be strong nonerodible bases (i.e.. dense-graded aggregate bases, cement treated bases, asphalt concrete bases). These materials were not only impermeable, they were also found to be erodible in many cases. Infiltrated moisture was trapped in the pavement structure and, under the effects of heavy loads, led to a weakening or erosion of the base, subbase, and/or subgrade often resulting in premature distress of the pavement structure.

A significantly different pavement design philosophy is now receiving a great deal of consideration. Rather than using impermeable dense-graded materials, several States have opted to use open-graded or permeable bases to allow infiltrated moisture to rapidly drain through the base and out from beneath the pavement structure. A permeable base is normally characterized by an open-graded crushed angular aggregate with essentially no fines. Recognizing the problems moisture distress has played on pavements, primarily on PCC pavements, many States are routinely using or experimenting with permeable bases beneath new or reconstructed high-type pavements. A longitudinal edgedrain collector system is commonly used to rapidly drain the moisture that collects in the permeable base. Typical permeable base pavement sections are shown in Figure 1.

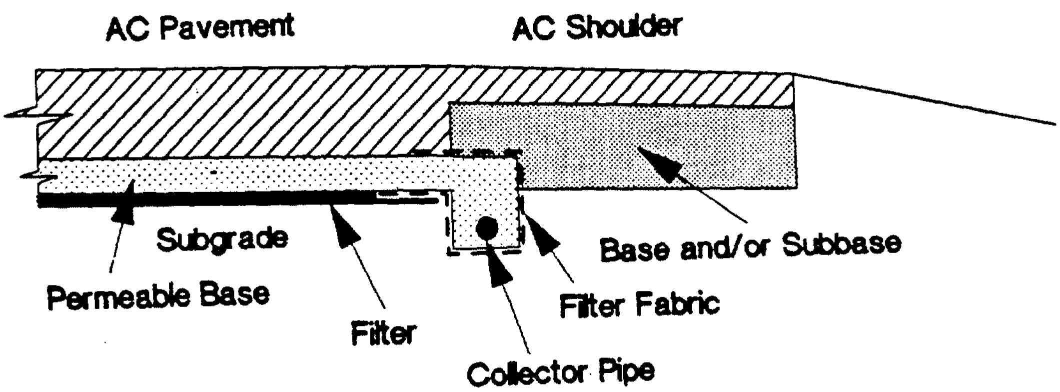
Figure 1. Typical Permeable Base Pavement Sections



PCC Pavement/AC Shoulder Section



PCC Pavement/PCC Shoulder Section



AC Pavement/AC Shoulder Section

OBJECTIVE

The primary objective of this paper is to synthesize the design and construction of permeable base pavement systems being used in this country. It is the intent to summarize the findings, to communicate the experiences of various States, and to demonstrate that permeable base pavements can be designed and constructed without significant changes to conventional practices.

SCOPE

Reviews were conducted in those States that were known to have recently constructed permeable base pavements. States included in the review were California, Iowa, Kentucky, Michigan, Minnesota, New Jersey, North Carolina, Pennsylvania, West Virginia, and Wisconsin. The review included gathering information from each State in design, use, construction, cost, and performance of permeable bases.

FIELD SURVEY RESULTS

In general, the States that are using permeable base pavements can be grouped into two categories -- those that use an untreated permeable base and those that use a treated permeable base. The untreated permeable base materials generally have a lower coefficient of permeability, whereas treated permeable bases have a much higher coefficient of permeability. The untreated permeable base material contains more smaller sized aggregate to give it stability and, thus, it tends to be less permeable. On the other hand, a treated permeable base had a cementing agent, generally 2-3 percent asphalt cement, for stability. The result was a more open material with high permeability.

Summary of State's Philosophy on Pe able Base Pavement Design

The approach to permeable base design varied among the States reviewed with California, Michigan, New Jersey, and Pennsylvania having the most experience. Most of the other States constructing permeable bases investigated the designs used by these States and modified them for their own use. The majority of States are primarily using permeable bases beneath PCC pavements, however, several States are using permeable bases beneath AC pavements as well.

Although the philosophies differ with respect to degree of permeability, the end result is that all States believe that rapid base drainage is extremely important. Some States believed that the highest permeability that could be obtained with readily available materials was best. Whereas, other States believed that a less permeable material which was similar to their existing base material in availability, cost, and stability, but which had some of the fines removed to provide drainability, was sufficient.

Review of Current Permeable Base Pavement Design

Type of Permeable Base

Those States that are predominantly using untreated permeable bases include: Iowa, Kentucky, Michigan, Minnesota, New Jersey, Pennsylvania, and Wisconsin. Iowa's, Minnesota's, and Pennsylvania's permeable base gradation is essentially derived from their conventional dense-graded aggregate base gradation with some of the fines removed. Kentucky's and New Jersey's gradations are based on readily available AASHTO aggregate gradations (i.e., Kentucky uses the AASHTO No. 57 stone and New Jersey uses a 50/50 blend of AASHTO No.'s 57 and 9 stone). Michigan's and Wisconsin's gradations were developed through testing of various permeable gradations. Both Iowa and Michigan allow recycled PCC pavement with some of the fines removed to be used for their permeable base.

Those States that are using treated permeable bases include California, North Carolina, and West Virginia. The predominant material used for stabilization is asphalt cement at approximately 2 percent, although California allows Portland cement at 2-4 bags per cubic yard as an option. Both North Carolina and West Virginia utilize AASHTO's No. 57 stone gradation. California's gradation is similar.

Degree of Permeability

There was a wide range in permeabilities desired. The untreated permeable bases generally had a lower coefficient of permeability -- in the range of 200 to 3,000 feet per day. The treated permeable bases all had a very high coefficient of permeability -- from 3,000 to 20,000 feet per day or higher. The permeabilities were determined using either a falling head or constant head permeameter using standard test procedures. The gradations used by the 10 States reviewed for the treated and untreated permeable bases, respectively, follow.

TREATED PERMEABLE GRADATIONS

Sieve Size	Percent Passing California	Percent Passing North Carolina/West Virginia
1 1/2-inch	. -	100
1-inch	100	95-100
3/4-inch	90-100	-
1/2-inch	35-65	25-60
3/8-inch	20-45	-
No. 4	0-10	0-10
No. 8	0-5	0-5
No. 200	0-2	0-2
Coefficient of Permeability (feet per day)	15,000	20,000

UNTREATED PERMEABLE GRADATIONS

Sieve Size	IA	KY	MI	MN	NJ	PA	WI
				Percent Passing			
2-inch	-	-	-	-	-	100	-
1 1/2-inch	-	100	100	-	100	-	-
1-inch	100	95-100	-	100	95-100	-	100
3/4-inch	-	-	-	65-100	-	52-100	90-100
1/2-inch	-	25-60	0-90	-	60-80	-	-
3/8-inch	-	-	-	35-70	-	35-65	20-55
No. 4	-	0-10	0-8	20-45	40-55	8-40	0-10
No. 8	10-35	0-5	-	-	5-25	-	0-5
No. 10	-	-	-	8-25	-	-	-
No. 16	-	-	-	-	0-8	0-12	-
No. 30	-	-	-	-	-	0-8	-
No. 40	-	-	-	2-10	-	-	-
No. 50	0-15	-	-	-	0-5	-	-
No. 200	0-6	0-2	-	0-3	-	0-5	-
Coefficient of Permeability (feet per day)	500	20,000	1000	200	2000	1000	18,000

Extent of Use

Nine of the 10 States use their permeable base under new or reconstructed high-type PCC pavements. Also, most States have constructed at least one permeable base AC pavement experimentally. Kentucky has only constructed permeable bases under AC pavements to date. It has been within the past 5 years that permeable bases beneath high-type PCC pavements has become standard in these States, with California specifying them beneath AC pavements, as well.

Thickness and Width of Permeable Base

The thickness of permeable bases varied from 3 to 6 inches, with 4 inches being the most common. Although the thickness required for drainage can be calculated, 4 inches seems to provide sufficient capacity, is easily constructed, and provides for construction tolerances. California specifies 0.25-feet (3 inches) for its asphalt cement treated permeable base and 0.35-feet (approximately 4 inches) for its Portland cement treated permeable base. The difference in thickness specified is attributed to the asphalt cement treated permeable material having a higher coefficient of permeability -- approximately 15,000 feet per day -- than the Portland cement treated material -- approximately 4,000 feet per day.

The width of permeable base, whether treated or untreated, was generally placed 1 to 3 feet outside either pavement edge. In most cases, the tracks of the paver ran on this widened section. Kentucky, New Jersey, and West Virginia carried the permeable base layer out to the edge of either shoulder.

Method Used to Drain Permeable Base

All States reviewed use a longitudinal edgedrain collector system to drain accumulated water from their permeable bases. Seven of the 10 States used an excavated trench design exclusively. Kentucky, West Virginia, and Wisconsin have also used a V-ditch design for the longitudinal edgedrain collector. Both Kentucky and West Virginia noted problems with this design. Not only is constructing and maintaining the V-ditch a problem, but protecting the pipe from crushing under construction traffic was also noted as a problem. Several States that use the excavated trench design also expressed a concern with possible crushing of the pipe, however, there is generally more cover over the pipe than with the V-ditch design. Both Minnesota and Wisconsin allow the contractor to construct the longitudinal edgedrain collector system either before or after pavement construction. They were concerned with the possible damage to the pipes in the longitudinal trenches and the outlet lateral trenches by construction equipment.

Generally, the inside edge of the edgedrain trench is located immediately below the longitudinal pavement/shoulder joint (see Figure 1). To avoid settlement or crushing of the collector pipe beneath construction equipment, several States locate the trench 2-3 feet out from the joint beneath the shoulder. Michigan, however, installs the trench beneath the PCC pavement.

Most States that construct crowned pavement sections install a longitudinal edgedrain collector system along both the inner and outer pavement edge. This effectively shortens the drainage path and significantly lessens the time for the permeable base to drain.

Most States backfill the edgedrain trench with the same permeable material that is used for the permeable base. A few used a more permeable material as backfill. All 10 States used an outlet pipe to convey the accumulated water from the edgedrain collector to the ditch or other inlet structure. West Virginia tried a fabric wrapped pea gravel outlet system. After several years, these outlets became increasingly difficult to locate because they had become overgrown with vegetation and/or plugged with roadside slope debris. Daylighting of the permeable base to the ditch slope is not recommended because of these reasons.

A number of States had experienced problems with maintaining the proper outlet grade with flexible corrugated plastic pipe and now specify the use of rigid PVC pipe for outlet laterals. Iowa is the only State that does not use a filter fabric lined edgedrain collector trench. The subbase and subgrade material acts as a filter and is compatible with the permeable trench backfill material. Also in Iowa, edgedrains are installed 4 feet below the pavement surface and are generally installed 2-3 years prior to reconstruction, primarily to drain the subbase and subgrade before reconstruction.

Type of Filter Layer Used

Those States that use an untreated permeable base use a filter aggregate layer, which in most cases, is the States's conventional dense-graded aggregate base material. The gradation of this material is compatible with the permeable material to prevent intrusion of fines from the subgrade. Those States that use a treated permeable base predominantly, use a filter fabric (primarily non-woven) to protect the permeable base layer from intrusion of fines. One State, West Virginia, allows the use of a woven fabric. It is interesting to note that research by Penn State University in the use of filter fabrics, found that filter fabrics act a as a wick or blotter actually holding moisture in the material immediately below the filter fabric and may act as an internal source of moisture.(1) California was the only State that used an impermeable aggregate subbase as a separator or filter layer with a treated permeable base.

Structural Value

Five of the seven States that predominantly use an untreated permeable material believe it was structurally equivalent to a dense-graded aggregate base. New Jersey had gyratory shear and repeated load triaxial tests performed on their untreated and asphalt cement treated permeable materials at the Corps of Engineers Waterways Experiment Station (WES). Results indicated that both had bearing capacities similar to dense-graded aggregate base. Also, 1/2 million wheel loads were applied to the same test section which was subject to periodic flooding and it exhibited good performance.(2) Pennsylvania had tests performed on their untreated permeable material at the Penn State University Test Track and found that it provided support similar to a dense-graded aggregate base.(3) Kentucky and Minnesota do not give the untreated permeable material credit in their structural sections.

The three States that used a treated permeable base believed that the permeable material provided support similar to a dense-graded AC base. West Virginia performed a plate load bearing test on their first asphalt treated permeable base. A resultant K-value of 200 pounds per cubic inch (psi).(4) California performed laboratory compressive tests on their asphalt cement treated permeable material and found that it provided more support than dense-graded aggregate material.

Review of Current Permeable Base Construction Practices

Construction Considerations

Overall, construction of permeable base pavements requires more care than unstabilized or stabilized dense-graded aggregate bases. The treated permeable bases have sufficient stability for construction traffic, however, extra care is needed to prevent contamination of the layer. Untreated permeable bases, although sufficiently stable to pave on, are more easily displaced than dense-graded base. Additional care is required by equipment operators and truck drivers when placing and finishing the pavement. Quite often, a roller was used to "dress up" the permeable material immediately in front of the paver.

Most States restrict construction equipment other than the paving and finishing equipment from traversing the permeable base. Also, most States found that when placing an AC pavement on a permeable base, rubber-tired pavers rutted and displaced the permeable material. They now specify tracked pavers which better distribute the

Another concern with the untreated permeable aggregate material was the possible segregation of the material during placement and degradation of the aggregate under construction traffic. Several States specify that untreated permeable aggregate be placed at a certain percent moisture to reduce segregation.

The grade of the treated permeable materials was more difficult to modify once it had been placed and compacted. High and/or low spots at the longitudinal joint between asphalt cement treated paving passes was common and some method of modifying the grade (i.e., trimming with a blade or autograder) was required. Also, keeping the highly permeable base material clean and free from contamination was a concern. Both North Carolina and West Virginia require that the filter fabric between the subgrade and permeable base layer be wrapped or lapped up around both edges of the permeable base. California required sufficient filter fabric to line the edgedrain collector trench and to wrap up and over the low side of the permeable base layer.

Equipment Modifications

Only very minor equipment modifications are required to more easily construct permeable base pavements. One modification noted in a couple States was the use of wider rubber tires on the reinforcing mesh cart (for jointed reinforced concrete pavements (JRCP)) to distribute the load over a larger area of the permeable base, thereby, reducing the potential displacement of the untreated material. Also, as mentioned previously, when placing an AC pavement use of tracked pavers on untreated permeable bases in lieu of rubber-tired pavers was specified. In addition, use of longer pins to hold dowel baskets in place was necessary on permeable bases.

Stability

No stability problems were observed or indicated by any of the States that were reviewed. Many State and contractors' personnel expressed reservations regarding the paving on the more open-graded permeable treated or untreated base materials on their initial contact with it. However, in all cases, after working with the material the doubts vanished. All States required at least 85 percent crushed material which provided additional stability through aggregate interlock.

There were no problems noted with stability of the asphalt cement treated permeable materials under construction equipment even under high ambient air temperatures. All three States used a conventional paving grade asphalt cement as the stabilizing material. California noted a problem on one project where the asphalt treated permeable base did not set up properly and took up to a week to provide sufficient stability to pave on. The State attributed this to the permeable aggregate temperature not being in the 275-375 degree F range specified at the time the asphalt cement was introduced.

As expected, stability was more of a concern with untreated permeable materials. Although stability varied from state to state and gradation to gradation, all untreated permeable materials were stable under paving equipment. However, many States did not allow any equipment other than that needed to place and finish the paving to traverse the material. Those States that did allow construction traffic on the untreated base, required a roller in front of the paving operation to compact and smooth out any disturbance to the material from trucks hauling on the base. Both Minnesota and Pennsylvania require a minimum coefficient of uniformity (D_{60}/D_{10}) of 4 to ensure a stable gradation.

Performance of Existing Permeable Base Pavements

Performance information available to date indicates that properly designed and constructed permeable bases virtually eliminate pumping, faulting, and cracking. There is no long-term performance data available (in excess of 15 years). However, based on a comparison of the performance of existing permeable base sections to undrained sections. States anticipate a 50 percent increase in PCC pavement service life.

California continues to evaluate their permeable base pavements versus those that are not drained. On PCC, they found that in terms of percent cracked slabs, the permeable base (drained) sections had significantly lower rates of slab cracking. Of the four permeable base sections evaluated, three of which were constructed in 1980, two of the permeable base sections had no cracking, whereas the undrained control sections had 18 and 47 percent cracking. A drained section constructed in 1965 had 5 percent cracking compared to 10 percent for the undrained section. One project with both a permeable base and an undrained section had exhibited no cracking yet. A 500-foot section of AC pavement on a logging road was reconstructed in 1967 using a highly permeable open-graded base drainage layer after it had failed twice in just a few years. The State conducted a review in 1986 and found that the original pavement was still in excellent condition with no patching, whereas, the adjoining pavement had been extensively patched. The 19-year service life of this section (to date) is well beyond the normal 12-year life of AC pavements in California. Studies performed by California suggest a minimum service life increase of 33 and 50 percent, respectively, for AC and PCC pavements constructed on a permeable base.(5)

Iowa has performed a substantial amount of nondestructive testing (NDT) with a Road Rater on their PCC pavements. They indicated that permeable base pavements provide greater structural support than undrained pavements. The support on undrained pavement sections deteriorates for approximately 5 years, whereas support on permeable base sections does not deteriorate. They felt the constant support of the permeable base sections was equivalent to 3 to 5 inches of effective pavement. In addition, crack surveys which are performed every 2 years revealed that permeable base pavements have virtually no cracks, unlike conventional undrained pavements of the same age.

Michigan's oldest permeable base sections are on the Clare test road (US 10) constructed in 1975. An inspection of the three 1/2-mile permeable base test sections in comparison to the other sections was conducted during the review. There was no faulting or cracking and less apparent D-cracking on the permeable base sections than on the other two base types (i.e., bituminous base and dense-graded aggregate base). The dense-graded bituminous base sections were the worst performing in terms of pavement distress (i.e., faulting, cracking, D-cracking, and spalling). Pumping was noted on these sections as well. Some spalling of the longitudinal joints was noted on all test sections, but was noticeably worse on the bituminous base sections.

Minnesota's oldest permeable base pavement section, a 1600-foot section of JRCP with 27-foot skewed dowelled joints on Trunk Highway 15 near Fairmont constructed in 1983, was evaluated. After 5 years, only one of the 59 permeable base slabs had a mid-panel crack, whereas the undrained JRCP with conventional dense-graded aggregate base adjacent to either end of the pavement was found to have approximately 50 percent mid-panel cracking. The section adjacent to the south had 15 of 33 slabs that exhibited mid-panel cracks and the section north had 5 of 10 slabs with mid-panel cracks.

New Jersey reported the performance of their experimental permeable base pavement sections constructed in 1979-1980 at the 1988 Transportation Research Board Meeting. Their initial observations/findings on the AC sections were that the thinner sections were performing as well as the thicker sections with rutting being about the same. On PCC pavement sections, there was less deflection, no faulting or pumping, and substantially reduced frost penetration.

Pennsylvania rated the performance of their experimental permeable base sections constructed in 1980 much better than dense-graded aggregate base sections. Based on the positive interim results of these sections, a permeable base layer between the PCC pavement and dense-graded aggregate subbase became the State standard in 1983.(3)

Rideability

All of the States indicated that the rideability of permeable base pavements was no different than that on dense-graded bases. This was substantiated in California and North Carolina (asphalt cement treated) and Michigan (untreated). The rideability of some recently constructed PCC pavements in these States had been measured using the California and Rainhart profilographs at 0-5 inches per mile. In general, those States using a stringline for both horizontal and vertical control had a substantially better ride quality than those that did not. Also, those States that had incentives/disincentives for rideability had projects with very good ride quality.

Cost

Bids for permeable base materials were generally found to have slightly higher costs per unit weight than the impermeable dense-graded materials they replaced. Five of the seven States that used an untreated permeable base found that they were slightly more costly per unit measure than conventional dense-graded aggregate bases while two States, Iowa and Michigan, indicated that the unit costs for their permeable base material were the same or sometimes less.

As expected, the treated permeable base materials were two to three times more costly per unit measure than conventional dense-graded aggregate bases. However, all three States that predominantly used treated permeable base material found that the unit costs for it were about the same as those for dense-graded AC base. In addition, all three noted that because of the higher void content of the permeable material, the yield was 15-30 percent higher than dense-graded AC. California found that asphalt cement treated permeable base was generally less costly per unit measure than cement treated base (CTB) and lean concrete base (LCB). The material unit costs were the same or slightly more than asphalt concrete base but because of the large void content the yield was 20 percent higher. Kentucky, which had used some asphalt treated permeable base within the past year, also found that its material unit costs were about the same as the dense-graded bituminous base, but the permeable base material had 25-30 percent higher yield.

SUMMARY

A review of current design and construction practices has proven that permeable base pavements can be designed and constructed to rapidly drain moisture that infiltrates the pavement surface without significant changes to conventional practices.

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2. "Design and Implementation of Internal Roadway Drainage," by V. Mottola, New Jersey Department of Transportation, discussion of paper entitled "Subsurface Drainage Systems," by G. W. Ring, Federal Highway Administration, in the proceedings from the National Crushed Stone Association Conference on Crushed Stone for Road and Street Construction and Reconstruction, June 14-15, 1984.

3. "Subbase Permeability and Pavement Performance," by K. L. Highlands, and G. L. Hoffman, Pennsylvania Department of Transportation, paper No.870717 prepared for the 67th Annual TRB Meeting, January 1988.

4. "Evaluation of a Free Draining Base Course," by J. S. Baldwin and J. G. Jarvis, West Virginia Department of Highways, February 1986.

5. "Economic Impact of Pavement Subsurface Drainage," by R. A. Forsyth, G. K. Wells, and J. H. Woodstrom, California Department of Transportation, paper prepared for presentation at the annual meeting of the Transportation Research Board (TRB), January 1987.

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

DATE: June 8, 1989

FIELD TRIP REPORT

TO: Mr. Lou Papet, Chief
Pavement Division

FROM: Mr. Daniel M. Mathis
Highway Engineer

THRU: Mr. Paul Teng, Chief
Pavement Design & Rehabilitation Branch

Mr. John P. Hallin
Team Leader

INCLUSIVE DATES: April 11-14, 1989

PURPOSE: To participate in a pavement edgedrain review.

ACCOMPLISHMENTS OR RESULTS:

The State noted staining from fine soil particles on the shoulder of several recently cracked, seated, and overlaid Portland cement concrete (PCC) pavement sections. These appeared to be the result of moisture being pumped through the asphalt concrete (AC) overlay in the vicinity of the lane/shoulder joint. Since edgedrains were installed on these sections as part of the crack and seat project, the State felt that an investigation was warranted to determine if the drains had failed.

On Tuesday afternoon (4/11), we were briefed on the pavement staining problem and the State's work plan for investigation. They met with individuals from the State to discuss the procedure and operations for the investigation of the pavement edgedrain and pavement structure which were to take place on Wednesday and Thursday. The majority of shoulder staining was occurring on cracked and seated and overlaid PCC projects which were constructed in 1988 using a geocomposite edgedrain manufactured by Advanced Drainage Systems known as AdvanEdge. The State was considering a ban on this particular geocomposite edgedrain, but was encouraged to undertake an investigation of the problem in the field.

On Wednesday morning, numerous edgedrain outlets were observed on a section which exhibited some of the heaviest shoulder staining. All of the outlets were clear and functioning as evidenced by the red stains from eroded subgrade fines on the concrete headwall. At several outlets, a crystalline growth was observed on the rodent screens and outside the outlet pipe. It was speculated that this was the result of latent calcium carbonate precipitate being released from the cracked and seated PCC. Although all the outlets observed had drained, several of the flexible outlet pipe laterals had a slight reverse grade which inhibited the flow from the edgedrain system.

After traffic control had been set up, four cutouts approximately 9 inches square were jack hammered through the shoulder pavement to expose the top of the geocomposite edgedrain for observation and for insertion of the borescope. The edgedrain was located approximately 1-foot from the longitudinal pavement/shoulder joint interface area as shown in figure 1. In the afternoon, the operation moved to another location further south in the vicinity of post mile 109 to borescope another section of geocomposite edgedrain where the staining was apparent. Again, four 9-inch square cutouts were made to expose the top of the geocomposite edgedrain. The top of the edgedrain was cut open to expose the inside and to allow insertion of the borescope. Again the edgedrain was found to be approximately 1-foot away from the edge of the PCC pavement. In both areas borescoped, fines were observed in the bottom of the edgedrain core and a small amount of water was observed as well. Once the borescope was inserted into the edgedrain, fines could be observed coming from the slots with the bottom row of slots in many cases being completely silted up. Fines were observed adhering to the inside of the geotextile encapsulating the plastic core and being carried away in the outflow in the core. The geocomposite edgedrain did not show any signs of crushing. Also, fine material at 3/4 to 1 inch depth was noted in the bottom of the edgedrain.

On Thursday morning, a section of the right shoulder in the vicinity of post mile 108 (southbound) which exhibited the heaviest shoulder staining was excavated. A trench approximately 20 feet long, 3 feet wide, and 3 1/2 feet deep, 1-foot out from the pavement structure was excavated. Midway in the excavated trench, a 2-foot section of the geocomposite edgedrain (ADS' AdvanEdge) was carefully excavated and cut out from the edgedrain system for testing and evaluation. The bottom of the edgedrain contained 1 to 1 1/2 inches of silt/clay (minus No. 200 sieve) material. Excavation of the material between the edgedrain and the pavement structure ensued. Very little free moisture was apparent in the material surrounding the edgedrain or in the base material beneath the pavement and shoulder. This material was moist but not saturated until the excavation came within a half-inch of the pavement structure. Once the material had been removed adjacent to the PCC pavement, water was observed flowing through the cracks of the cracked and seated PCC pavement, along voids between the PCC and shoulder base material, and at the PCC/AC overlay interface. No moisture was observed traversing the slab/subbase interface as the PCC was well cracked and seated on the subbase. Observations of the AC overlay in the staining areas, revealed a high percentage of voids in the mix. Also, fines were observed adhering to the AC overlay aggregate throughout the base course and the surface course layers.

Two crude percolation tests were conducted on the AC overlay. A paper cup with the bottom cut out was placed on the surface and sealed with grease. Water was then poured into the cup and the movement of water in relation to a reference point was observed. The first test was performed in the right wheelpath and the second was performed near the lane lines between the two southbound lanes. Very little water percolated through the traffic compacted AC in the wheel path. However, the second area tested, which was not in the wheelpath, accepted water at a surprising rapid rate -- an inch of water in the 2 1/2-inch diameter cup percolated through the AC surface in approximately 30 seconds. Again this was a crude test but it gave a good indication of the permeability of the AC overlay. This suggests that poor compaction of the AC overlay is allowing water to permeate down through the overlay and the cracked and seated PCC before being pumped back up through the overlay and staining the shoulder.

A closeout discussion was held at the site. The recommendations suggested are those that appear below.

We then went on to review other previously cracked and seated and overlaid PCC pavements from this section on up to the stateline. Staining of the right shoulder was observed at isolated locations. The longitudinal edgedrain used on the sections varied from two different types of geocomposite edgedrain types to the State's conventional geotextile wrapped pipe edgedrain. The extent and degree of staining on these sections was less than that noted on sections on projects that were observed previously.

CONCLUSIONS:

The staining of the shoulder is the result of moisture infiltrating down through the insufficiently compacted AC overlay and into the cracked and seated PCC pavement. There it travels laterally through the cracks to the pavement/shoulder interface. The base material surrounding the cracked and seated PCC is dense-graded and impermeable. In addition, location of the edgedrain prevents the free flow of moisture from the cracked and seated PCC to the edgedrain. As a result, moisture is trapped in the cracked and seated pavement. Under traffic loadings, sufficient pressures are developed to pump water and fines through the AC overlay and out onto the surface.

RECOMMENDATIONS:

There were several recommendations suggested:

1) Retrofit longitudinal edgedrains, whether conventional trench or geocomposite, should be placed such that a large area of the edgedrain is in contact with the cracked and seated PCC pavement. Moisture cannot be effectively drained from a very dense impermeable material (i.e., the aggregate base or clay subgrade) and as a result more surface area of the edgedrain should be provided adjacent to the PCC pavement to drain the moisture that moves through the cracked pavement (see figure 2).

2) Additional attention to AC paving and compaction is recommended. Tighter more compacted AC pavement layers will reduce the amount of moisture infiltrating the pavement structure. This, in turn, will reduce other potential problems such as stripping of the asphalt cement from the aggregate from occurring.

3) It is recommended that rigid PVC pipe be used for outlet laterals to ensure a proper grade for the outlet.

4) It is recommended that various types of geocomposite edgedrains, conventional aggregate pipe edgedrains (with different geotextile placements), and a control section be evaluated to determine which if any edgedrain performs better and if retrofit longitudinal edgedrains themselves increase the service life of the pavement.

REMARKS:

The State and the FHWA Division should be commended for undertaking this type of investigation to determine the probable cause for the problem noted. Good engineering is extremely important in understanding the problem and in making sound decisions on modifications to design and construction practices and procedures to alleviate the problem.

ADDITIONAL INFORMATION:

On June 5, 1989 another section of cracked and seated and overlaid PCC pavement further north was excavated. This 9-inch PCC pavement had been rehabilitated with a geocomposite edgedrain and a 4-inch AC overlay approximately 1 year ago. A section of the pavement exhibited staining on the shoulder similar to that discussed previously. The geocomposite edgedrain used on this section was Monsanto's Hydraway. Again, it had been placed approximately 1-foot from the edge of the pavement, however, because of the different nature of the subbase material, water was able to flow through the material and into the drain. Outlets were located 200 feet south and 300 feet north of the excavation. When material surrounding the geocomposite edgedrain was excavated, water seeped through the geotextile and into the trench. A section of the geocomposite edgedrain was cut out for laboratory testing. When the section was removed, water drained into the trench from either end indicating that this section of edgedrain was located in a sag vertical curve with no outlet. The accumulated water ponded and filled the edgedrain. When sufficient pressure was exerted water and fine material was forced up through the AC overlay and out onto the shoulder. An edgedrain outlet was installed by State personnel at this location to rectify the problem.

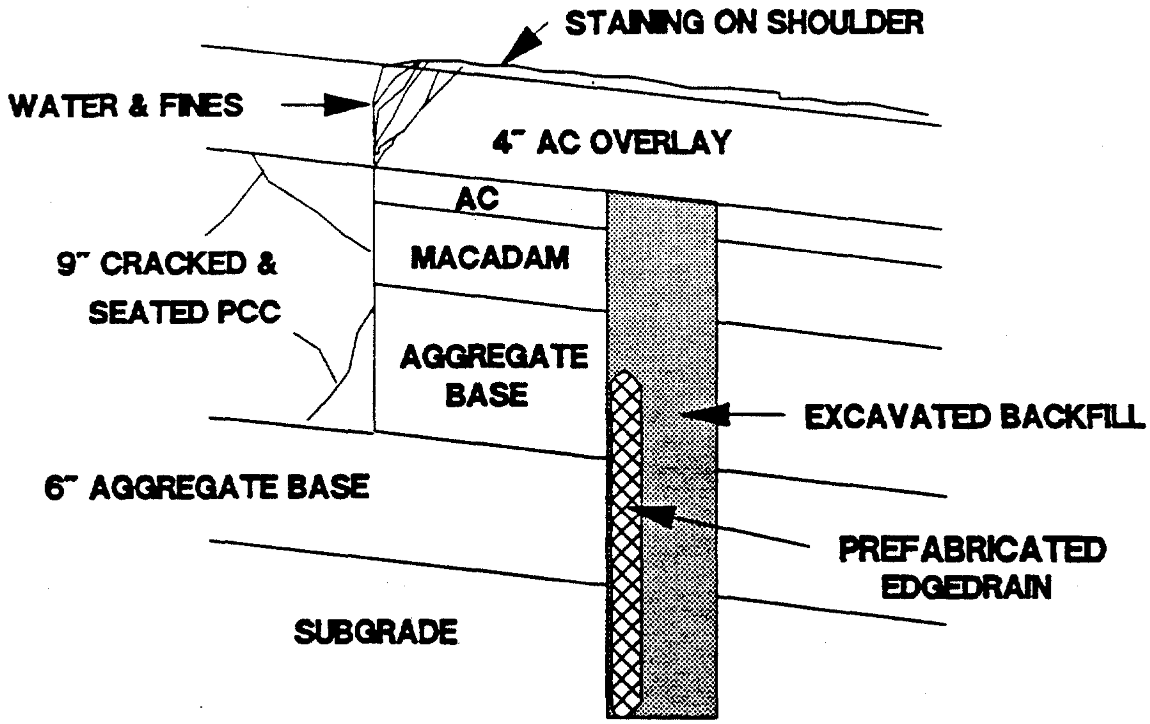


Figure 1. ACTUAL PREFABRICATED EDGEDRAIN PLACEMENT

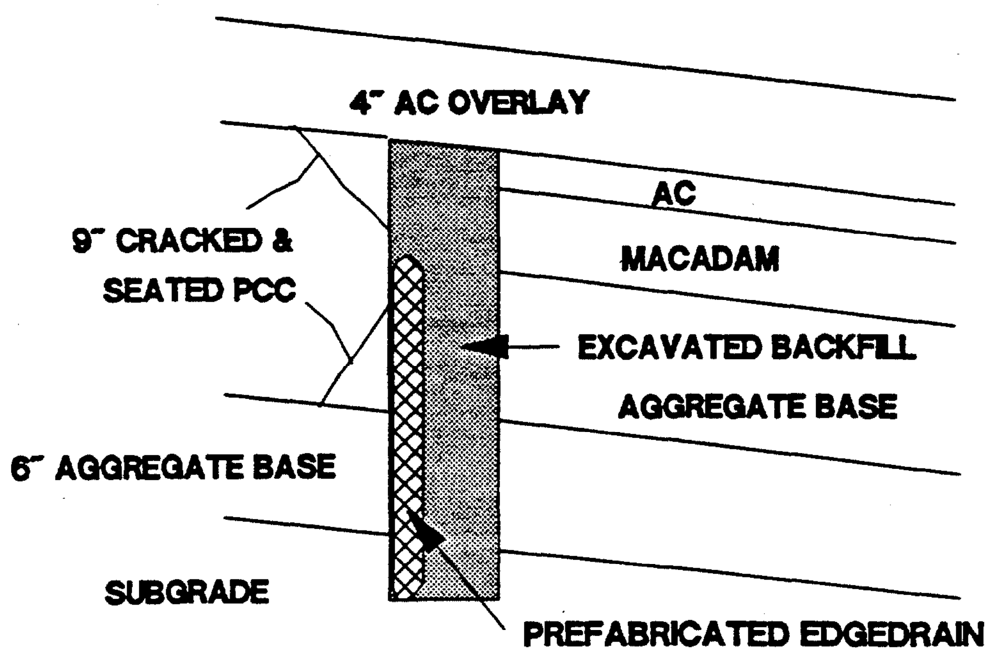
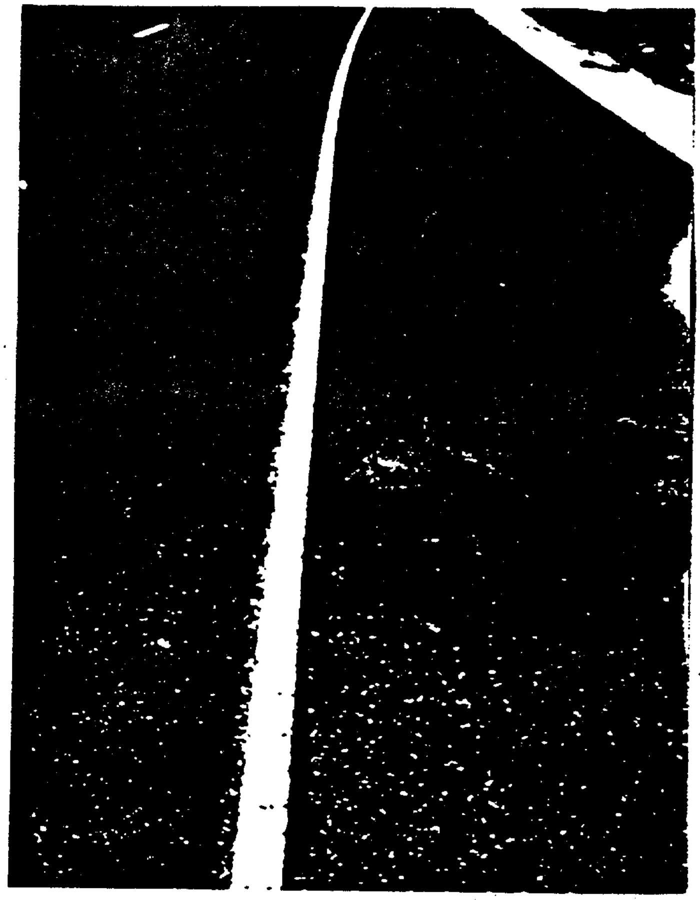
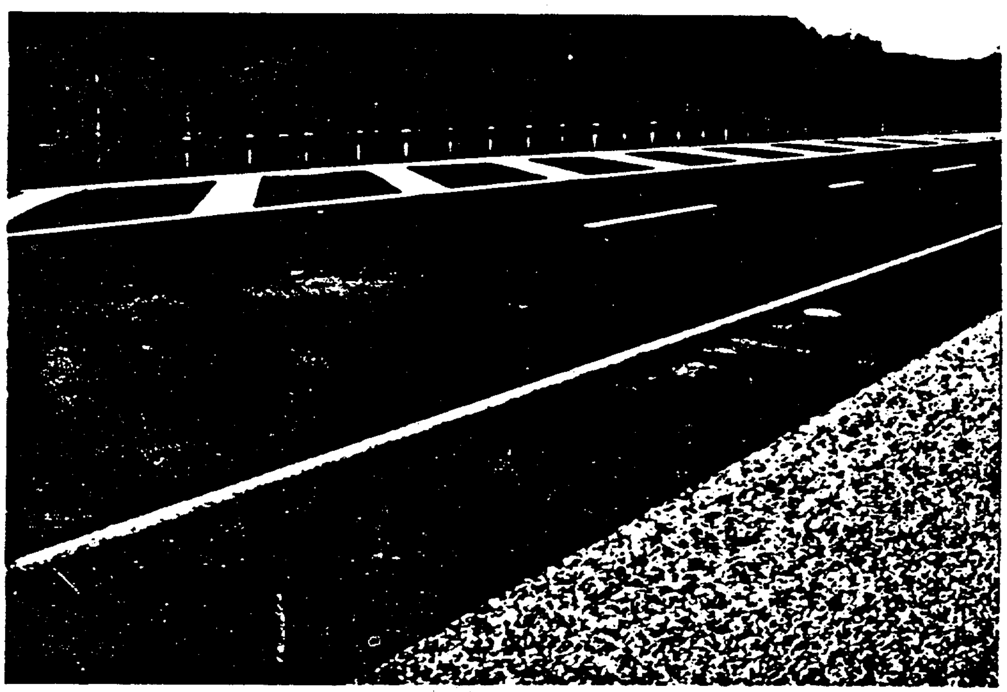


Figure 2. RECOMMENDED PREFABRICATED EDGEDRAIN PLACEMENT

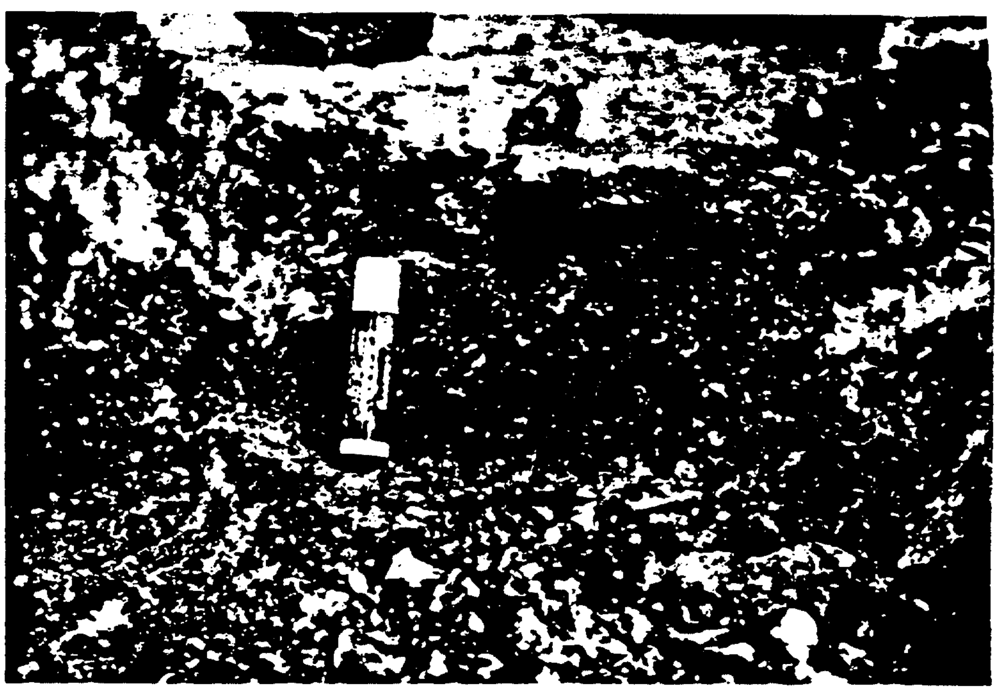


Subbase/subgrade fines staining shoulder.



Subbase/subgrade fines staining AC overlay on superelevated section.

Geocomposite edgedrain (lower left) located approximately 1-foot from cracked and seated PCC pavement (upper right). Impermeable material lies between edgedrain and pavement.



Subbase\subgrade fines adhering to AC overlay aggregate.

SUBSURFACE DRAINAGE
OF
PORTLAND CEMENT CONCRETE PAVEMENTS

WHERE ARE WE?

December 1991

BACKGROUND

The drainage of concrete pavements has been a significant activity in the Pavement Division, since the formation of the Division in 1986. The AASHTO Guide for the Design of Pavement Structures (1986) had been recently published and addressed drainage as an essential element of pavement design. The Federal Highway Administration's (FHWA) January 13, 1989 Pavement Policy encouraged a drainage analysis for each new, rehabilitation, and reconstruction pavement design.

During the summers of 1987 and 1988 reviews were conducted in those States that were constructing permeable bases under portland cement concrete (PCC) pavements. The States identified were California, Iowa, Michigan, Minnesota, New Jersey, North Carolina, Pennsylvania, West Virginia, and Wisconsin. During the previous 5 years permeable bases beneath high-type PCC pavements had become standard in these States, with California specifying them beneath AC pavements, as well. The review included gathering information from each State on design, use, construction, cost, and performance of permeable bases.

In general, the States that were using permeable base pavements could be grouped into two categories -- those that used an untreated permeable base and those that used a treated permeable base. The untreated permeable base materials generally had a lower coefficient of permeability, whereas treated permeable bases had a much higher coefficient of permeability. The untreated permeable base material contained more smaller sized aggregate to give it stability and, tended to be less permeable. On the other hand, a treated permeable base had a cementing agent, 2 to 3 percent asphalt cement or 2 to 4 bags of cement per cubic yard, for stability. The result was an open material with high permeability.

The approach to permeable base design varied among the States with California, Michigan, New Jersey, and Pennsylvania having the most experience. Most of the other States constructing permeable bases investigated the designs used by these States and modified them for their own use.

Although the philosophies differed with respect to degree of permeability, the end result was that the States believed that rapid base drainage was extremely important. Some States believed that the highest permeability that could be obtained with readily available materials was best. Whereas, other States believed that a less permeable material which was similar to their existing base material in availability, cost, and stability, but which had some of the fines removed to provide permeability, was sufficient.

States using untreated permeable bases were; Iowa, Michigan, Minnesota, New Jersey, Pennsylvania, and Wisconsin. Iowa's, Minnesota's, and Pennsylvania's permeable base gradation was derived from their conventional dense-graded aggregate base gradation with some of the fines removed. New Jersey's gradations were based on a 50/50 blend of AASHTO No.'s 57 and 9 stone. Michigan's and Wisconsin's gradations were developed through testing of various permeable gradations. Both Iowa and Michigan allowed recycled PCC pavement, with some of the fines removed, to be used for their permeable base.

States using treated permeable bases were California, North Carolina, and West Virginia. The predominant material used for stabilization was asphalt cement at approximately 2 percent, although California allowed portland cement at 2-4 bags per cubic yard as an option. Both North Carolina and West Virginia utilized AASHTO's No. 57 stone gradation. California's gradation was similar to the AASHTO No. 57.

There was a wide range of permeability. The untreated permeable bases generally had a lower coefficient of permeability -- in the range of 200 to 3,000 feet per day. The treated permeable bases all had a very high coefficient of permeability -- from 3,000 to 20,000 feet per day or higher. The permeability was determined using either a falling head or constant head permeameter following standard test procedures.

The States that used an untreated permeable material believed it was structurally equivalent to a dense-graded aggregate base. New Jersey had gyratory shear and repeated load triaxial tests performed on their untreated and asphalt cement treated permeable materials at the Corps of Engineers Waterways Experiment Station (WES). Results indicated that both had bearing capacities similar to dense-graded aggregate base. Also, 1/2 million wheel loads were applied to the same test section, which was subject to periodic flooding, and it exhibited good performance. Pennsylvania had tests performed on their untreated permeable material at the Penn State University Test Track and found that it provided support similar to a dense-graded aggregate base. Kentucky and Minnesota did not give the untreated permeable material credit in their structural sections.

The States that used a treated permeable base believed that the permeable material provided support similar to a dense-graded AC base. West Virginia performed a plate load bearing test on their first asphalt treated permeable base. A resultant K-value of 200 pounds per cubic inch (pci). California performed laboratory compressive tests on their asphalt cement treated permeable material and found that it provided more support than dense-graded aggregate material.

Overall, construction of permeable base pavements was found to require more care than dense-graded aggregate bases. The treated permeable bases had sufficient stability for construction traffic, however, extra care was needed to prevent contamination of the layer. Untreated permeable bases, although sufficiently stable to pave on, were more easily displaced than dense-graded base. Additional care was required by equipment operators and truck drivers when placing and finishing the pavement. Quite often, a roller was used to "dress up" the permeable material immediately in front of the paver.

Another concern with the untreated permeable aggregate material was the possible segregation of the material during placement and degradation of the aggregate under construction traffic. Several States specified that untreated permeable aggregate be placed at a certain percent moisture to reduce segregation.

The grade of the treated permeable materials was more difficult to modify once it had been placed and compacted. High and/or low spots at the longitudinal joint between asphalt cement treated paving passes was common and some method of modifying the grade (i.e., trimming with a blade or autograder) was required. Also, keeping the highly permeable base material clean and free from contamination was a concern. Both North Carolina and West Virginia required that the filter fabric between the subgrade and permeable base layer be wrapped or lapped up around both edges of the permeable base. California required sufficient filter fabric to line the edgedrain collector trench and to wrap up and over the low side of the permeable base layer.

PROMOTION

The permeable base reviews conducted during 1987 and 1988 revealed that permeable bases could be constructed without significant changes to conventional practices. However, there were questions raised that needed to be addressed. In Michigan early distress on several projects were partially attributed to the lack of stability of the permeable base. The paving contractors raised a number of issues including: the cost effectiveness of using permeable bases on lower volume routes, adequate stability for normal construction operations, and the availability of aggregate in all locations (since a crushed gap graded aggregate is required there is more waste in a gravel source). The States also have raised questions on needed permeability and stability and the lack of long term performance data.

Reviews of existing pavement subsurface drainage systems indicated a general lack of maintenance. In every State visited, outlets were found that were completely plugged. There was a concern that, unless the maintenance of the outlets was given high priority, the use of permeable bases could cause more harm than good. An undrained permeable base would become a large reservoir of water under the pavement which could saturate and weaken the subgrade.

Since the findings of the review were generally positive it was concluded that an effort should be undertaken to promote the use of permeable bases, while continuing to evaluate existing and ongoing projects. The promotional activities can be grouped into three areas: Conferences and Presentations, Issuance of Technical Guidance, and Development of Demonstration Project 87.

Conferences and Presentations

Since 1988 members of the Pavement Division staff have made presentations on the design and construction of permeable bases to numerous seminars and meetings across the United States such as: Region 3 Quality Assurance Workshop, University of Wisconsin Short Courses, Fourth International Conference on Concrete Pavement Design and Rehabilitation, Illinois Transportation Conference, Western Concrete Pavements Conference, Nevada Transportation Conference, Annual Convention of the National Stone Association. In addition multi state drainage conferences were held in

Williamsburg, Virginia; Memphis, Tennessee; Denver, Colorado; and Madison, Wisconsin. The Pavement Division has also provided technical assistance to many individual States.

Issuance of Technical Guidance

Technical Guide Paper 90-01 on Subsurface Pavement Drainage was issued November 15, 1990 to provide interim guidance. Originally the information contained in the guide paper was going to be issued as a technical advisory (TA). However, State and industry reviewers voiced concerns that the technology had not developed to the point where it should be included in a TA, given the "policy" status that a TA sometimes implies. We concurred with this viewpoint. We plan to issue a TA when procedures have been fully developed and evaluated. The purpose of the guide paper is to provide guidance on the current state-of-the-art for the design construction and maintenance of subsurface drainage systems.

The Concrete Pavement Drainage Rehabilitation State of the Practice Report was published in April 1989. This report summarized the current edgedrain practices in ten States.

Chapter 10, FHWA Pavement Rehabilitation Manual, Longitudinal Edgedrains was issued. This chapter examines subsurface drainage and the need for and the use of longitudinal edgedrains in relation to the design, construction, rehabilitation and maintenance of pavements.

Development of Demonstration 87.

This Demonstration Project is being developed to focus on the proper design, construction, and maintenance of permeable bases under PCC pavements. In April 1990, a meeting was held with participants from FHWA, States, and the concrete paving industry to discuss the best approach to follow in the development of a Demonstration Project for permeable bases. It was consensus of the participants that we should develop a demonstration project which presented the benefits of permeable bases, discussed proper design and construction procedures, and highlighted the importance of proper maintenance.

It was the conclusion of the group that the information contained in the Highway Subsurface Design Manual needed to be updated. Therefore, as part of the development of the demonstration project a new text is being written. The group also recommended that models be developed to demonstrate visually the velocity of flow through aggregate gradations with different coefficients of permeability. These models have been constructed and used at several persentations.

There was also the recommendation that we participate with Wisconsin in a study of cement stabilized open graded base (CSOGB). This study was undertaken in cooperation with the State and contractor. The study resulted in a better understanding of the relationship between cement content, strength, and the ability of CSOGB to carry construction traffic.

A pilot of the demonstration project presentation was given in November 1991. Based on comments received at the pilot, revisions are now being made. The demonstration project is expected to be ready for presentation in March 1992.

OTHER ACTIVITIES

The Pavement Division has worked closely with the Strategic Highway Research Program (SHRP) in the development of permeable base sections for SPS-1 and SPS-2. These sections reflect the state-of-the-art and should provide the information needed to verify the structural capacity and performance benefits of permeable bases.

A research contract is underway to evaluate the effects of various design features on the performance of jointed concrete pavements. Subsurface drainage is one of the features which is being evaluated.

RESULTS TO DATE

Nineteen of the 43 States and Territories, that build PCC pavements, routinely use permeable bases under their PCC pavements. An additional 12 States have constructed an experimental permeable base project or plan to construct one. Table 1 is a list showing which States construct PCC pavements and the type of bases used. In addition, 19 States and Puerto Rico have State funded or HPR studies involving improved pavement drainage.

CONCLUSIONS AND RECOMMENDATIONS

We believe that permeable bases provide a viable alternative for PCC pavements on higher volume routes where pumping and moisture related distress are the principle cause of pavement failure.

The technology is gaining wide acceptance as evidenced by the large increase in the number of States which are now using or plan to use permeable bases.

It is recommended that FHWA focus its activities on providing the States with information on the best available technology and the results of performance and design studies currently underway. We also need to continue to emphasize the importance of proper maintenance. This can best be accomplished through the demonstration projects program, emphasizing the need to consider drainage in our public presentations, and working with the States on a one-on-one basis. We need to continue to work closely with the States and contractors to be aware of their successes and failures, so the latest information is included in our presentations.

Table 1

STATES	USE PCC	PCCP TYPE			TYPE OF BASE						TRY PERM. BASE?	COMMENTS
					Dense Graded			Open Graded				
		CRCP	JRCP	JPCP	AGG	CTB	ATB	OGAB	ATPB	CTPB		
Alabama	N										N	
Arizona	Y	X		X	X						N	
Arkansas	Y			X	X						N	
California	Y			X					X	X		
Colorado	Y			X		X	X				Y	
Connecticut	Y		X		X						N	
Delaware	Y			X					X	X		
Dist. of Columbia	N										N	
Florida	N										N	
Georgia	Y		X	X	X	X					N	
Hawaii	Y			X		X					N	
Idaho	Y			X			X				N	
Illinois	Y	X		X		X	X				Y	
Indiana	Y		X	X	X						Y	
Iowa	Y			X				X				
Kansas	Y		X	X	X						Y	
Kentucky	Y			X				X	X			
Louisiana	Y			X			X				Y	Use drainable shoulder base
Maine	N										N	
Maryland	Y			X					X	X		

able 1

STATES	USE PCC	PCCP TYPE			TYPE OF BASE						TRY PERM. BASE?	COMMENTS
					Dense Graded			Open Graded				
		CRCP	JRCP	JPCP	AGG	CTB	ATB	OGAB	ATPB	CTPB		
Massachusetts	N										N	
Michigan	Y		X					X				
Minnesota	Y			X				X				
Mississippi	Y			X					X			
Missouri	Y		X		X						Y	Use drainable shoulder base
Montana	Y			X	X						N	
Nebraska	Y			X							N	Have drainable subgrades
Nevada	Y			X		X					Y	
New Hampshire	N										N	
New Jersey	Y		X						X			
New Mexico	N										N	
New York	Y		X		X						N	Prefer dense graded bases
North Carolina	Y			X					X			
North Dakota	Y		X	X				X				ATPB in future
Ohio	Y			X	X						N	
Oklahoma	Y	X								X		
Oregon	Y	X					X		X			
Pennsylvania	Y			X				X				
Rhode Island	N										N	
South Carolina	Y			X					X			
South Dakota	Y		X	X	X						Y	OGAB in 1992

Table 1

STATES	USE PCC	PCCP TYPE			TYPE OF BASE						TRY PERM. BASE?	COMMENTS
					Dense Graded			Open Graded				
		CRCP	JRCP	JPCP	AGG	CTB	ATB	OGAB	ATPB	CTPB		
Tennessee	Y			X					X			Considering No. 57 perm.
Texas	Y	X	X	X		X	X				Y	Two projects being built
Utah	Y			X		X					N	
Vermont	N										N	
Virginia	Y	X								X		
Washington	Y			X	X		X				N	Use perm base if needed
West Virginia	Y			X					X	X		
Wisconsin	Y			X						X		
Wyoming	Y							X	X			
Puerto Rico	Y			X			X				Y	

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: **INFORMATION**: Distribution of Proceedings Western States Drainable PCC Pavement Workshop

Date: AUG 10 1994

From: Director, Office of Engineering
For: Director, Office of Technology Applications

Reply to Attn of: HNG-42

To: Regional Administrators
Federal Lands Highway Program Administrator
ATTENTION: Technology Transfer Coordinators

The Federal Highway Administration, in cooperation with the California Department of Transportation along with the Southwest Concrete Pavement Association, sponsored the subject conference in Sacramento, California, during July 21-22, 1993. This memorandum transmits copies of the proceedings (Publication No. FHWA-SA-94-045) and provides you with an update on our pavement drainage efforts.

Presentations describing the design and construction procedures used in the construction of permeable bases were made by the various western State highway agencies (Arizona, California, Nevada, Oregon, Washington, and Wyoming). The proceedings were compiled by Mr. James H. Woodstrom of the Southwest Concrete Pavement Association.

Currently, we have completed presentations of Demonstration Project No. 87 (DP 87), "Drainable Pavement Systems" in 42 States, Puerto Rico, and the District of Columbia. This demonstration project primarily covered drainage of Portland Cement Concrete (PCC) pavements. Unfortunately, one of the reoccurring comments during the presentation was that it did not cover drainage of flexible pavements or retrofit longitudinal edgedrains.

On June 6-8, a Technical Working Group (TWG) on Flexible Pavement Drainage Design was convened to develop input for the design and construction of permeable bases for flexible pavements. Discussions and input from the TWG are being reviewed by the Pavement Division and a design consensus will be formulated. This guidance will be provided to the field.

The National Highway Institute will also incorporate this new guidance on flexible pavement drainage design in its new NHI Course No. 13126, "Pavement Subsurface Drainage Design." This training course will be a complete drainage

FEDERAL HIGHWAY ADMINISTRATION 1893 1993

package covering PCC and flexible pavements and retrofit longitudinal edgedrains. A Request for Proposal for the course has been developed and has been forwarded to the Office of Contracts and Procurement. The development time will be approximately 2 years.

Sufficient copies of the publication have been distributed to provide one copy to each regional office, and two copies to each division office. Direct distribution has been made to the division offices, which are asked to forward one copy to the State. If additional copies of the proceedings are desired, or if you have any questions regarding DP 87, the western States report, or pavement drainage, please contact Project Manager Bob Baumgardner at 202-366-4612.

Ray G. Griffith for

William A. Weseman

Attachment

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Subject: ACTION: Demonstration Project No. 87 "Drainable Pavement Systems"

Date: APR 6 1992

From: Director, Office of Engineering
Director, Office of Technology Applications

Reply to Attn of: HNG-42

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator
ATTN: Technology Transfer Coordinators
Regional Pavement Engineers

We are pleased to announce that the subject demonstration project is available to State highway agencies (SHA's).

The pavement structural section is the single most costly element of a highway system. Water in the pavement section has been determined to be a factor in premature pavement deterioration. Inadequate base drainage has been identified as a nationwide problem, particularly in concrete pavements. A number of SHA's have developed innovative pavement designs and construction practices that have been successful in draining the pavement section. Application of these innovative techniques can reduce premature pavement failures and extend the useful life and investment in the Nation's roadways.

To demonstrate these newer pavement drainage techniques and other concepts, the Federal Highway Administration's (FHWA) Office of Technology Applications and Office of Engineering have developed Demonstration Project No. 87, "Drainable Pavement Systems." The project centers around classroom discussions that provide current state-of-the-art guidance for designing, constructing, and maintaining permeable base drainage systems. Detailed guidance will be provided for the design and construction of both unstabilized and stabilized permeable bases. The staff will also demonstrate the permeability of different base course materials.

Forwarded under separate cover are additional copies of the attached project flyer. These flyers are for distribution to the State agencies in your region. Interested agencies should submit requests for the demonstration project through the local FHWA office.

Please call Project Manager Robert Baumgardner at (202) 366-4612, should you have any questions.

Douglas A. Bernard

T. A. Stam for Thomas O. Willett

Attachments

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: **INFORMATION**: "Effectiveness of Highway Edgedrains," Experimental Project No. 12, Concrete Pavement Drainage Rehabilitation

Date: APR 14 1993

From: Chief, Pavement Division
Chief, Engineering Applications Division

Reply to Attn of: HNG-40
HTA-20

To: Federal Regional Highway Administrators
Division Administrators
Federal Lands Highway Program Administrator

Transmitted under separate cover are sufficient copies of the subject report for use by you and your States. This study measured concurrent rainfall and edgedrain discharges, piezometric water levels and soil moisture under the pavement and shoulders in 10 States (Alabama, Arkansas, California, Illinois, Minnesota, New York, North Carolina, Oregon, West Virginia, and Wyoming). This report should be of interest to State pavement design and research engineers in your region. We would like to take this opportunity to thank you and the participating State and division staffs for making this project a success.

We believe that a principal contribution that this report makes is that it provides an excellent guide to any State interested in developing a pavement drainage study. The pavement instrumentation necessary for drainage is well documented.

Your attention is particularly directed to the CONCLUSION, Effectiveness of Edgedrains, section on page 78 of the subject report. We feel that the following three statements have considerable impact on the national pavement subsurface drainage effort to reduce damage to the pavement structure caused by surface infiltration through joints and cracks:

- "Retrofitting longitudinal edgedrains to an existing highway provides a sink to collect water draining laterally off pavement surfaces, as well as water reaching the edgedrain through subgrade voids and channels."

- "Tight, low permeability subgrade material precludes ready, lateral drainage with or without edgedrains."

- "If highway restoration, as well as construction, includes provisions for a permeable subgrade (base), as well as edgedrains, the two together should prove the most efficient in restoring the highway."

We would like to direct your attention to Column (8) of Table 3 on page 64. The wide range of the percent of rainfall that shows up in the edgedrain discharges indicates how difficult it is to design edgedrain systems. Therefore, this study fully supports the "Time-to-Drain" concepts presented in Demonstration Project No. 87, "Drainable Pavement Systems" (Demo 87).

We would like to take this opportunity to update you on our pavement drainage efforts. Currently, we are making presentations of Demo 87. Attached is a map showing the progress of the project. It should be noted that this project only covers drainage of new or reconstructed portland cement concrete (PCC) pavements with permeable bases, a separator layer and edgedrains. Drainage of asphalt concrete (AC) pavements or retrofit longitudinal edgedrains is not covered in the demonstration project.

The next generation of our pavement drainage activities will include the development of the National Highway Institute Course No. 13126, "Pavement Subsurface Drainage Design." Drainage of pavement infiltration for both PCC and AC pavements, along with retrofit longitudinal edgedrains, will be covered. This project is in the conceptual stage with a National Highway Institute proposal under development.

A limited number of additional copies of the attached report are available from our Report Center, or by purchase from the Geological Survey (Report No. WRRI 92-4147, cost - $13.00, and telephone number (303) 236-7476):

U.S. Geological Survey
Books and Open-File Reports Section
Box 25286, Federal Center
Denver, Colorado 80225

If you have any additional questions, please contact Mr. Robert Baumgardner (202) 366-4612 in the Pavement Division.

Theodore R. Ferragut

Louis M. Papet

Attachments

U.S. Department
of Transportation

Federal Highway
Administration

Memorandum

Subject **ACTION**: Maintenance of Pavement Edgedrain Systems

Date MAR 21 1995

From Associate Administrator for Program Development

Reply to Attn of HNG-42

To Regional Administrators
Federal Lands Highway Program Administrator
ATTENTION: Regional Pavement Engineers

The purpose of this memorandum is to strongly reiterate the need for maintenance of edgedrain systems. We have become increasingly concerned about the lack of maintenance of the edgedrain systems that we have observed around the country. Recently, one of our division offices made an extensive review of the maintenance of pavement edgedrain systems and prepared an excellent report documenting their findings. Attached is a copy of their report "Maintenance of Pavement Underdrain System." The reference to the identity of the division office and the State highway agency has been removed at their request. We recommend that the division offices in your region conduct similar field evaluations of existing edgedrain systems.

Sufficient copies of the publication are attached to provide one copy to each regional office, and two copies to each division office. We ask that this report be forwarded to the State. If additional copies of the report are needed, please contact Mr. Robert Baumgardner at (202) 366-4612.

We cannot over emphasize the importance of proper construction and maintenance of pavement edgedrain systems. If water is not rapidly removed from these systems, they will serve as reservoirs saturating pavement bases and causing rather than preventing accelerated pavement deterioration.

Currently, we are finalizing a service contract for the video inspection of highway edgedrains. This service will assist you and the State in evaluating pavement drainage systems. The video inspection will provide a qualitative picture of edgedrain conditions in the State.

For Thomas J. Ptak

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject **INFORMATION**: Pavement Subsurface Drainage Activities

Date DEC 16 1994

From Chief, Pavement Divison

Reply to Attn of HNG-42

To Regional Administrators
Federal Lands Highway Program Administrator

The purpose of this memorandum is to update you on our pavement drainage activities and transmit a copy of the Demonstration Project No. 87, (Demo 87) "Drainable Pavement Systems Instructor's Guide". This publication provides a capsulized picture of pavement subsurface drainage design. Demo 87 was presented in over 40 States, Puerto Rico and the District of Columbia. Attached is a map showing participation.

With the successful completion of the first phase of Demo 87, we are moving into Phase II of Demo 87, which consists of three activities:

First, a Technical Working Group (TWG) on Flexible Pavement Drainage Design consisting of participants from FHWA, State highway agencies (SHA's), universities, and industries was convened in June of this year. The participants provided input as a TWG by drawing on their experience and expertise. Wide ranging discussions on the design and construction of flexible pavements revealed that there was no clear definition of the role of drainage in flexible pavements. One point of consensus was that, if a permeable base was provided in a flexible pavement, it would primarily combat pavement infiltration water; it would not solve ground water problems. A summary of the TWG workshop's notes was transmitted to each regional office by memorandum dated November 21, 1994.

Second, we have developed a Proposal (RFP) entitled "Video Inspection of Highway Edgedrains," which is now being considered for contract award. This will provide SHA's with a qualitative video picture of edgedrain conditions. Upon request of the SHA, the video contractor will be available to the SHA for up to a week to investigate the edgedrain in-situ conditions. Both existing edgedrains and new construction could be viewed on both AC and PCC pavements. After the inspection, the Contractor will provide the SHA with a copy of video tapes and 35 mm slides taken during the inspection. Also available will be Graphic Information System (GIS) output documenting both the vertical and horizontal alinement of the edgedrain system. We expect this activity to be available about March 1, 1995.

Third, we are interested in continuing to develop expertise and provide technical support in the construction of permeable base and drainage systems for both flexible and concrete pavements. We would appreciate feedback from your office to identify upcoming construction projects, so that we can assess developing construction techniques and practices and provide technical support as appropriate. We encourage studies to evaluate the effect of drainable systems on pavement performance (particularly AC pavements) which includes a non-drained control section. Please keep us informed of any studies underway or planned.

Attached is a brief one-page description of our current drainage activities that you may want to disseminate to your division offices and SHA's.

for T. Paul Teng

2 Attachments

SUMMARY OF FHWA'S CURRENT PAVEMENT SUBSURFACE DRAINAGE ACTIVITIES

December 1994

Demonstration Project No. 87, "Drainable Pavement Systems" (Demo 87) provided detailed design and construction guidance for drainage systems under Portland Cement Concrete (PCC) pavements. Established drainage design procedures were combined with the state-of-the-art in practical permeable base construction to provide a well balanced approach for the drainage of PCC pavements. Detail design and construction guidance was provided for permeable bases, separator layers and edgedrains. Demo 87 was presented in over 40 States, Puerto Rico and the District of Columbia. With the successful completion of the first phase of Demo 87, we are moving into Phase II of Demo 87 which consists of athree activities.

First, a Technical Working Group on Flexible Pavement Drainage Design (TWG) consisting of participants from FHWA, State highway agencies (SHA's), Universities, and Industry was convened in June of this year. The participants provided input as a TWG by drawing on their experience and expertise. Wide ranging discussions on the design and construction of flexible pavements revealed that there was no clear definition of the role of drainage in flexible pavements. The only point of consensus was that, if a permeable base was provided in a flexible pavement, it would primarily combat pavement infiltration water; it would not solve ground water problems. A summary of the TWG workshop is available.

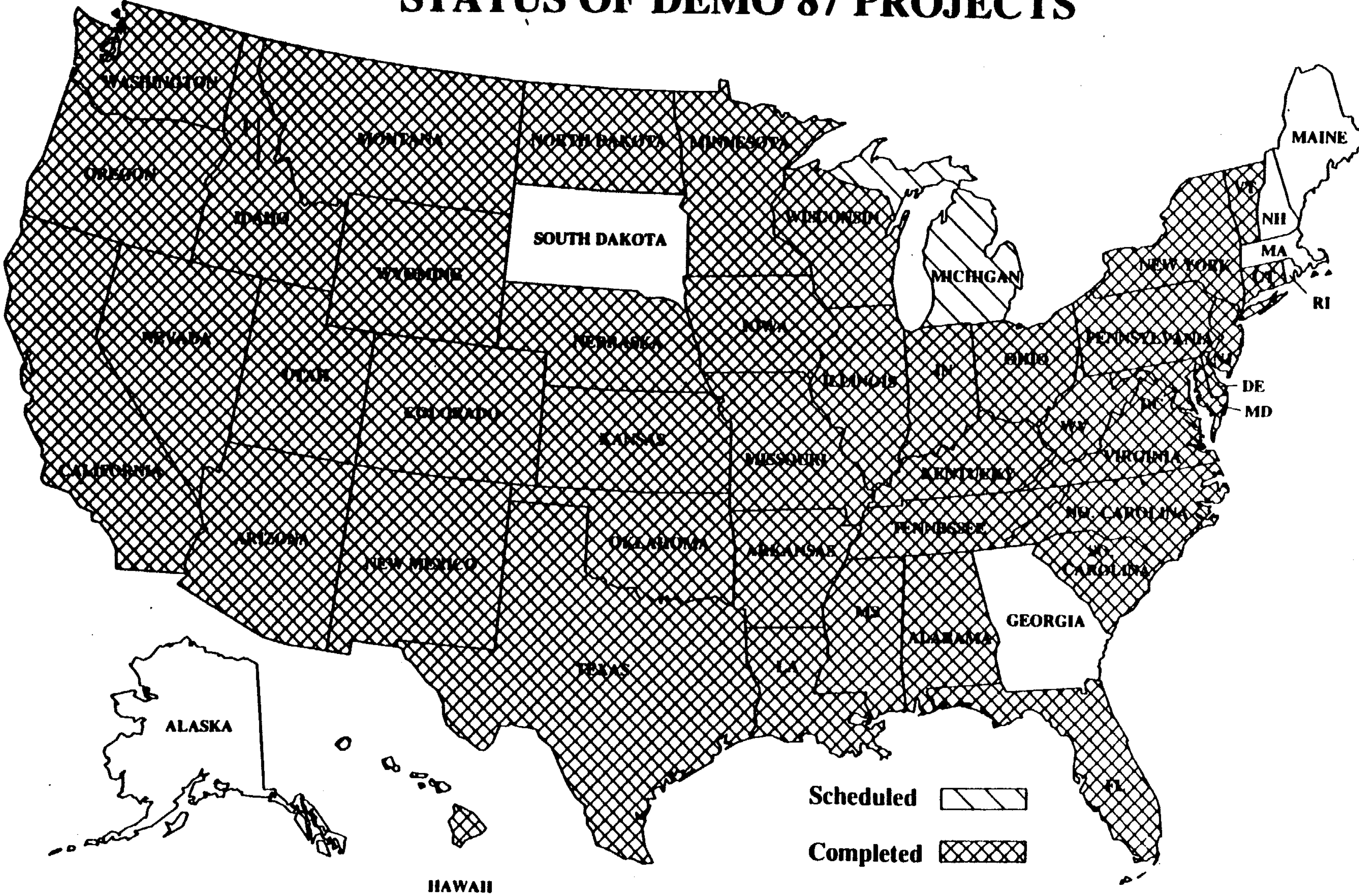
Second, we are preparing to award a contract in response to a Request for Proposal (RFP) entitled "Video Inspection of Highway Edgedrains" contract. This will provide State highway agencies (SHA's) with a qualitative video picture of edgedrain conditions. Upon request of the SHA, the Contractor will be available to the SHA's for up the a week to investigate the edgedrain in situ conditions. Both existing edgedrains and new construction for AC and PCC pavements could be viewed. The equipment cannot inspect "fin" drains or round pipe less than 100 mm diameter. After the inspection, the Contractor will provide the SHA with a copy of video tapes and 35 mm slides taken during the inspection. Also, Graphic Information Systems (GIS) information on edgedrain vertical and horizontal alinement will be provided. We expect this activity to be available by March 1, 1995.

Third, we are interested in continuing to develop expertise and provide technical support in the construction of permeable base and drainage systems for both flexible and concrete pavements. To accomplish this activity, field trips will be made to view construction and provide technical support for placing permeable bases in both rigid and flexible pavements. We are also interested in studies evaluating the effect of these systems on pavement performance.

We are now finalizing a RFP entitled "**Pavement Subsurface Drainage Microcomputer Program.**" This microcomputer program will replicate the design procedures contained in the Demo 87 Participant Notebook. This will provide engineers with a useful tool for drainage design.

The National Highway Institute (NHI) has advertised a RFP for developing a training course entitled **NHI Course No. 13126 "Pavement Subsurface Drainage Design."** Drainage guidance for PCC and flexible pavements, along with retrofit edgedrains, will be compiled into a comprehensive pavement drainage training course. The length of the course will be about 3 days and will follow a slide-lecture format. This training course will be available to all SHA's and Industry though NHI.

STATUS OF DEMO 87 PROJECTS



CHAPTER 6

SHOULDER

6.1 **TA 5040.29, Paved Shoulders, February 2, 1990.**

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

SUBJECT	FHWA TECHNICAL ADVISORY
PAVED SHOULDERS	T 5040.29 February 2, 1990

1. PURPOSE. To outline recommended practices for the design of paved shoulders.

2. CANCELLATION. Technical Advisory T 5040.18, Paved Shoulders, dated July 29, 1982.

3. DEFINITION. Shoulder - the portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles for emergency use, and for lateral support of the base and surface courses.

4. BACKGROUND

   a. Over the years, the function of shoulders adjacent to mainline pavements has broadened considerably. Some of the added functions of modern roadway shoulders are: to accommodate an increasing encroachment of traffic; to expedite water runoff from travel lane pavement; to provide added space for construction and maintenance activities; to provide other usage such as bicycle paths or slow moving vehicles and equipment lanes; to reduce edge stresses and edge and corner deflections; and to reduce the development of pavement edge drop-offs.

   b. Paved shoulders are required on all Interstate routes. The decision to pave a shoulder on other routes is an engineering determination based on traffic volume, past experience and availability of funds. Paved shoulders are justified by improved and smoother traffic operations, expectation of better pavement performance, increased pavement life, enhanced highway safety and reduced maintenance. Studies included in TRB Special Report 214, "Designing Safer Roads - Practices for Resurfacing, Restoration, and Rehabilitation" and Publication No. FHWA/RD-87/094, "Safety Cost-Effectiveness of Incremental Changes in Cross-Section Design -- Informational Guide" have cited reduced accident rates with the use of paved shoulders.

5. SHOULDER TYPE SELECTION

a. It is recommended that the shoulder be constructed of the same materials as the mainline pavement in order to facilitate construction, improve pavement performance and reduce maintenance costs.

b. The use of full-width paved shoulders is desirable. However, the additional cost of this design may not be warranted on all projects. In those cases, the use of widened lanes should be given strong consideration. Widened lanes reduce edge stresses and the potential for edge drop-offs, increase safety, and reduce maintenance costs. A monolithic widening of 2 to 3 feet outside of the traveled way is recommended. Widened lanes are only effective when striped as 12-foot travel lanes. Consideration should be given to the placement of rumble strips on the shoulder portion of the widened lane.

6. DESIGN

a. General

(1) Background. The structural design of shoulders has received less attention than the mainline pavement. Shoulder design has developed gradually through experience rather than from a rational pavement design approach. There are currently no nationally recognized procedures for designing shoulders. When designing a shoulder, the following should be considered:

(a) Consider whether the shoulder will be used as a temporary or permanent traffic lane in the future.

(b) Integrate shoulder drainage with the overall pavement subdrainage design.

(c) Avoid the use of aggregate base courses having more than 6 percent minus 200 mesh sieve materials to prevent frost heaving, pumping, clogging of the shoulder drainage system, and base instability.

(d) Have a definite program of shoulder maintenance.

(e) Take advantage of the desirable performance of concrete shoulders adjacent to concrete mainline pavements.

(2) Width and Cross Slope. Documents containing applicable geometric criteria are listed in Federal-Aid Highway Program Manual 6-2-1-1, "Design Standards for Highways." The shoulder cross slope should be at least 1 percent more than the mainline pavement cross slope on tangent sections to facilitate drainage but should not be so steep as to preclude the use of the shoulder as a temporary travel lane during future construction. Asphalt and concrete shoulders should be sloped from 2 to 6 percent, gravel or crushed rock from 4 to 6 percent and turf about 8 percent. However, care must be exercised so that the algebraic difference in cross slope at the pavement edge does not exceed 0.08 in order to avoid a hazardous roll-over effect.

(3) Pavement Markings and Shoulder Texture Treatments

(a) Distinguishing paved shoulders from the mainline pavement is recommended to discourage the use of the shoulder as a travel lane and provide guidance and warning to drivers. This can be accomplished by pavement markings and differences in shoulder surface texture.

(b) Shoulder texture treatments that provide an audible/vibrational warning to errant drivers have proven effective in keeping traffic off the shoulder and reducing accidents on long tangent or monotonous highway sections with a history of run-off-the-road accidents. Attachment 1 (Figures 1 through 3) describes treatments used by States on bituminous and concrete shoulders.

(4) Drainage. The presence of free water beneath the pavement and/or shoulder is detrimental to performance. Grooved shoulder texture treatments, such as discussed in paragraph 6a(3)(b), can affect surface drainage. It is recommended that these treatments be offset from the longitudinal

lane/shoulder joint to facilitate joint sealing and minimize surface water infiltration. An analysis to provide adequate surface and subsurface drainage should be conducted on each project.

b. <u>Concrete Shoulders</u>

(1) <u>General</u>

(a) Concrete shoulders should be tied to the mainline with properly spaced and sized tiebars. Tied concrete shoulders will reduce pavement stresses and edge deflections. Tied concrete shoulders will also result in a tighter, easier to seal longitudinal joint that, when properly maintained, will effectively reduce water infiltration into the pavement structure.

(b) Retrofitting tied concrete shoulders or lane widening will reduce edge stresses and deflections. The age, condition and remaining service life of the existing pavement play a significant role in determining whether a retrofit is practical. It is recommended that a retrofit be added only when an engineering and economic analysis indicates it to be a cost-effective solution.

(2) <u>Thickness</u>

(a) Shoulders should be structurally capable of withstanding wheel loadings from encroaching truck traffic. On urban freeways or expressways, the shoulders should be constructed to the same structural section as the mainline pavement to ensure adequate load capacity at the interface between the mainline and shoulder; to provide for ease and economy of construction; and to prevent a "bathtub" condition under the pavement. This will also allow the shoulder to be used as a temporary detour lane during rehabilitation or reconstruction.

(b) As an option for other than urban freeways and expressways, a tapered shoulder may be considered. Adjacent to the mainline, the shoulder should be the same thickness as the mainline to permit mid-depth tiebar placement and to provide structural support for truck wheel encroachments. The shoulder may then be tapered to no less than 6 inches at the outside edge. Care must be exercised with a tapered section since a "bathtub" type condition can result, ponding water in the area of the lane/shoulder interface.

(3) Subbase. It is recommended that the same type of subbase be used under the shoulder as under the mainline, especially on high volume facilities. Care must be taken in designing the subbase cross slope under concrete shoulders to avoid pocketing of water under the lane/shoulder joint and at the shoulder edge. Problems are often encountered at this location due to changes in subbase type, resulting in non-uniform support or difference in drainage characteristics.

(4) Transverse Joint

(a) Mainline pavement joints should be extended across the shoulder. All transverse shoulder joints should be sawed to a depth of 1/3 the slab thickness.

(b) Where plain jointed concrete shoulders are used adjacent to continuously reinforced mainline pavement, the shoulder joints should be sawed at 15-foot intervals. Plain concrete shoulders should not be constructed integrally with continuously reinforced concrete pavement. Transverse saw cuts in the integrally constructed shoulders will propagate cracks across the mainline.

(c) Where plain jointed concrete shoulders are used adjacent to jointed reinforced mainline pavement with skewed joints, intermediate joints should not be sawed in the shoulder. Skewed intermediate shoulder joints tend to propagate 2 parallel transverse cracks across the mainline pavement.

(d) Where plain jointed concrete shoulders are used adjacent to jointed reinforced mainline pavement with perpendicular joints, intermediate shoulder joints are optional. However, intermediate joints should not be sawed if the shoulder is constructed integrally with the mainline pavement. Intermediate transverse saw cuts in integrally constructed shoulders will propagate cracks across the mainline.

(5) Longitudinal Joint. Combined lane and shoulder or lane widening widths of 15 feet for the right (outside) lane and 16 feet for the left (inside) lane have generally performed satisfactorily. For widths greater than these, a longitudinal joint should be sawed and sealed.

(6) Keyway. Keyways are not recommended for use, and should never be used for pavements less than 10 inches thick. If used for pavements 10 inches or greater in thickness, keyways should be placed at mid-slab depth to ensure maximum strength. Proper concrete consolidation, both above and below the keyway, is essential.

(7) Tiebars

(a) Tiebars are needed between the mainline pavement and concrete shoulders to keep the longitudinal joint tight so as to provide the necessary load transfer. Tiebars are typically placed on 30-inch centers at mid-slab depth. If tiebars are to be bent and later straightened during construction, Grade 40 steel is recommended, as it better tolerates the bending. When using Grade 40 steel, 5/8-inch by 30-inch tiebars should be used. When using Grade 60 steel, 5/8-inch by 40-inch or 1/2-inch by 32-inch tiebars should be used. These lengths are necessary to develop the allowable working strength of the tiebar.

(b) Tiebars should not be placed within 15 inches of transverse joints. When using tiebars longer than 32 inches with skewed joints, tiebars should not be placed within 18 inches of the transverse joints.

(c) The structural adequacy of tiebars can be reduced through corrosion. Corrosion resistant tiebars are recommended.

(8) Reinforcement. The majority of concrete shoulders are plain with short joint spacing. They have performed well when placed adjacent to either plain jointed, reinforced jointed, or continuously

reinforced concrete mainline pavements. The plain jointed design is therefore recommended. In cases where jointed reinforced or continuously reinforced shoulders are placed integrally with the same type of mainline pavement, the steel in the shoulder is normally placed at the same percentage as required for the mainline pavement.

(9) Lane Widening. A 2- to 3-foot integral widening of the mainline slab will reduce edge strains and deflections. To be effective, the travel lane should be striped at 12 feet with the edge of the slab being moved into the shoulder and away from traffic load applications. The remaining portion of the shoulder may also be paved.

c. Flexible Shoulders

(1) Types

(a) Bituminous surface treated shoulders consist of an aggregate shoulder on which coats of liquid bituminous material and aggregate chips have been applied and rolled. Regional terminology such as armor coat, double or triple surface treatment, or seal coat all apply.

(b) Bituminous aggregate shoulders consist of a bituminous mat on top of an aggregate base course of variable depth that may or may not contain a stabilizing agent.

(c) Full-depth asphalt shoulders consist of asphalt mixtures for all courses laid directly on the prepared subgrade.

(d) Widened lanes consist of a 2- to 3-foot widening of the mainline structural section with the remaining width of shoulder composed of a bituminous surface treatment, bituminous aggregate section, aggregate or turf. For the widening to be effective, the widened lane should be striped as a 12-foot travel lane.

(2) Thickness

(a) Shoulders should be structurally capable of withstanding wheel loadings from encroaching truck traffic. On urban freeways or expressways, the shoulders should be constructed to the same structural section as the mainline pavement to ensure adequate load capacity at the interface between the mainline and shoulder; to provide for ease and economy of construction; and to prevent a "bathtub" condition under the pavement. This will also allow the shoulder to be used as a temporary detour lane during rehabilitation or reconstruction.

(b) For other than urban freeways and expressways, a structural section less than that of the mainline may be warranted. It is recommended that the thickness be based on an evaluation of life cycle costs and past performance under similar conditions. The use of widened lanes should be considered in the life-cycle cost analysis.

7. COMMON DISTRESSES

a. Most shoulder deterioration is attributable to one or more of the following causes: truck encroachments, water intrusion at the longitudinal joint, use of lower quality materials, and inadequate thickness.

b. Field observations have shown that shoulder distress is primarily concentrated within 24 inches of the longitudinal mainline/shoulder joint.

c. The longitudinal joint between a concrete mainline pavement and a flexible shoulder is a primary source for infiltration of surface water into the subbase. The longitudinal joint has proven to be one of the weakest parts of the mainline/shoulder system. Distress in the form of excessive cracking, breakage, and settlement is concentrated at this location. Separations generally range from 1/8 to as much as 2 inches. The infiltration of water can be minimized by a properly sealed and maintained joint. Some highway agencies add a "wedge" (6 to 19 inches wide) of hot-mixed bituminous material in the low areas where the flexible shoulder has settled. Tied concrete shoulders in lieu of flexible shoulders will minimize the problem associated with a longitudinal joint.

8. SUMMARY. Shoulders are an important element in the performance and overall service provided by highway pavements. Proper attention to the selection, design, construction and maintenance of shoulders is an important part of pavement management and can result in improved performance and service cost-effectiveness.

9. REFERENCES. Attachment 2 lists reference documents that should prove useful in the area of design, construction and rehabilitation of paved shoulders.

10. FIGURES. Attachment 1 contains 3 figures which describe shoulder surface treatments used by States on bituminous and concrete shoulders.

Thomas O. Willett
Acting Associate Administrator for
Engineering and Program Development

Attachments

Typical Shoulder Treatments

Bituminous Shoulders

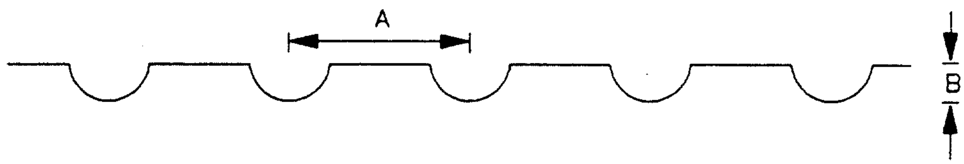


Figure 1. Indented Strip - A continuous stretch of indentations is impressed in the shoulder through the use of a steel roller. In a method developed by California, steel bars are welded to a vibratory roller to impress indentations. Georgia specifies that a nominal 1 and 1/2 inch diameter pipe be welded to the roller drum to form the indentations. Typically the indentations are spaced 8 inches apart (A) and 3/4 inch to 1 inch deep (B). Most States offset this treatment 6 to 12 inches from the edge of the mainline pavement and the typical treatment width is 3 feet.

Stone Aggregate Gradations (Percent Passing by Weight)

State	Appl	1	3/4"	1/2"	3/8"	#4	#8	#16	#100	#200	#/SY
NC	1st	90-100	20-55	0-10	0-5	---	---	---	---	0-1.5	45-50
	2nd	100	90-100	20-55	0-15	0-5	---	---	---	0-1.5	35-40
	3rd	---	100	98-100	75-100	20-95	0-15	---	---	0.1.5	17-22
SC	1st	100	90-100	---	0-20	0-5	---	---	---	---	28-32
	2nd	---	100	95-100	80-100	20-50	---	0-6	0-2	---	18-22

Figure 2. Bituminous Surface Treatment - The effectiveness of a textured shoulder is largely dependent upon the gradation of aggregate used. Treatments containing 3/4 inch to 1 inch stone have been observed to be very effective as an alerting texture.

Concrete Shoulders

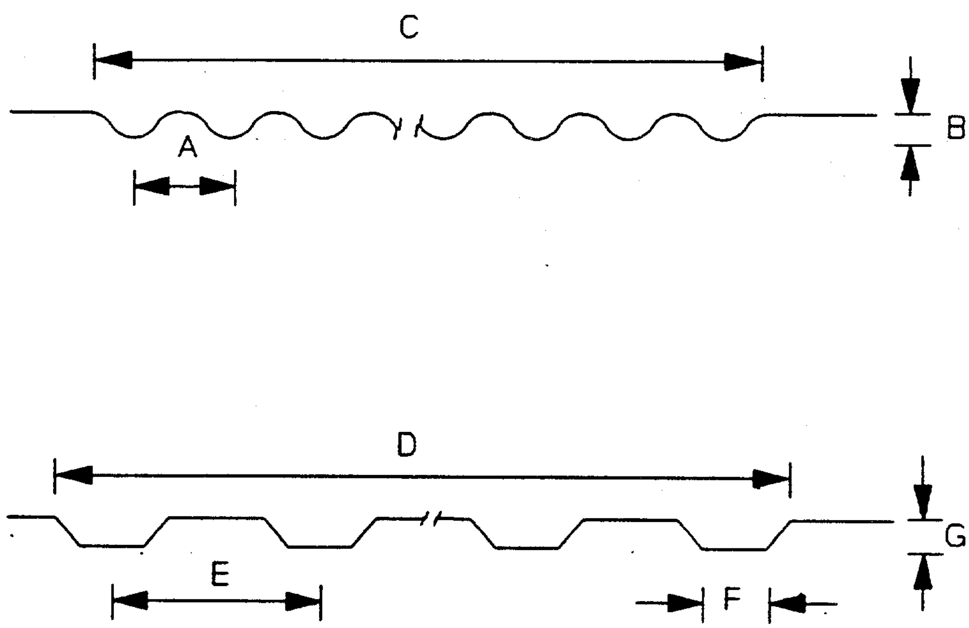


Figure 3. **Corrugations or rumble strips** - The top configuration is the one most commonly used. Most States build this type of section 3 to 6 feet wide (C); 3 to 4 and 1/2 inches between corrugations (A); and 1 inch deep (B). Corrugations generally extend the full width of the shoulder and are placed at 40 to 100 foot intervals. One variation of this treatment is to offset the section 1/2 to 4 feet from the edge of mainline pavement. The corrugations may also be placed in the middle 1/3 of the shoulder, if desired, to accommodate traffic during future rehabilitation or maintenance work or to accommodate bicyclists.

The bottom configuration is generally built 4 feet wide (D); 6 to 12 inches between corrugations (E); and 3/4 to 1 inch deep (G). Corrugations are generally offset 1/2 to 2 feet from the edge of the mainline pavement and are placed at 40 to 90 foot intervals.

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CHAPTER 7

PAVEMENT REHABILITATION

7.1 Concrete Pavement Restoration Performance Review, May 22, 1997.
- Concrete Pavement Restoration Performance Review, April 1987.

7.2 Crack and Seat Performance Review Report, April 1987.

7.3 Saw and Seal Pavement Rehabilitation Technique, February 22, 1988.
- Saw and Seal Pavement Rehabilitation Technique, Technical Paper 88-01.

7.4 Reserved

7.5 FHWA Notice N5080.93, Hot and Cold Recycling of Asphalt Pavements, October 6, 1981.

7.6 Reserved.

7.7 Use of Recycled Concrete in Portland Cement Concrete Pavement, July 25, 1989.

7.8 Use of Recycled PCC as Aggregates in PCC Pavements, February 1985.

7.9 Overview of Surface Rehabilitation Techniques for Asphalt Pavements, Report Number FHWA-PD-92-008, April 6, 1992.

7.10 State of the Practice Design, Construction, and Performance of Micro-Surfacing, Report Number FHWA-SA-94-051, July 12, 1994.

7.11 Retrofit Load Transfer, Special Project 204, February 10, 1994.

7.12 Reserved.

7.13 Thin Bonded Overlay and Surface Lamination Pavements and Bridges, ISTEA 6005, July 1, 1994.

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Washington, D.C. 20590

Subject: Concrete Pavement Restoration and Crack and Seat Rehabilitation Performance Evaluation Reports

Date: MAY 22 1987

From: Associate Administrator for Engineering and Program Development

Reply to Attn of: HHO-13

To: Regional Federal Highway Administrators Regions 1-10

In a previous memorandum dated March 25, 1986, we noted the growing concerns that certain concrete pavement rehabilitation strategies and individual techniques were performing below expected levels or were not appropriate for actual project conditions. A plan was developed by Demonstration Projects Division and the Pavement Branch to conduct detailed reviews of completed Concrete Pavement Restoration (CPR) and Crack and Seat (C&S) rehabilitation projects during the remainder of the year. These reviews have been completed and the subject reports are being disseminated to provide interim technical guidance.

The University of Illinois under a FHWA research contract entitled "Determination of Rehabilitation Methods for Rigid Pavements" is undertaking a more extensive data collection and analysis effort which will provide further information on these strategies. Our CPR and C&S reviews have been coordinated with this research contract to minimize duplication of effort. Much of the initial project description data they had collected was utilized to select projects for our reviews. Conversely, detailed plan data that was gathered during our reviews is being shared with the university. The research report is expected to be available by the end of the year.

The report on CPR included an in-depth review of 26 projects in eight States. The review found that proper preliminary engineering and timing of the individual techniques are critical to project performance. When properly designed and constructed it was found that CPR will generally reduce pavement deterioration thereby, prolonging pavement life. However, continued maintenance throughout the project design life will be required. In addition, it was noted that pavements having an accelerated rate of slab cracking will continue to have a high rate of slab deterioration immediately after completion of the CPR project. The techniques which were reviewed included full depth patching, partial depth patching, diamond grinding, joint resealing, and slab stabilization (subsealing). In brief, it was found that: dowelled jointed full depth concrete patches provided satisfactory long-term performance (up to 8 years observed); partial depth patches limited to the top third of the slab and containing a compressible material in all working joints and cracks are exhibiting good performance after 6 years; diamond grinding can provide long-term improvement in ride quality, however, further evaluation on whether there is a long-term improvement in pavement friction is required; transverse joint resealing using silicone provided good performance, whereas, hot-poured sealants experienced significant adhesion failures generally within 2 years; hot-poured sealants, on the other hand, appear to be the most effective material to use in the longitudinal asphalt shoulder joint; the benefits of subsealing were not readily observed, although it did not appear to adversely affect pavement performance.

As in any other rehabilitation strategy, adequacy of design, quality and timeliness of construction, and continued maintenance determine the effectiveness of the CPR strategy. Close adherence to the technical guidance contained in the "Pavement Rehabilitation Manual", the "Techniques for Pavement Rehabilitation" course notebook, and the "1985 AASHTO-AGC-ARTBA Joint Committee's Guide Procedures for Concrete Pavement 4R Operations", generally resulted in good performance of the individual techniques within a CPR project. In cases where the pavement is suitable for rehabilitation and proper procedures are used, up to 10 years of service life can be achieved using CPR techniques.

The report on C&S included an in-depth review of 22 projects in eight States. The projects reviewed were the classic C&S type (ie., small hairline cracks, no rupturing of the reinforcement, etc.) and did not include "rubblizing" or pulverizing the pavement. Both positive and negative aspects of C&S were discovered during the review. The most positive aspect is the delay of reflective cracking. A majority of the projects reviewed showed a reduction in reflective cracking during the first few years after construction. However, most of the C&S sections exhibited the same amount of reflective cracking as the control sections after approximately 4 to 5 years. Two projects that have shown significant reduction in reflective cracking had the following similiarities: non-reinforced pavement; small changes in seasonal temperatures; and a strong base (cement treated). The primary negative aspect of C&S is the reduction in structural capacity of the pavement. To compensate for this, more overlay thickness is required, thus increasing the cost. When thick overlays (5 to 8 inches) are proposed by State highway agencies, very little structural value is given to the cracked pavement. These findings suggest that this rehabilitation strategy should be approached with caution. The costs for additional overlay thickness, the cracking and seating, and other required work such as shoulder and guardrail raising, must be evaluated when determining the most cost effective rehabilitation strategy to employ.

In developing a rehabilitation project, the process for preliminary engineering and economic analysis outlined in Administrator Barnhart's November 15, 1983 memorandum should be followed. In addition, States need to continually monitor and evaluate their previous experience with various rehabilitation strategies to determine the expected service life of these strategies in their State.

A sufficient number of copies of each report are enclosed for distribution to the Division Offices and State highway agencies in your Regions. Additional copies or technical assistance can be obtained by contacting Mr. John P. Hallin at FTS 366-1323.

Rex C. Leathers

Rex C. Leathers

CONCRETE PAVEMENT RESTORATION

PERFORMANCE REVIEW

Federal Highway Administration
Pavement Division
and
Demonstration Projects Division

April 1987

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I. INTRODUCTION

Federal Highway Administrator R.A. Barnhart's November 15, 1983, memorandum on pavement rehabilitation design identified the lack of good performance data as the weakest point in the rehabilitation process. Reliable performance data is a key element in evaluating alternate rehabilitation strategies and making network and project level engineering analyses.

Concerns had been expressed that the performance of certain rehabilitation strategies and individual techniques were below expected levels, or in some cases, strategies selected for specific project conditions may not have been the appropriate solution. One of the pavement rehabilitation strategies for portland cement concrete (PCC) pavements that has caused such concerns is concrete pavement restoration (CPR). Individual techniques within a CPR project include slab stabilization (subsealing), full depth patching, partial depth patching, load transfer restoration, subdrainage, shoulder restoration, diamond grinding, and joint resealing.

In order to assess the effectiveness of the CPR strategies being undertaken by State highway agencies, the Federal Highway Administration (FHWA) conducted a review of selected CPR projects. The review focused on three aspects of CPR completed on jointed plain and jointed reinforced concrete pavements:

1. Expected service life based on observed performance.
2. Variables that significantly affect the performance of individual CPR techniques.
3. Conditions under which each strategy has been used in a cost-effective manner.

Field reviews were conducted jointly by the FHWA's Pavement Division and Demonstration Projects Division between May and October 1986. Teams composed of an engineer from each division conducted in-depth reviews of 26 CPR projects in eight States. These teams were assisted by engineers from the appropriate FHWA regional and division offices and State engineers familiar with the design, construction, and maintenance of each project. The States also provided historical and inventory data for each project.

This review was closely coordinated with an ongoing research contract entitled "Determination of Rehabilitation Methods for Rigid Pavements." The research project will gather data on a large number of the variables that affect the performance of individual techniques as well as data on the success or failure of the overall strategy from approximately 150 projects in more than 20 States. Standard statistical analysis procedures will then be applied to develop conclusions. The Strategic Highway Research Program (SHRP) is expected to provide additional information regarding pavement rehabilitation strategies.

This CPR review provides interim technical guidance until the research project is completed. The findings represent the consensus of the FHWA engineers conducting the reviews based on their experience, data gathered, field observations, and discussions with field practitioners.

II. SUMMARY

Twenty-six completed CPR projects in eight States were reviewed. The review found that proper preliminary engineering and timing of the individual techniques are critical to project performance. When properly designed and constructed it was found that CPR will generally reduce pavement deterioration thereby, prolonging pavement life.

The age of the completed CPR projects ranged between 1 and 14 years. The findings presented in this report are based on project data, discussions with State personnel, field inspections of each project, and engineering judgement and experience of the FHWA team conducting the review. It is hoped that these results can be used to assist highway engineers in determining whether CPR techniques would be an effective rehabilitation strategy for particular highway projects. The report also provides information on the practices used by State highway agencies where good performance of CPR techniques was observed.

The number of individual CPR techniques undertaken on any specific project varied for project to project and State to State. Like any other pavement rehabilitation strategy the overall effectiveness of CPR techniques is highly dependent on adequacy of design, quality of construction, and maintenance practices.

The individual CPR techniques covered by this report include subsealing, full depth patching, partial depth patching, grinding, and joint resealing. Very few of the projects reviewed included pressure relief joints, subdrains retrofit load transfer devices, and shoulder restoration techniques.

Therefore, detailed comments on these techniques are not provided. It was also concluded that proper evaluation of subdrainage is not possible without performing in-depth testing. A separate project has been initiated to evaluate subsurface drainage on a variety of in-service installations.

Based on our field observations and discussions with State engineers, an effective CPR strategy will generally reduce the rate of pavement deterioration and properly designed and constructed CPR techniques can be expected to provide 6 to 10 years of service life. However, continued maintenance throughout the project design life will be required. On most projects, a followup maintenance effort was needed within 1 year of project completion.

The available preliminary engineering data developed for each CPR project was reviewed. On many projects, very little detailed information concerning the cause and extent of distress had been assembled. Some projects experienced large overruns in quantities and at least one project was terminated due to cost overruns before all CPR work could be completed. The lack of timely detailed condition data likely contributed to the major overruns.

Based on review of these 26 projects, we believe that pavements having an accelerated rate of slab cracking prior to rehabilitation will continue to have a high rate of slab deterioration immediately after completion of a CPR project. Furthermore, the percent of pavement in the right lane requiring full depth replacement of cracked slabs appears to be a good indicator of a project's suitability for CPR. The following criteria is based on our field observations of the 26 projects:

a. When 5 percent or more of the right lane required full depth replacement, the project was probably not a suitable CPR candidate.

b. When 2 percent or less of the right lane required full depth replacement, and other forms of pavement distress were within reasonable limits, the project was a suitable CPR candidate.

c. Projects requiring between 2 and 5 percent full depth replacement of the right lane were marginal CPR candidates. In these cases, we recommend that pavement deterioration be more closely monitored and evaluated. This will assist in determining whether to undertake CPR.

Of the 19 CPR projects incorporating full depth patching, 14 had a minimum patching dimension of one lane width in the transverse direction. Based on our field observations of the patches, we believe a minimum length of patch in the longitudinal direction should be 6 feet to prevent longitudinal cracking. Full depth concrete patches with dowelled joints provided satisfactory long-term performance. However, patches using the inverted tee method or those with aggregate interlock did not provide satisfactory performance. High cement factors (7 bags or more), Type III cement, and up to 2 percent calcium chloride (by weight of cement) were used to accelerate the concrete mix strength in the full depth patching projects. These projects were opened to traffic in as little as 4 hours and were performing satisfactorily after 8 years.

Partial depth patching was performed on 13 CPR projects. On eight of these projects, less than 5 percent of the total number of patches had failed. Field reviews of the patches and discussions with State engineers showed that a compressible material must be placed in all working joints and cracks within and adjacent to the patch to obtain satisfactory performance. Our field observations also confirmed that partial depth patches should be limited to the top one-third of the slab and should not extend to a depth that allows the dowel bars to bear directly on the patching material. Satisfactory long-term performance (up to 6 years observed) was achieved with standard and high-early strength PCC mixes.

Grinding was performed on 13 CPR projects to improve poor ride quality due to faulting. A ride or profile equal to or better than that for a new concrete pavement was achieved. It appears that specifications for a grinding project could reasonably include profile requirements at least as stringent as those for new PCC pavements. Grinding does not appear to have a significant positive long-term affect on pavement friction. On the four projects where friction data was available, the friction numbers returned to pregrinding levels within 2 years. Several other States reported similar trends.

Seventeen CPR projects included joint resealing. Hot-poured transverse joint sealants were used on seven projects. Those sealants experienced adhesion failure, generally occurring within 2 years after construction. Silicone sealants provided considerably better performance. However, minor adhesion failures were noted in approximately 25 percent of the joints inspected. Discussions with field personnel indicated these failures may be due to improper cleaning of the joints prior to resealing.

The benefits of subsealing could not be readily determined. Field reviews on eight projects in four States showed there was no apparent visual difference in pavement performance between States that had subsealed as part of CPR versus those that did not. Where recommended procedures were followed subsealing did not appear to have any adverse effects on pavement performance.

III. DISCUSSION

PROJECTS SELECTED

Twenty six CPR projects were reviewed in eight States. The projects reviewed and techniques evaluated are listed in Table 1. Pertinent information on each of the pavements rehabilitated is summarized in Table 2.

All of the projects evaluated were jointed concrete pavements. Thirteen of the projects were plain concrete pavements with undowelled joints. The remaining 13 projects were reinforced concrete pavements with dowelled joints. The average age of the pavements at the time of rehabilitation was 18 years, with a range of 10 to 38 years.

Very little traffic loading data was available for most of the projects. However, an attempt was made to classify the current truck loadings on the projects into four groups. The groups are based on daily volume of "5-axle or greater" trucks. This grouping was selected because these trucks generally provide 85 percent or more of the 18-kip equivalent single axle loadings on rural highways. The following groups were selected.

Loading Class	Daily 5 Axle or Greater Truck Volume
1	>1500
2	1001-1500
3	501-1000
4	<501

Table 1. Projects and rehabilitation techniques reviewed.

STATE	ROUTE	PROJECT LIMITS	YEAR REHAB	PAV'T AGE	REHABILITATION TECHNIQUES: SUBSEAL	EDGE DRAINS	PRESSURE RELIEF	FULL DEPTH	PARTIAL DEPTH	DIAMOND GRINDING	JOINT SEALING
CALIFORNIA	I-5	SHASTA CO. MP 3.8 - 14.0	1983	17	Y	Y		Y			Y
	I-80	PLACER CO. MP4 - 11.4	1984	25	Y	Y		Y	Y		
	I-5	YOLO CO. MP 23 - 27.1	1984	18	Y	Y		Y		Y	
GEORGIA	I-75	MP 226 - 232	1981	12	Y					Y	Y
	I-475	MP 0 - 15	1980	13	Y			Y		Y	Y
	I-75	MP 64 - 72	1978	17	Y			Y	Y		Y
	I-75	MP 22 - 59	1978	17	Y			Y			Y
S.CAROLINA	I-85	MP 21 - 30	1979	15	Y			Y		Y	Y
	I-20	MP 0 - 6	1984	17	Y			Y	Y	Y	Y
VIRGINIA	I-81	MP 147.2 - 161.0 NB	1984	19	Y	Y		Y	Y	Y	Y
	I-64	MP 238.4 - 254	1982	19				Y			Y
	I-64	MP 278.7 - 283.3	1983	16			Y	Y			Y
MINNESOTA	I-694	MP 37 - 66	1981	20				Y	Y		Y
	US-10	MP 204 - 211.6	1981	35						Y	
	US-71	MP 124.9 - 129.2	198[illegible]	14						Y	
	I-94	MP 81 - 103	1981	14					Y		Y
WISCONSIN	STH29	CHIPPEWA FALLS TO THORP	1983	16				Y	Y		Y
	USH61	FENNIMORE TO BOSCOBEL RD	1982	30				Y	Y	Y	
	USH151	COLUMBUS - BEAVER DAM RD	1982	20				Y		Y	
	I-90	MP 138 - 142	1981	21				Y		Y	Y
MICHIGAN	I-75	MP 64 - 80	1983					Y			
	M-47	SAGINAW CO. ST.RD TO DIV.	1983	16					Y		Y
SO. DAKOTA	I-29	MP 27 - 62	1972	10					Y		Y
	I-29	MP 0 - 15	1980	19			Y	Y	Y		Y
	I-90	MP 395.5 - 412	1985	24			Y	Y	Y		Y
	I-90	MP 265 - 292.2	1982	17					Y		Y

AVERAGE AGE	18.76
MIN	10.00
MAX	35.00

Table 2. Description of the original pavement on each project reviewed.

STATE	ROUTE	PROJECT LIMITS	YEAR OPENED	LOADS* CLASS	PAV'T TYPE	PAV'T DEPTH (IN.)	JOINT SPACING (FEET)	SKEW '/12	DOWELS	CLIMATE ZONE **	AVG. MONTHLY TEMPERATURE MIN.	AVG. MONTHLY TEMPERATURE MAX.	FREEZE INDEX	BASE MATERIAL	SUBGRADE	YEAR REHAB	PAV'T AGE
CALIFORNIA	I-5	SHASTA CO. MP 3.8 - 14.0	1966	2	JPCP	8	VAR 12 - 19	2	N	IIC	37	97	0	CTB	CLY-GR	1983	17
	I-80	PLACER CO. MP4 - 11.4	1959	1	JPCP	8	15	0	N	IIC	39	95	0	CTB	SD-ST	1984	25
	I-5	YOLO CO. MP 23 - 27.1	1966	2	JPCP	8	VAR 12 - 19	2	N	IIC	38	96	0	CTB	ST-CLY	1984	18
GEORGIA	I-75	MP 226 - 232	1969	1	JPCP	10	30	0	N	IC	35	88	0	CTB	ST-CLY	1981	12
	I-475	MP 0 - 15	1967	2	JPCP	9	30	0	N	IC	38	93	0	ATB	SELECT	1980	13
	I-75	MP 64 - 72	1961	1	JPCP	9	30	0	N	IC	38	93	0	ATB	SELECT	1978	17
	I-75	MP 22 - 59	1961	1	JPCP	9	30	0	N	IC	38	93	0	ATB	SELECT	1978	17
S.CAROLINA	I-85	MP 21 - 34	1964	1	JPCP	9	25	0	N	IC	35	92	0	EARTH	A-7-5&6	1979	15
	I-20	MP 0 - 6	1967	2	JPCP	9	25	0	N	IC	35	91	0	EARTH	A-2-4	1984	17
VIRGINIA	I-81	MP 147.2 - 161.0 NB	1965	3	JRCP	9	61.5	0	Y	IB	29	88	0	CR. AGG.	A-6 & A-7	1984	19
	I-64	MP 238.4 - 254	1963	2	JRCP	9	61.5	0	Y	IC	32	88	0	AGG.	A-6 & A-7	1982	19
	I-64	MP 278.7 - 283.3	1967	1	JRCP	9	61.5	0	Y	IC	32	88	0	CTB	A-3 & A-2	1983	16
MINNESOTA	I-694	MP 37 - 46	1961	1	JRCP	10	40	0	Y	IA	1	81	1567	GRAVEL	A-6	1981	20
	US-10	MP 204 - 211.6	1946	4	JPCP	9-7-9	15	0	N	IA	-2	81	1987	NONE	A-1-B	1981	35
	US-71	MP 124.9 - 129.2	1969	4	JPCP	8.5	20	2	N	IIA	-1	84	1597	AGG.	A-4	1983	14
	I-94	MP 81 - 103	1967	3	JRCP	9	39.3	0	Y	IIA	-4	81	2262	GRAVEL	A-6	1981	14
WISCONSIN	STH29	CHIPPEWA FALLS TO THORP	1967	3	JRCP	9	90	0	Y	IA	3	82	1750	CR. AGG.	GRANULAR	1983	16
	USH61	FENNIMORE TO BOSCOBEL RD	1952	4	JPCP	8	20	0	N	IA	9	84	1000	CR. AGG.	GI = 14	1982	30
	USH151	COLUMBUS - BEAVER DAM RD	1954	4	JPCP	9	80	0	N	IA	8	81	1000	GRAVEL	A-4	1982	28
	I-90	MP 138 - 142	1960	1	JRCP	9	80	0	Y	IA	8	81	1000	GRAVEL	N/A	1981	21
MICHIGAN	I-75	MP 64 - 80	N/A	1	JRCP	9	99	0	Y	IA	19	83	750	N/A	N/A	1983	
	M-47	SAGINAW CO. ST.RD TO DIV.	1967	4	JRCP	9	71	0	Y	IA	15	81	750	N/A	N/A	1983	16
SO. DAKOTA	I-29	MP 27 - 62	1962	N/A	JRCP	9	61.5	0	Y	IIA	5	86	N/A	AGG.	A-7-6-12	1972	10
	I-29	MP 0 - 15	1961	N/A	JRCP	10	61.5	0	Y	IIA	5	86	N/A	AGG.	A-7-6-12	1980	19
	I-90	MP 395.5 - 412	1961	N/A	JRCP	9	61.5	0	Y	IIA	5	86	N/A	GRANULAR	A-7-6-12	1985	24
	I-90	MP 265 - 292.2	1965	N/A	JRCP	9	46.5	0	Y	IIA	5	86	N/A	LIME/AGG	A-7-6-12	1982	17

*LOAD CLASSES ARE BASED ON THE FOLLOWING ONE-WAY VOLUMES OF FIVE AXLE OR GREATER TRUCKS: >1500 = 1, 1001 - 1500 = 2, 501 - 1000 = 3, <501 = 4

AVE AGE 18.76
MIN 10.00
MAX 35.00

**CLIMATE ZONE AS DEFINED IN PART III OF AASHTO "GUIDE FOR DESIGN OF PAVEMENT STRUCTURES"

N/A = NOT AVAILABLE

A plot of pavement age at rehabilitation versus traffic load class is shown in Figure 1. As can be seen, there was no relationship for the projects covered by this review.

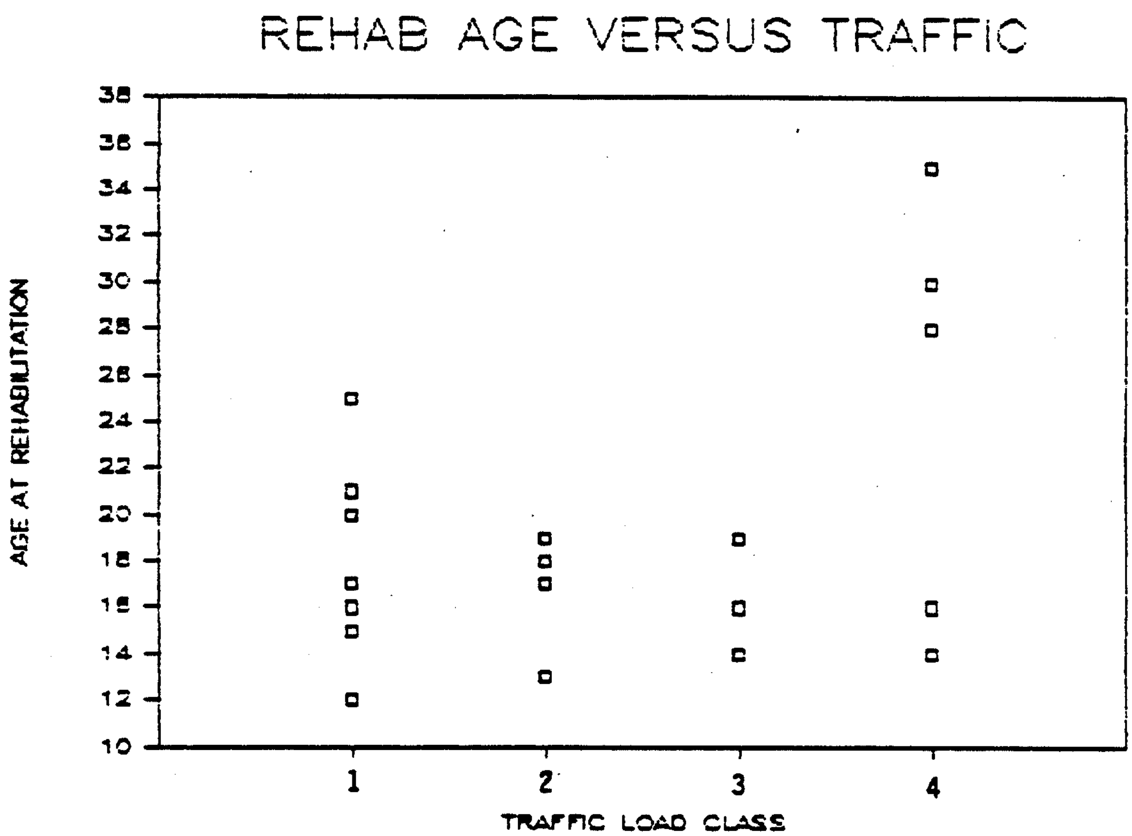


Figure 1. Age of the pavement at the time rehabilitation was required as a function of traffic.

III.B. PERFORMANCE

For each major rehabilitation project reviewed, a subjective evaluation of the performance was made. All projects were expected to provide 6 to 10 years service life. Therefore, if the project required early rehabilitation or major maintenance it was considered to have been a poor overall candidate for rehabilitation.

In addition, the performance of the individual techniques was evaluated. These techniques included full depth patching, partial depth patching, grinding, joint resealing, and subsealing. Very few projects reviewed included pressure relief joints, subdrains, retrofit load transfer devices, and shoulder restoration techniques. Therefore, detailed comments on these techniques are not provided. It was also concluded that it is not possible to evaluate subsurface drainage without performing in-depth testing. A separate project has been initiated to evaluate subsurface drainage on a variety of in-service installations. A detailed discussion of the CPR techniques follows.

OVERALL CPR STRATEGY

On many of the projects reviewed there was very little information available detailing the condition of the pavement prior to the rehabilitation project. This lack of data made it difficult to make before and after evaluations of the effectiveness of the CPR techniques. The absence of information on the rate of pavement deterioration prior to CPR made it difficult to determine how much CPR slowed the rate of deterioration.

In addition to the lack of detailed information on the extent and causes of distress, on most projects the pavement condition was not formally checked during the latter stages of project development. As a result several projects experienced overruns, for full and partial depth patching quantities, exceeding 500 percent. On one project, the quantity of partial depth patching increased from 11,511 square feet to 89,893 square feet for an overrun of $470,292. At least one project was terminated before all CPR work was completed because overruns in quantities exceeded available funds. These findings emphasize the need for detailed condition monitoring throughout the preliminary engineering phase of candidate CPR projects.

Four of the 26 projects reviewed were probably, in retrospect, poor candidates for CPR. This judgement was based on the condition of the pavements 5 years or less after rehabilitation. Three of these projects were in need of major maintenance or complete reconstruction. The other project is showing significant distress less than 2 years after rehabilitation. The principal distress on these projects was the structural failure of the slabs. At the time these projects were rehabilitated, approximately 4.7 to 16.3 percent of the pavement in the right lane was replaced by full depth patching. Generally, additional patching was required, but contract overruns were limited by fiscal constraints. In most cases, full depth patches were constructed to replace slabs that were breaking up.

Of the remaining 22 projects reviewed, 3 had a significant amount of full depth patching to correct joint distress. In general the slabs were in good structural condition other than at the joints. The remaining 19 projects were considered proper candidates for CPR and exhibited satisfactory performance for up to 8 years. The maximum quantity of full depth patching on these 19 projects amounted to 2.6 percent of the right lane surface area.

The following were findings on the use and performance of CPR:

1. An effective CPR strategy will generally reduce the rate of pavement deterioration. Properly designed and constructed CPR techniques can be expected to provide 6-10 years of service life, however, continued maintenance throughout the project's design life will be required.
2. The level of performance of each individual CPR technique was highly dependent on the adequacy of design, quality of construction, and the appropriateness of the technique selected to address the cause of distress. Even with high quality controls a few early failures of the repairs occurred, and maintenance within 1 year following completion of CPR is generally required.
3. Lack of detailed condition data during project development resulted in major overruns.
4. Pavements which have an accelerated rate of slab cracking prior to rehabilitation had a high rate of slab deterioration immediately after completion of a CPR project. It was found that the percent of the right lane requiring full depth replacement of cracked slabs appeared to be a good indicator of a project's suitability for CPR. The criteria listed below is based on field observations of the 26 projects:
   a. When 5 percent or more of the right lane required full depth replacement, the project was probably not a suitable CPR candidate.
   b. When 2 percent or less of the right lane required full depth replacement, and other forms of distress were within reasonable limits, the project was a suitable CPR candidate.

c. Projects requiring between 2 and 5 percent full depth replacement of the right lane are marginal CPR candidates and require careful monitoring to establish the current rate of pavement deterioration.

FULL DEPTH PATCHING

Full depth concrete pavement repairs were reviewed on 19 CPR projects in eight States. The age of the oldest patches observed were 8 years with an average age of 4 years. The projects reviewed are summarized in Table 3.

Three methods of providing load transfer at the transverse boundary between the patch and the existing concrete were observed. Dowels were used on eight projects. Undercutting (inverted tee) was used on four projects. Aggregate interlock only was provided for patches on six projects. In addition, on one project in Wisconsin all three methods were tried.

Full depth patches using dowels for load transfer are providing the best overall performance. A majority of the patches observed where undercutting was used exhibited settlement of the patch and/or faulting at the joint. In many cases both sides of the patch would be lower than the adjacent slabs. Patches with aggregate interlock have severe faulting, which generally appeared to be greater than the faulting on the original pavement.

Patch cracking was not a major distress, although a number of States permitted early opening (4-8 hours) after concrete placement. Typical concrete mix designs required 3000 psi compressive strength in 24 hours. Georgia indicated patches were opened to truck traffic after 6 hours at concrete compressive strengths of 1200-1500 psi with no resulting problems.

Table 3. Summary of full depth patching projects.

STATE	ROUTE	PROJECT LIMITS	YEAR REHAB	PAV'T AGE	PATCH AGE (YR.)	SAW DEPTH (IN.)	MINIMUM SIZE	REMOVAL METHOD	LOAD TRANSFER	CONCRETE MIX STRENGTH	OPENING TIME (HRS)
CALIFORNIA	I-5	-SHASTA CO. MP 3.0 - 14.0	1983	17	3	3	3' x 3'	BREAK	NONE	7 BAG/2% CaCl	4
	I-80	-PLACER CO. MP4 - 11.4	1984	25	2	3	N/A	BREAK	NONE	7 BAG/2% CaCl	4
	I-5	-YOLO CO. MP 23 - 27.1	1984	10	2	3	6' x 6'	BREAK	NONE	7 BAG/2% CaCl	4
GEORGIA	I-475	-MP 0 - 15	1980	13	6	FULL	LANE WIDTH x 15'	LIFT	DOWELS	7.5 BAG/TYPE III/3000 PSI/24 HRS	6
	I-75	-MP 64 - 72	1978	17	6-8	FULL	LANE WIDTH x 15'	LIFT	DOWELS	7.5 BAG/TYPE III/3000 PSI/24 HRS	6
	I-75	-MP 22 - 59	1978	17	8	FULL	LANE WIDTH x 15'	LIFT	DOWELS	7.5 BAG/TYPE III/3000 PSI/24 HRS	6
S.CAROLINA	I-85	-MP 21 - 34	1979	15	7	FULL	LANE WIDTH x 6'	BREAK	UNDERCUT 1 SIDE	8 BAG/TYPE III/2% CaCl	N/A
	I-20	-MP 0 - 6	1984	17	2	FULL	LANE WIDTH x 12'	BREAK	DOWELS	6.5 BAG/TYPE I	N/A
VIRGINIA	I-81	-MP 147.2 - 161.0 NB	1984	19	2	FULL	LANE WIDTH x 2'	LIFT	UNDERCUT BOTH SIDES	8.5 BAG/TYPE III/3000 PSI/24 HRS	48
	I-64	-MP 238.4 - 254	1982	19	4	FULL	LANE WIDTH x 2'	BREAK	UNDERCUT	8.5 BAG/TYPE III/3000 PSI/24 HRS	6
	I-64	-MP 278.7 - 283.3	1983	16	3	FULL	LANE WIDTH x 2'	LIFT	NONE	8.5 BAG/TYPE III/3000 PSI/24 HRS	N/A
MINNESOTA	I-694	-MP 37 - 46	1981	20	5	FULL	NONE SPECIFIED		DOWELS	8.5 BAG/TYPE I/5600 PSI/28 DAYS	7
WISCONSIN	STH29	-CHIPPEWA FALLS TO THORP	1983	16	3	FULL	LANE WIDTH x 6'	LIFT	DOWELS	9 BAG/TYPE I/2-4% CaCl	8
	USH61	-FENNIMORE TO BOSCOBEL RD	1982	30	4	FULL	LANE WIDTH x NO MIN.	LIFT	UNDERCUT	9 BAG/TYPE I	8
	USH151	-COLUMBUS - BEAVER DAM RD	1982	20	4	FULL	LANE WIDTH x NO MIN.	LIFT	DOWELS	9 BAG/TYPE I/2-4% CaCl	8
	I-90	-MP 130 - 142	1981	21	5	FULL	4' x 4'	LIFT	VARIED	6 BAG/TYPE IA	N/A
MICHIGAN	I-75	-MP 64 - 80	1983	N/A	3	FULL	LANE WIDTH x 6'	LIFT	DOWELS (NO GROUT)	2% CaCl/3000 PSI/8 HRS	8
S. DAKOTA	I-29	-MP 0 - 15	1980	19	6	8	LANE WIDTH x 10'	BREAK	NONE	N/A	96
	I-90	MP 395.5-412	1985	24	1	8	LANE WIDTH x NO MIN.	BREAK	NONE	N/A	96

Of the 19 CPR projects incorporating full depth patching, 14 had a minimum patching dimension of one lane width in the transverse direction. The length varied from 2 feet to a full slab length. It was noted that many of the patches less than 4 feet in length developed longitudinal cracking in the middle third of the patch. Most States reviewed specify a minimum patch of a full lane width by 6 foot length. Additional research conducted by others confirms that patches less than 6 feet in length are prone to crack and those less than a full lane width create a weakened plane in the slab.

The review confirms that full depth concrete repairs can be constructed to provide satisfactory long-term performance. The additional expense of the dowel bars appears very cost effective in reducing the recurrence of faulting.

Patching is a major cost item on most CPR projects. Therefore, the rate of continued deterioration of the pavement joints and slabs not repaired must be given careful consideration in the economic (life-cycle cost) analysis of rehabilitation alternatives.

The following were findings on full depth patching:

1. Dowelled jointed full depth concrete patches provide satisfactory long-term performance (up to 8 years observed).
2. Patches should have a minimum dimension of one lane width in the transverse direction and 6 feet in the longitudinal direction.
3. Satisfactory performance was achieved from patches opened to traffic in as little as 4 hours: higher cement factors, Type III cement, and up to

2 percent calcium chloride (by weight of cement) were used to accelerate the strength gain.

PARTIAL DEPTH PATCHING

Partial depth patching was performed on 13 CPR projects. Nine of the projects were jointed reinforced pavements with dowelled joints. The remaining four projects were plain pavements with undowelled joints. On all projects reviewed, the partial depth patches were generally used to correct spalling at transverse contraction joints.

No problems were indicated with partial depth saw cuts and concrete removal. Georgia noted excellent results with their procedure. They specify a series of parallel partial depth saw cuts within patch boundaries to facilitate concrete removal with lightweight chipping hammers (30 lb. maximum size).

The patching materials included "Set 45" on one project in Michigan and high aluminum cement on one project in California. The remaining projects were patched with PCC using either Type I or Type III cement. Some of the projects used up to 2 percent calcium chloride (by weight of cement) as an accelerator.

The cement content for the PCC patches ranged from 6.5 to 8.5 bags. Air contents were between 5.5 and 7 percent. A bonding agent was used on all of the PCC patches. The most commonly used material was a sand-cement slurry applied to either a dry or damp surface. On several projects, epoxy resin was applied to a dry surface.

The projects reviewed, specifications, and comments on performance are outlined in Table 4. The performance of the partial depth patching projects is summarized as follows:

Failure Rate (Percent of Patches)	Number of Projects
<5 percent	8
15 percent	1
75 percent	2
>90 percent	2

The only consistent difference between the projects having a low failure rate versus those with greater than 5 percent was the use of compressible material in the joints. One of the projects with a 75 percent failure rate required a compressible material in the joints, however, the depth of many of the patches was reported to be below the depth of the dowel bars. The specifications on the project required the compressible material to be placed to the top of the dowels. A bond breaker was to be used below the dowels.

Problems were observed on several projects where partial depth patches extended into the dowel bars. It is very difficult to place the compressible material in the joint adjacent to the dowels. Also, the dowel to patch contact may result in excessive stresses in the patch and at the interface of the patch and existing concrete. These stresses develop during joint movement or during curling and warping of the slab.

Table 4. Summary of partial depth patching projects.

PARTIAL DEPTH PATCHING

Project	Depth Saw Saw	PCC Removal	Prep.	Patch Mat.	Bond Agent	Surface (Dry/Wet)	Air Temp	Joint Form	Remarks
<u>California</u>									
I80-MP-4-11-.4	1.5"	Air Hammer	Sand blast	High Aluminum Cement	Epoxy	Dry	>45	Styrofoam or Cardboard	<than 5 percent failure
<u>Georgia</u>									
I475, MPO-15	Depth of Failure, 2" beyond spall	Air Hammer	Sand blast	8 bags Type III, 3000 psi/ 24 hours	Unknown	Unknown		None Saw within 24 hours	Most of the original partial depth patches have been replaced (>90% failure)
<u>South Carolina</u>									
I20, MPO-6.0	2" depth, 6" beyond spalls	Air Hammer Min. length 12" Min. width 12" Max. depth 5"	Sand blast Air	6.5 bags Type I, 2000 psi/min to open to traffic	Epoxy resin	Dry		Insert 5" deep, 6" beyond each side patch	Patches looked good, 1 failure in a 1/2 mile sample. (<5% failure)
<u>Virginia</u>									
I81, MP147.22 to 161.77	3"	Air Hammer 15 lbs. Max. Min. length 12" Min. width 12" Min. depth 3" Max. depth 4"	Water & Air blast	8.5 bags Type III Cement, 3000 psi/ 24 hours	Cement Slurry	Damp		Bituminous expansion board	Partial depth patches were performing good. (<5% failure)

PARTIAL DEPTH PATCHING (Con't)

Project	Depth Saw Saw	PCC Removal	Prep.	Patch Mat.	Bond Agent	Surface (Dry/Wet)	Air Temp	Joint Form	Remarks
<u>Minnesota</u>									
I-694, MP37 to MP46	1"	Air Hammer 30 lbs. max. Min. depth 1" Max. depth 5"	Sand blast & Air	8.5 bags Type I Cement, 5600 psi/ 24 hours, 5.5% Air	Equal parts Sand & Cement, Water to produce a stiff slurry	Dry		20 mil. plastic	Approximately 15% have failed, common distress was noted as spalling at edges.
I-94, MP81 to MP 103	1"	Air Hammer 30 lbs. max. Min. depth 1" Max. depth 5"	Sand blast & Air	8.5 bags Type I, 5600 psi/ 24 hours, 5.5% Air	Equal parts Cement, water to produce stiff slurry	Dry		Compressible materials	<5% showed any distress and none observed had failed completely.
<u>Wisconsin</u>									
S.T.H. 29 Chippewa Falls to Thorp	2"	Air Hammer Min depth 2" Max not spec.	Brooming & Water pressure	Portland Cement Concrete Type 1.	Acryl 60	Dry		Premolded filler strip or sawing	Approximately 75% of patches have failed.
U.S.H. 61 Feminore to Boscobel Rd.	2 1/2"	Air Hammer Min depth 2 1/2" Max depth-Bottom of slab	Water blasting	7 bags Type IA	Acryl 60	Dry		None-Tooled Surface	Most were loose. (>90% have failed)

PARTIAL DEPTH PATCHING (Con't)

Project	Depth Saw	PCC Removal	Prep.	Patch Mat.	Bond Agent	Surface (Dry/Wet)	Air Temp	Joint Form	Remarks
<u>Michigan</u>									
M 47 Saginaw Co. N58 to US10	1 3/4"	Air Hammer 15# Spec. 30# Actual for some	Compres. Air	Set 45	None	Damp		Polyurethene Foam	Approximately 2% of the patches have failed.
South <u>Dakota</u>									
I-90 MP265-M292.22	1"	Light Jack Hammer	Sand blast	7 bags Type III		Dry		Joints formed to top of dowels only.	0% failure
I-29 MP MP 27-66				7 bags Type III Air 7 ± 2	Cement 1 part sand 1 part cement 1/2 part water	Dry		See above	Bottom of patches were at the dowels or below. (75% failure)
I-29 MP 0-15	1"	Air Hammer 30# Max		7 bags Type III	Cement Grout	Dry		See above	<5% failure.
I-90 MP 395.5 412	1"	Air Hammer 30# Max	Sand blast	7 bags Type III		Dry		See above	<5% failure.

The following were findings on partial depth patching:

1. A compressible material must be placed in all working joints and cracks within or adjacent to the patch. The compressible material should extend 1 inch below and 3 inches laterally beyond patch boundaries.
2. Partial depth patches should be limited to the top one-third of the slab and should not extend to a depth that allows the dowel bars to bear directly on the patching material.
3. Standard and high-early strength PCC mixes provided satisfactory long-term performance (up to 6 years observed).

GRINDING

Thirteen projects reviewed included diamond grinding. Eleven of these projects were on plain undowelled pavements. The purpose of grinding was to improve poor ride quality due to faulting.

Before and after ride or joint faulting data were limited. Based on the data available and comments by personnel familiar with the projects, grinding appears to provide a significant long-term improvement in ride quality. Continuous grinding of the entire lane achieved uniform ride, friction, and appearance. It was noted that grinding in the direction of or against traffic flow met ride specifications. Several States prefer to grind in the direction of traffic as a safety precaution.

Pavement texture was another area that was discussed. Three of the six States that performed grinding require the contractor to adjust the blade spacing to achieve a specified texture. Wisconsin specifies that 95 percent of any 3 foot by 100 foot section being ground must meet the texture (depth and spacing) requirements.

A ride or profile equal to or better than new concrete pavement can be obtained through grinding. It appears that specifications for a grinding project could reasonably include profile requirements at least as stringent as those for new PCC pavements. Data on the projects with grinding are summarized in Table 5.

Performance to date indicates 5 to 10 years of service life can be expected before faulting returns to the condition existing prior to grinding. However, grinding is only appropriate if the project meets criteria for CPR as previously outlined. Figures 2 and 3 are plots of the faulting index recorded on two Georgia grinding projects. Georgia's faulting index is the sum of 1/32 inches of faulting over five consecutive joints. Both these projects were on pavements in the high loading category. The data indicates that faulting is still well below the prerestoration condition 5 years after the grinding was completed.

On the four projects where friction data was available, grinding did not provide a long-term increase in pavement friction. On one project in California, the pavement had an average SN 40 of 38, 3 years prior to grinding as compared to an average of 35, 1 year after grinding. Friction data for a project in Georgia is shown in Figure 4. The figure indicates that there is a significant increase in friction number immediately after grinding, however, the friction numbers generally returned to their pregrinding levels within about 2 years. Several other States reported similar trends.

Table 5. Summary of grinding projects.

GRINDING

Project	Extend	Friction Before	Friction After	Ride Before	Ride After	Specifications Profile	Specifications Equipment	
California								
I5, Yolo MP 23-27.1	All lanes entire project	3 years before 38	1 year after 35	3 years before PCA-29 PSI=2.6	1 year after PCA-12 PSI=3.8	California Profileograph 15"/mile	None	
Georgia								
I75, MP227 to 232	Outer lane (by Maint.)	see figures		see faulting index		PCA Roughness 300 max., if >300 must meet Rainhart of <7" mile		
1475, MP0-15	All lanes for entire project by State forces	see figures		see faulting		work performed by maintenance		
175, MP64-72	Outside lane					PCA <300 Areas not meeting PCA to meet Rainhart 7"/mi. Diff. at Jts. shall not exceed 1/6" Cross slope 1/4" in 12'	Self pro- pelled machine Peaks 1/32 higher than bottom 60 grooves/ft. 60 grooves/ft	
175, MP22 to 59	Outside lane					"	"	Ride good

Project	Extend	Friction Before	Friction After	Ride Before	Ride After	Specifications Profile	Specifications Equipment	
South Carolina								
I-85, MP21-34	All lanes		N/A		N/A	1/8" in 10'	Machines using Diamond	Average faulting 1/16"-1/8" with areas of 1/4"
120 MP 0-6.0	All lanes		N/A		N/A	Mayes Meter <55 inches per mile	"	Diamond grinding saw blade spacing appeared to have been wide resulting in high fines.
Virginia								
I81, MP147-16	All lanes	35	N/A			1/8" in 10 ft.	Power Driver Self propelled Min wt. 15,000 lbs. Min. HP 200	
Minnesota								
US10, MP204-211	All lanes					3/16" in 10' parallel to centerline 1/8 in 3' at right angle to centerline 1/16" at joints		Grooves between 0.097 and 0.128" wide, spaced 0.062" - 0.115" apart. Depth = 0.31 - 0.115'
US71,MP124-129	All lanes					"		" "

N/A = Not Availab'

GRINDING

Project	Extend	Friction		Ride		Specifications		
		Before	After	Before	After	Profile	Equipment	
<u>Wisconsin</u>								
USH 61 Fennimore to Boscobel	All lanes							95% of any 3' x 100' section 1/16" from top to bottom 50 grooves/ft. min.
U.S.H. 151 Columbus-Beaver Dam	All lanes					1/8" or 10' or 0.3" in 25'		" "
I-90,MP138-142	All lanes					Transverse 1/8" in 3.		" "

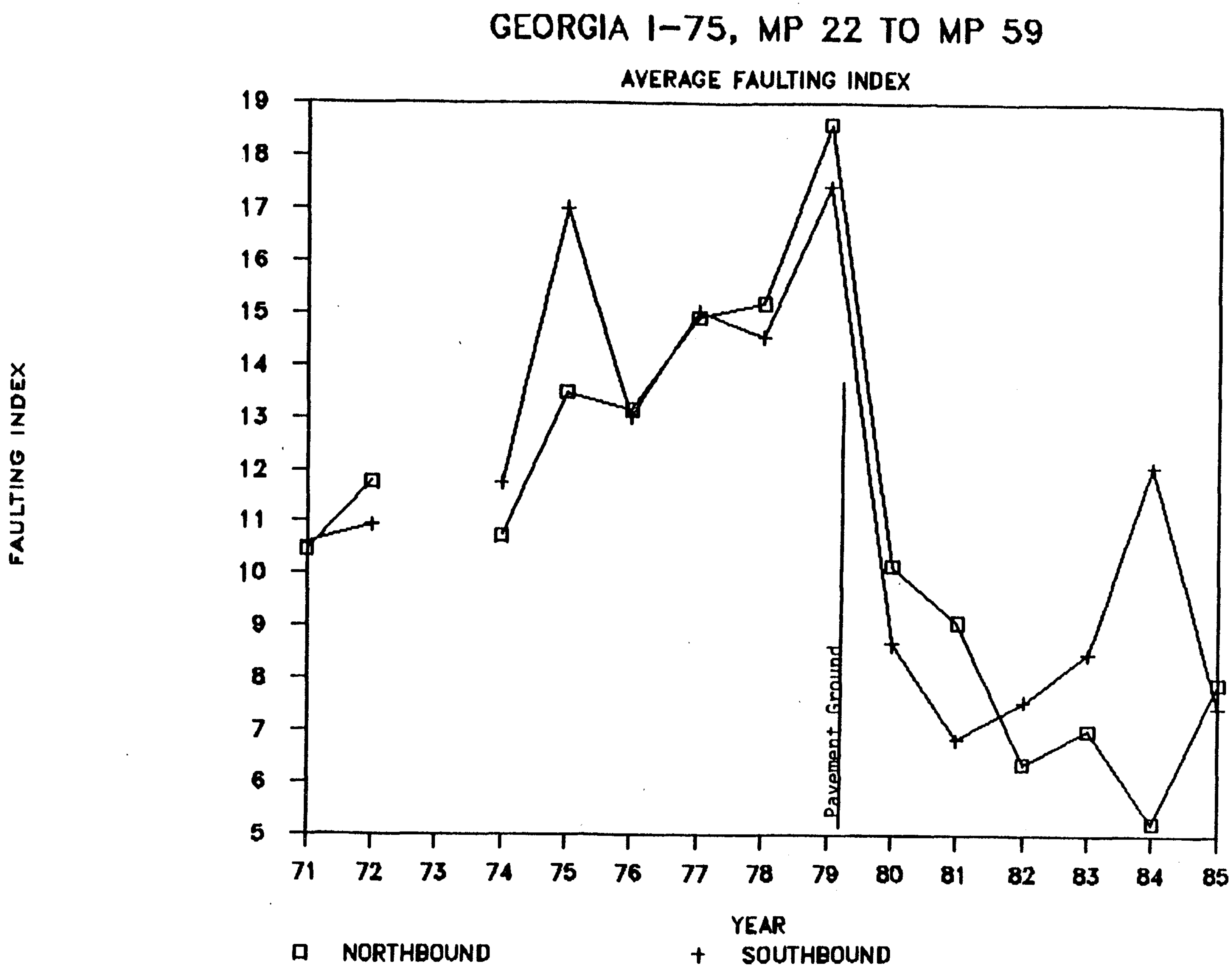


Figure 2. Faulting index before after grinding.

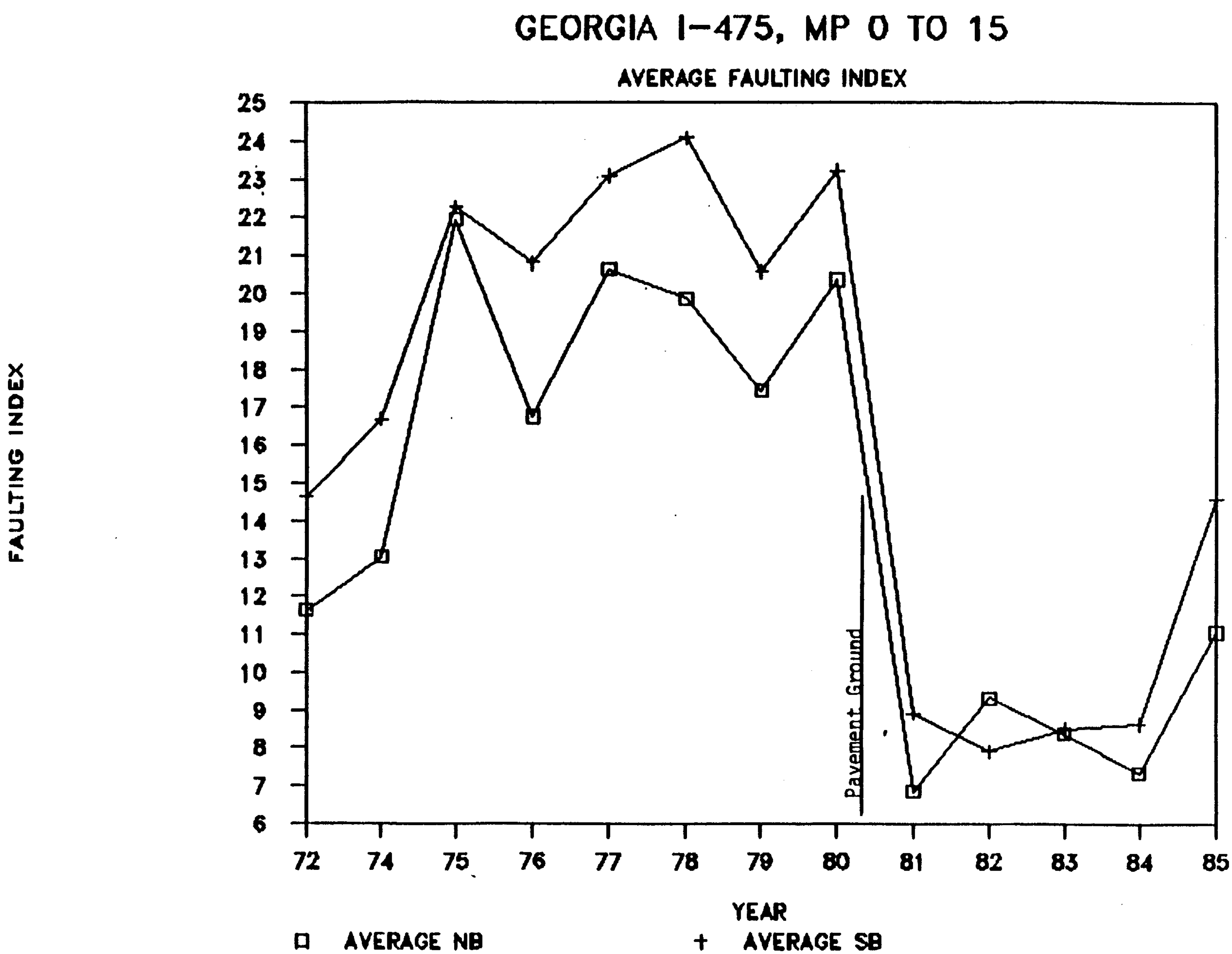


Figure 3. Faulting index before and after grinding.

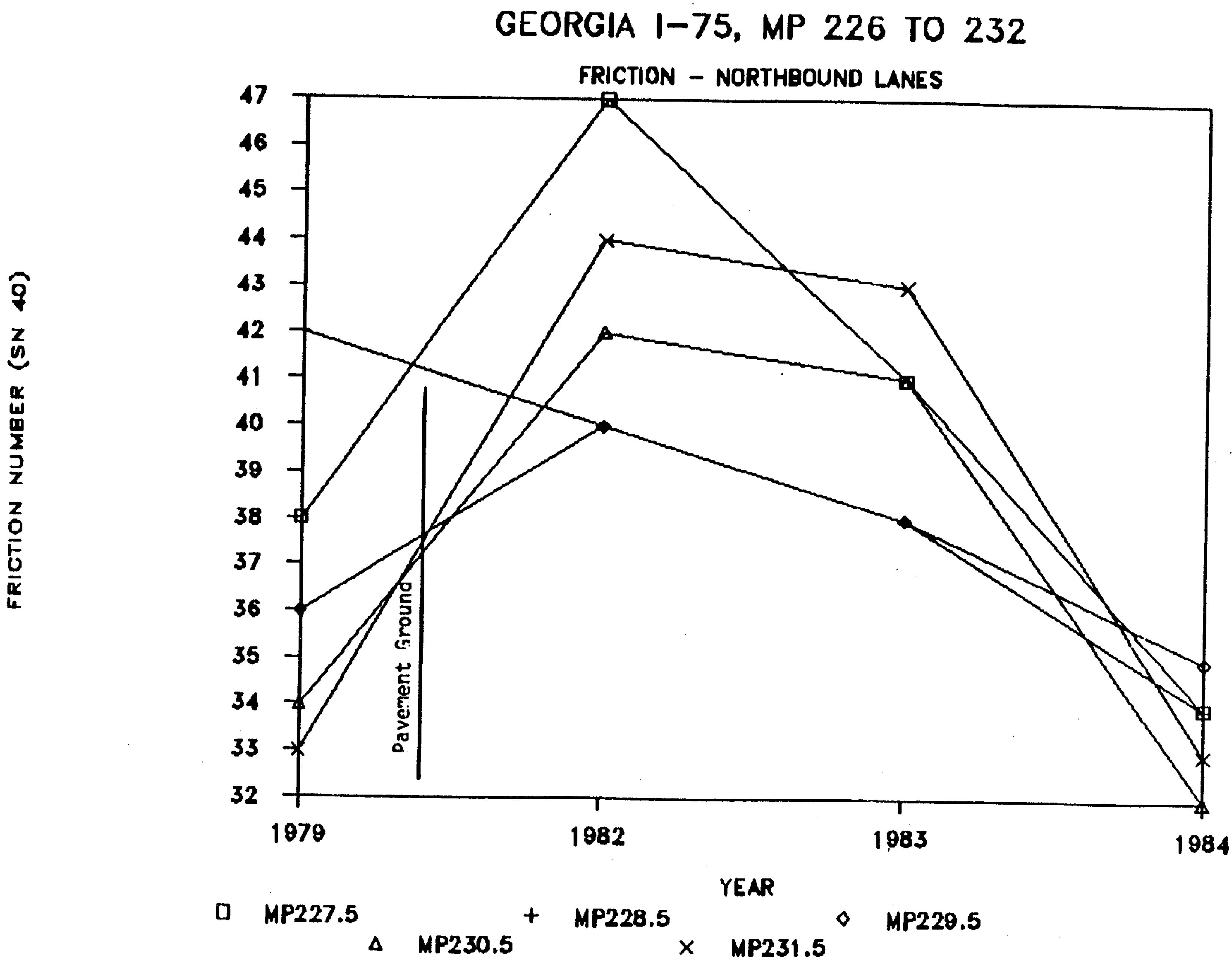


Figure 4. Friction before after grinding.

The following were findings on grinding:

1. A significant long-term improvement in ride quality can be obtained through grinding on CPR projects. Performance to date indicates a service life of 5 to 10 years can be expected before faulting returns to the pregrind condition.
2. Grinding does not appear to provide a significant long-term increase in pavement friction. Available data indicated that skid number values returned to their pregrind levels within 2 years.
3. Ride specifications were met when grinding was performed in either direction with respect to the flow of traffic.

JOINT RESEALING

Fourteen of the CPR projects included joint resealing. It should be noted that the review concentrated primarily on resealing of the transverse joints. As such, the following observations and subsequent findings are applicable to transverse joints unless indicated otherwise. The projects reviewed are summarized in Table 6. Eleven projects used one type of sealant material exclusively — six used silicone, four used hot-poured sealant, and one used neoprene. The remaining three projects used two sealant materials.

On the projects inspected, the hot-poured joint seals experienced adhesion failures, generally within 2 years after construction. Preformed neoprene seals appear better suited to new construction. Because minor joint spalling is generally present in rehabilitation projects, the full bearing required for preformed seals to remained compressed and in place may not be achieved.

Table 6. Summary of Joint Resealing Projects.

STATE	ROUTE	PROJECT LIMITS	YEAR REHAB	PAV'T AGE	TYPE OF JOINT	SEALANT MATERIAL	SEALANT RESERVOIR SHAPE WIDTH (IN.)	DEPTH (IN.)	TOP BELOW SURFACE	BACKER ROD	COMMENTS Page 1
CALIFORNIA	I-5	SHASTA CO. MP 3.8 - 14.0	1983	17	Transverse	Rubber-asphalt	1/2	1 1/4	N/A	Not required	Adhesion failures were noted.
					Longitudinal	Rubber-asphalt	1/2	1 1/4	N/A	Not required	
					Shoulder(Long.)	Not Sealed					
GEORGIA	I-75	MP 226 - 232	1981	12	Transverse	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	Minor failures at 25-50% of joints
					Longitudinal	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	
	I-475	MP 0 - 15	1980	13	Transverse	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	Joints regularly maintained. Most in good condition.
					Longitudinal	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	
	I-75	MP 64 - 72	N/A		Transverse	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	Installed by Maintenance. Condition still good.
	I-75	MP 22 - 59	N/A		Transverse	Low Mod. Silicone (Dow)	3/8-5/8	1 1/2	1/2	Polyethylene	Installed by Maintenance. Condition still good.
S.CAROLINA	I-85	MP 21 - 34	1979	15	Transverse	Hot-poured ASTM D3405	3/8-1/2	1 1/4-1 1/2	N/A	Upholstery chord	Sealant failed in adhesion.
					Long.(ctr.line)	Not sealed					
					Shoulder(Long.)	Hot-poured ASTM D3405	3/4	3/4	N/A		
	I-20	MP 0 - 6	1984	17	Transverse	Low Mod. Silicone (Dow)	Manufacturer's Recommendation			Polyethylene	Minor adhesion failures in 25% WB, 2% EB.
					Longitudinal	Low Mod. Silicone (Dow)	Manufacturer's Recommendation			Polyethylene	
					Shoulder(Long.)	Low Mod. Silicone (Dow)	1/4	1/4	1/2		
					Shoulder(Trans)	Low Mod. Silicone (Dow)	3/8-1/2	2 1/2	Flush		
VIRGINIA	I-81	MP 147.2 - 161.8 NB	1984	19	Trans. (1-1/8"	Low Mod. Silicone (Dow)	Varied	Varied	1/4	Polyethylene	Numerous adhesion failures,possibly due to aggregate incompatibilty.
					Trans.)1-1/8"	Pref. Compression Seal					Seals looked to be in good condition.
					Long. (1-1/8"	Low Mod. Silicone (Dow)	Varied	Varied	1/4	Polyethylene	
					Long.)1-1/8"	Pref. Compression Seal					
	I-64	MP 238.4 - 254	1982	19	Transverse	Hot-poured Elastomeric	1/2	1/2	Flush	N/A	All failed in adhesion.
					Longitudinal	Hot-poured Elastomeric	1/2	1/2	Flush	N/A	
	I-64	MP 278.7 - 283.3	1983	16	Transverse	Rubber-Asphalt	3/4	3/4	Flush	N/A	Adhesion failures were noted.
					Longitudinal	Rubber-Asphalt	3/4	3/4	Flush	N/A	

N/A = NOT AVAILABLE

Table 6 con't.

STATE	ROUTE	PROJECT LIMITS	YEAR REHAB	PAV'T AGE	TYPE OF JOINT	SEALANT MATERIAL	SEALANT RESERVOIR SHAPE WIDTH (IN.)	DEPTH (IN.)	TOP BELOW SURFACE	BACKER ROD	COMMENTS
MINNESOTA	I-694	MP 37 - 46	1981	20	Transverse	Rubber-Asphalt	3/4	3/4	0-1/8	N/A	Sealants failed in adhesion.
					Longitudinal	Rubber-Asphalt	3/4	3/4	0-1/8	N/A	
	I-94	MP 81 - 103	1981	14	Transverse	Rubber-Asphalt	3/4	3/4	0-1/8	N/A	Sealants failed in adhesion.
					Longitudinal	Rubber-Asphalt	3/4	3/4	0-1/8	N/A	
WISCONSIN	STH29	CHIPPEWA FALLS TO THORP	1983	16	Transverse	Hot-poured Elastomeric	1/2-3/4	1/2-1	1/8-1/4	N/A	All failed in adhesion.
					Longitudinal	Hot-poured Elastomeric	1/2-3/16	1/2	1/8-1/4	N/A	
	I-90	MP 130 - 142	1981	21	Transverse	Low Mod. Silicone (Dow)	3/8	1/2	1/8	Polyethylene	75% had failed in adhesion.
					Longitudinal	Hot-poured ASTM D3405	(Exist.)	3/4	1/4	N/A	
MICHIGAN	M-47	SAGINAW CO. ST.RD TO DIV.	1983	16	Transverse	Neoprene	(Exist.)	(Exist.)	1/4	None	
					Longitudinal	Sof Seal	1/4-3/8	1-1 1/4	1/8	Polyurethane	50% had failed in adhesion.
SO. DAKOTA	I-29	MP 27 - 62	1972	10	Transverse	Neoprene	N/A	N/A	N/A	N/A	Construction data not available.
	I-29	MP 0 - 15	1980	19	Transverse	Low Mod. Silicone (Dow)	7/8	1/2	1/4	Polyethylene	
					Longitudinal	Not sealed					
	I-90	MP 395.5 - 412	1985	24	Transverse	Low Mod. Silicone (Dow)	7/8	1/2	1/8	Polyethylene	
					Longitudinal	Hot rubber-asphalt	7/8	1 7/8	1/8		
					Shoulder (Long.)	Not Sealed					
	I-90	MP 265 - 292.2	1982	17	Transverse	Low Mod. Silicone (Dow)	N/A	N/A	N/A	Polyethylene	

N/A = NOT AVAILABLE

A majority of projects with joints resealed using silicone material are providing good performance. However, periodic maintenance is required to cut out and repair isolated failures. Where field personnel indicated that manufacturer's guidance for installation was not followed explicitly, significant failures (up to 75 percent) were observed. Items such as refacing the joint to allow for the proper shape of the joint sealant material, providing a clean and dry bonding surface, and close inspection of the application and tooling of this sealant are critical to performance.

Several projects included sealing of the longitudinal asphalt shoulder joint with hot-poured sealants. This material appears to be effective for about 2 years before maintenance is required.

The following were findings on joint resealing:

1. Preformed neoprene joint seals are generally not suitable for rehabilitation projects because even a small amount of spalling in the existing joint can result in failure of the seal.
2. Hot-poured sealants observed experienced significant adhesion failure, generally within 2 years.
3. Silicone, when properly installed, provided good performance. However, in all cases, maintenance was required after 1 to 2 years to correct construction deficiencies.
4. Hot-poured sealants used in the longitudinal asphalt shoulder joint requires maintenance on about a 2 year cycle.

SLAB STABILIZATION (SUBSEALING)

Eight projects were reviewed in four States where subsealing had been used as a rehabilitation technique. Criteria for subsealing ranged from blanket subsealing of all faulted slabs to subsealing only those slabs which exceeded a minimum specified deflection under loading. The projects reviewed are summarized in Table 7.

It was difficult to evaluate the effectiveness visually. However, an attempt was made to determine if the subsealing had any apparent detrimental affects. Slabs were closely observed for cracks which passed through or radiated from the subsealing holes. No distress of this type was observed. One project had a high level of slab breakup following a CPR project which included subsealing. However, a review of the project records and discussion with project personnel indicated that there was a high rate of deterioration occurring prior to the CPR project.

A research project is currently underway to document and evaluate undersealing techniques. A product of that project will be a users manual for undersealing.

The following was the finding on subsealing:

1. Although the benefits of subsealing could not be readily observed, there appeared to be no adverse affect to pavement performance when procedures outlined in FHWA's "Pavement Rehabilitation Manual", "Techniques for Pavement Rehabilitation" course notebook, or the "1985 AASTHO-AGC-ARTBA Joint Committee's Guide Procedures for Concrete Pavement 4R Operations" were followed.

Table 7. Summary of Subsealing projects.

Project	Delfection	Grout	Hole Pattern	SUBSEALING Pressure	Depth of Hole	Min. Air Temp.	Holes Plugged	Lift	Remarks
<u>California</u>									
15, Yolo MP23-27.1	None available	2.5 parts Pozzo-lan 1 part cement time efflux 10-16	3-5 in leave slab	none spec-fied	15 inches	45 F	Wooden pegs	1st move-ment	No deterioration which could be related to sub-sealing noted.
180 Placer MP4.0-11.4	see remarks	" "	3-4 in leave slab	"	15 inches	45 F	Wooden pegs	1st move-ment	Deflections were taken before and after subsealing. When taking deflections the difference between the deflection of loaded and unloaded side were measured. before 0.007 inches after 0.008 inches
15, Shasta MP 3.8 to 14	load transfer only	3 parts pozzolan 1 part cement time efflux 11-16 spec., 10-11 actual, comp. strength spec. 750 psi/ 7 days, Actual 1430 psi/ 7 days	3-4 in leave slab	"	15 inches Nozzle not to extend below bottom of slab			1st move-ment but not moni-tored in all slabs	

SUBSEALING (Cont.)

Project	Delfection	Grout	Hole Pattern	Pressure	Depth of Hole	Min. Air Temp.	Holes Plugged	Lift	Remarks
<u>Georgia</u>									
I75, MP 227-232	Min 0.025"								No deflection >0.025 in. so subsealing was deleted.
I475, MPO-15	Min 0.005"	3 parts lime-stone 1 part cement time efflux 16-22 sec	2 holes 18" from leave, 4 additional along outer edge		8" below bottom of slab		No	1/8"	
<u>South Carolina</u>									
I85, MP 21 to 34	Tested under 18 kip Axle if movement visually observed it was subsealed	3 parts ag. lime, 1 part type III cement, water content to give slurry appear-ance of thick cream	1 hole on approach side, 4 on leave side	None speci-fied	Bottom of slab	35 and rising 40 and falling	Wooden pegs	When dial indicates movements Max. 1/8"	Slabs retested and regrout as necessary.

Project	Delfection	Grout	Hole Pattern	Pressure	Depth of Hole	Min. Air Temp.	Holes Plugged	Lift	Remarks
<u>South Carolina</u>									
I20, MPO-6	Min 0.020"	3 parts ag. lime, 1 part cement time efflux 14-22 sec	2 hole on approach 4 on leave	None speci-fied	8" to 10" below slabs	35 and rising	Wooden pegs	<1/8" any slab raised more than 1/8" replaced.	Slabs retested and any with movement >0.020 in. was regrouted.
<u>Virginia</u>									
I81, MP 147 to 161 NB	Not performed. All slabs in RT Lane were subsealed.	3 parts Pozzolan 1 part cement time of efflux 10-16 sec Mean compres-sive Str. 1 day 300 psi 3 day 620 psi 7 day 1000 psi	7 holes per slab	200 psi max.	Bottom of slab	35 F	Wooden pegs	0.125 in. per hole. Many holes pumped to max.	

VI. COST DATA

Available cost data for the rehabilitation techniques used on each project are listed in Table 8. Also shown are the planned and final quantities. The data is presented by project to give the reader a feel for the total scope of each project.

Table 8. Summary of quantities and bid prices.

STATE	ROUTE LIMITS	YEAR REHAB.	TECHNIQUE	UNITS	BID PRICE	QUANTITIES PLAN	QUANTITIES FINAL	COMMENTS
CALIFORNIA	I-5 SHASTA CO. MP 3.8 - 14.0	1983	Slab Replacement	sq. yd.	$36.27	8970	11307	No load transfer.
			Corner Patch	sq. yd.	$214.33	45	45	
			Subseal Hole	ea.	$4.50	21600	32521	
			Grout	cwt.	$15.00	11000	19562	
	I-80 PLACER CO. MP4 - 11.4	1984	Full Depth Patches	sq. yd.	$63.95	2340	2588	No load transfer.
			Partial Depth Patches	sq. yd.	$250.00	210	191	
			Subseal Hole	ea.	$4.50	14200	13181	
			Grout	ton	$320.00	230	222	
	I-5 YOLO CO. MP 23 - 27.1	1984	Full Depth Patches	sq. yd.	$73.33	2073	2706	No load transfer.
			Subseal Hole	ea.	$5.50	6318	5813	
			Grout	ton	$1,600.00	158	52	
			Grinding	sq. yd.	$2.99	95000	N/A	
GEORGIA	I-75 MP 227 - 232	1981	Subseal Hole	ea.	$3.00	2500	0	
			Grout	bags cem.	$27.50	600	0	
			Subseal Pre. Test	mile	$1,200.00	12	12	
			Subseal Stab. Test	joint	$600.00	800	0	
			Grinding	sq. yd.	$4.04	80800	80761	Includes traffic control.
			Joint Seal	lin. ft.	$0.64	167000	133826	Low mod silicone.
			Sawing Joints	lin. ft.	$0.50	167000	133826	
	I-475 MP 0 - 15	1980	Slab replacement	cu. yd.	$90.00	240	457	Dowelled load transfer.
			Slab removal	sq. yd.	$70.00	600	1259	
			Partial Depth Patches	sq. ft.	$19.50	4000	7869	
			Subseal Hole	ea.	$4.42	5300	4678	
			Grout	bags cem.	$19.50	1100	494	
			Subseal Pre. Test	mile	$1,050.00	20	20	
			Subseal Stab. Test	joint	$556.00	890	820	
			Grinding	sq.yd.				Done by maintenance forces
			Saw and Reseal	lin. ft.	$1.61	92300	80742	Existing joints were Uni-tube. Low mod silicone.
			Saw and Reseal	lin. ft.	$1.40	154200	154309	Other joints and cracks. Low mod silicone.
	I-75 MP 64 - 72	1978	Grinding	sq. yd.	$3.49	N/A	126900	
	I-75 MP 22 - 59	1978	Grinding	sq. yd.	$3.07	N/A	232600	
S. CAROLINA	I-85 MP 21 - 34	1979	Full Depth Patches	sq.yd.	N/A	3800	N/A	No load transfer.
			Subseal Hole	ea.	N/A	2375	N/A	
			Grout	bags cem.	N/A	1068	N/A	
			Grinding	sq. yd.	N/A	361891	N/A	Continuous grinding.
	I-20 MP 0 - 6	1984	Full Depth Patches	sq.yd.	$110.80	750	2155	Dowelled load transfer.
			Partial Depth Patches	sq. ft.	$24.00	1000	9555	
			Subseal Hole	ea.	$4.00	12240	12459	
			Grout	bags cem.	$22.00	4080	1472	
			Grinding	sq.yd.	$2.40	217621	183551	Continuous grinding.
			Joint Sealing	lin. ft.	$1.53	170311	171136	Existing jts. were Uni-tube. Low mod silicone.
			PCC Shoulder 4 ft.	sq. yd.	$14.75	36448	34431	Full depth retrofit shoulders.
			PCC Shoulder 6 ft.	sq. yd.	$14.05	8548	9142	Full depth retrofit shoulders.
			PCC Shoulder 10 ft.	sq. yd.	$13.55	78757	61984	Full depth retrofit shoulders.

Table 8 con't.

STATE	ROUTE LIMITS	YEAR REHAB.	TECHNIQUE	UNITS	BID PRICE	QUANTITIES PLAN	QUANTITIES FINAL	COMMENTS
VIRGINIA	I-81 MP 147.2 - 161.8 NB	1984	Full Depth Slab Rep.	sq. yd.	$100.00	1804	3502	Final quantities represent only one-half of entire project. Other half was readvertised. Inverted tee if patch 2' to 42'. Dowelled if longer.
			Partial Depth Patches	sq. yd.	$80.00	0	87	
			Subseal Hole	ea.	$2.00	9885	10956	
			Grout (Cement)	cu. ft.	$25.00	2275	10009	
			Grinding	sq. yd.	$2.00	202611	180851	
			Joint Sealing	lin. ft.	$1.30	104985	61980	Silicone
			Joint Sealing (2" pref.)	lin. ft.	$12.00	7001	808	Neoprene compression seal.
			Joint Sealing (3" pref.)	lin. ft.	$20.00	1046	871	Neoprene compression seal.
			Joint Sealing (3.5" pref.)	lin. ft.	$40.00	38	234	Neoprene compression seal.
			Edge Drains	lin. ft.	$10.00	16685	8835	
	I-64 MP 238.4 - 254	1982	Full Depth Patches	sq. yd.	$109.00	2300	2721	Final quantities represent only one-half of entire project. Project was terminated. Inverted tee.
			Joint Sealing	lin. ft.	$0.55	174010	79644	Hot poured elastomeric.
	I-64 MP 278.7 - 283.3	1983	Full Depth Patches	sq. yd.	$89.00	9323	8221	No load transfer.
			Joint Sealing	lin. ft.	$0.65	96222	150516	Rubber asphalt joint sealant.
			Pressure Relief Joint	lin. ft.	$25.00	2300	2044	
MINNESOTA	I-694 MP 37 - 46	1981	Full Depth Patches	N/A				Full and Partial depth patch quantities were intermixed. Partial depth patch boundaries are tapered. Dowelled load transfer.
			Partial Depth Patches	N/A				
	US-10 MP 204 - 211.6	1981	Grinding	sq. yd.	$2.80	59269	N/A	
	US-71 MP 124.9 - 129.2	1983	Grinding	N/A				Quantities and costs not available.
	I-94 MP 81 - 103	1981	Partial Depth Patches	cu. ft.	N/A	500	4421.	Boundaries are tapered.
			Joint Sealing	lin. ft.	N/A	271707	278500	Hot poured rubber.
			Crack Sealing	lin. ft.	N/A	6275	2864	
WISCONSIN	STH29 CHIPPEWA FALLS TO THORP	1983	Full Depth Patch 6' by 12'	sq. yd.	$47.00	14136	15440	Dowelled load transfer.
			Full Depth Patch 8' by 12'	sq. yd.	$42.84	960	1397	Dowelled load transfer.
			Full Depth Patch 10' by 12'	sq. yd.	$39.37	400	733	Dowelled load transfer.
			Full Depth Patch 12' by 12'	sq. yd.	$37.81	176	416	Dowelled load transfer.
			Full Depth Patch 16' by 12'	sq. yd.	$35.63	427	1024	Dowelled load transfer.
			Full Depth Patch 20' by 12'	sq. yd.	$34.50	720	347	Dowelled load transfer.
			Partial Depth Patches	sq. ft.	$9.00	2000	4154	
			Longitudinal Joint Sealing	lin. ft.	$0.85	167000	152898	Hot poured elastic.
			Transverse Joint Sealing	lin. ft.	$0.90	40000	37807	Hot poured elastic.
	USH61 FENNIMORE TO BOSCOBEL RD	1982	Full Depth Patches	sq. yd.	$30.60	7981	7197	Inverted Tee (6" by 6") load transfer.
			Partial Depth Patch	sq. ft.	$16.00	1000	1392	
			Grinding	sq. yd.	$2.50	77500	77460	
	USH151COLUMBUS - BEAVER DAM RD	1982	Full Depth Patches	sq. yd.	$60.00	700	1172	Inverted Tee (6" by 6") load transfer.
			Grinding	sq. yd.	$2.55	97350	89759	
	I-90 MP 130 - 142	1981	Full Depth Full Lane Patch	sq. yd.	$40.00	4074	7866	Dowelled and no load transfer methods used.
			Full Depth Corner Patch	sq. yd.	$42.00	475	800	No load transfer.
			Full Depth Inverted Tee Patch	sq. yd.	$50.00	210	416	
			Grinding	sq. yd.	$2.92	41710	41674	
			Longitudinal Joint Sealing	lin. ft.	$0.70	20785	30156	Hot poured elastic meeting ASTM D3405.
			Transverse Joint Sealing	lin. ft.	$1.25	40722	64941	Low mod. silicone (Dow 888).

Table 8 con't.

STATE	ROUTE LIMITS	YEAR RCHAB.	TECHNIQUE	UNITS	BID PRICE	QUANTITIES PLAN	QUANTITIES FINAL	COMMENTS
MICHIGAN	I-75 MP 64 - 80	1983	Full Depth Patches	sq.yd.	N/A	23532	26481	Dowelled load transfer.
			Pressure Relief Joints	ea.	N/A	0	844	
	M-47 SAGINAW CO. ST.RD TO DIV.	1983	Joint Spall Repair	lin. ft.	$16.00	7750	10918	
			Patching Material	cu. ft.	$42.00	930	1032	
			Sealing Transverse Cracks	lin. ft.	$1.75	10960	16514	Sealtight Sof-Seal (Not poured).
			Replace Neoprene Seals	lin.ft.	$3.10	N/A	20940	
			Exp. Jt. Removal and Reseal	lin. ft.	$4.25	N/A	7512	Sealtight Sof-Seal (Not poured).
SO. DAKOTA	I-29 MP 27 - 62	1972	Partial Depth Patch (Type B)	sq. ft.	$17.50	364	568	
			Partial Depth Spall Repair	sq. ft.	$5.79	24337	105336	
			Joint Sealing	lin. ft.	$1.01	117481	118223	Neoprene
	I-29 MP 0 - 15	1980	Full Depth Repair	sq. yd.	$60.00	N/A	800	No load transfer.
			Partial Depth Patch (Type B)	sq. ft.	$40.00	255	255	
			Partial Depth Patches	sq. ft.	$6.40	127653	109193	
			Joint Sealing	lin. ft.	$2.00	65424	67765	Silicone (Dow 888)
			Pressure Relief Joints	ea.	$800.00	107	106	
	I-90 MP 395.5 - 412	1985	Full Depth Patches	sq. yd.	$60.00	636	1581	Dowelled load transfer.
			Partial Depth Patch (Type A)	sq. ft.	$6.00	11511	89893	Type A patch is over 0.4-foot wide.
			Joint Sealing (Silicone)	lin. ft.	$1.68	77712	74981	No pave't/shoulder jt. sealing set-up in contract.
			Pressure Relief Joints	ea.	$400.00	32	32	
	I-90 MP 265 - 292.2	1987	Partial Depth Patch (Type A)	sq. ft.	$6.07	36806	32547	Type A patch is over 0.4-foot wide.
			Partial Depth Patch (Type B)	sq. ft.	N/A	72	181	Type B patch is 0.4-foot or less wide.
			Joint Sealing	lin. ft.	$1.59	6094	6094	Silicone sealant.
			Pressure Relief Joints	ea.	$695.00	45	45	

CRACK AND SEAT PERFORMANCE

Review Report

Federal Highway Administration

Demonstration Projects Division

and

Pavement Division

April 1987

TABLE OF CONTENTS

A. Executive Summary

Based on the findings of this review, the use of cracking, seating, and overlaying as a pavement rehabilitation alternate should be approached with caution. Since both positive and negative aspects of cracking and seating (C&S) were identified during the review, State agencies contemplating the use of C&S should do a thorough project by project analysis to determine if it is the most cost effective rehabilitation technique to employ.

Of the 22 projects reviewed, only four showed appreciably less reflective cracking in the C&S sections than in the control sections. Observations by the review team, coupled with previous State reports, indicate that there generally is a reduction in the amount of reflective cracks through the overlay during the first few years following construction of a C&S project. However, after 4 to 5 years the C&S sections exhibited approximately the same amount of reflective cracks as the control sections. A significant reduction in reflective cracks occurred on two of the projects reviewed. These projects are located on I-4 in Florida and on SR-99 in California. Both had the following similarities:

1. Constructed on a strong base (cement treated),
2. Small changes in seasonal temperatures, and
3. Non-reinforced pavement.

The main concern with C&S is the reduction of the structural capacity of the pavement. To compensate for the reduction in

structural capacity caused by cracking the pavement, more overlay thickness is required, thus increasing the cost. In addition, study is needed to determine if the delay in reflective cracking actually extends the life of the pavement as opposed to conventional overlays and if so, is it cost effective.

B. Background/Introduction

When portland cement concrete pavement (PCCP) approaches the end of its design life, a decision must be made on what action to take. The most common rehabilitation technique currently used for PCCP is to construct an overlay of asphalt concrete (AC). In time, cracks in the underlying PCCP reflect into the overlay. These cracks are primarily caused by stresses that develop at the bottom of the new overlay directly over the in-place cracks and joints of underlying pavement. These stresses are a result of vertical and horizontal movements of the underlying pavement. Vertical movements are differential movements at the joint/crack in the underlaying pavement and are caused by moving loads. Horizontal movements are due to expansion and contraction caused by temperature and/or moisture changes.

In addition to these changes in the underlying slab, total movement at a crack or joint is affected by slab length and the stiffness of the underlying material. The horizontal movement of cracked slabs under a bonded bituminous surface causes high tensile stresses in the immediate area over the crack. Likewise, vertical movement causes high stresses in the overlay. Because an AC surface is stiffer at lower temperatures, it loses some of its flexible characteristics and can withstand only small temperature-induced stresses.

One method that several States have tried for control of reflective cracking in an overlay is to crack the concrete pavement slab into small segments before overlaying with AC. The intent of pavement cracking and seating is to create pavement sections that are small enough to reduce movement to a point where thermal stresses will be greatly reduced, yet still be large enough to maintain some aggregate interlock between pieces and retain a significant percentage of the original structural strength of the PCC pavement. Seating of the broken slabs after cracking is intended to reestablish support between the subbase and the slab where voids may have existed.

C. Objectives

The objectives of this review were to obtain a better understanding of the expected performance of C&S and overlaying, and to identify the conditions under which this technique has been used in a cost-effective manner. It is hoped that the information obtained from the review will aid States in determining when and how to use C&S as an effective rehabilitation strategy.

D. Selection Criteria

A total of 22 projects in 8 States were reviewed. All of the projects reviewed were of the classic crack and seat method (small hairline cracks, no rupturing of the reinforcing, and no rubblizing of the pavement). The following factors were considered in selecting the projects to be reviewed:

- preferably 3 or more years of service;
- located on a high volume facility;
- historical data accessible;
- overlay thickness of 6 inches or less; and
- a control section.

Using these factors, C&S projects were selected for review in:

- California
- Michigan
- Minnesota
- South Dakota
- Wisconsin

After analyzing the data obtained on projects built in the originally selected States, it was decided to extend the review to include projects in Florida, Tennessee, and Indiana, as well as additional projects in California.

E. Field Survey Results

The general condition of each C&S project reviewed is described in this section.

1. California

   a. I-80 Alameda and Contra Costa Counties

I-80 is an 8-inch undoweled jointed plain concrete pavement (JPCP) on a 4-inch cement treated base (CTB) on 8 inches of select material. The original 6-lane pavement was opened to traffic in the mid-1950's.

In 1982, a rehabilitation project which included C&S with an AC overlay and with edgedrains retrofitted on both the C&S and the control sections was constructed. The pavement was broken into 3- by 4-foot segments with an air operated pile driver and then rolled with a vibratory sheepsfoot roller weighing not less than 12 tons to seat the slabs. The control sections were overlaid with 3 1/4 and 5 inches of AC, but not cracked and seated. The C&S section was overlaid with 5 inches of AC. This was the first C&S project in California, therefore, the bid price of $12.50 per square yard was very high. The current average daily traffic (ADT) is 177,000 with 7.3 percent trucks.

The original pavement was badly cracked and faulted (greater than 1/4 inch). Rocking slabs were reported.

With the exception of two reflective cracks from known rocking slabs, which were intentionally left unseated for evaluation purposes, no other reflective cracks were observed on the project. After nearly 4 years, both the 3 1/4-inch and 5-inch control sections and the C&S sections are performing about the same.

b. I-80 Yolo County

I-80 is a 9-inch undoweled JPCP with a 15-foot joint spacing over a 6-inch dense graded aggregate base (DGAB) over an additional 9-inch aggregate subbase. The original dual-lane facility was constructed in 1942 and two additional lanes were added in 1964.

In 1982, the pavement was C&S and overlaid with 4.8 inches of AC. A CMI hydraulic stamper was used to crack the pavement. The specified crack pattern was a minimum 2- by 2-foot and a maximum of 4- by 4-foot. A vibratory pneumatic tired roller weighing not less than 12 tons was used to seat the pavement. The project also included an uncracked control section with a 4.8 inch AC overlay. The C&S cost was $0.75 per square yard. The current ADT is 20,400 with 22.8 percent trucks.

After 4 years, no reflective cracks were observed. The C&S section and the control section are performing the same.

c. SR-99 Kern County south of Bakersfield

SR-99 is a 9-inch plain jointed, undoweled, PCCP. The pavement is 36 feet wide (three lanes) with AC shoulders. The "inside" two lanes were constructed in 1956 on an asphalt treated base (ATB). The "outside" lane (lane used for comparison purposes) was constructed in 1968 on a CTB. The C&S project, completed in June 1983, was an experimental project with seven 600-foot test sections:

Section	Description
A.	Control - 3.6 inch overlay no fabric
B.	Crack and seat, seated with vibratory sheepsfoot roller, 3.6 inch overlay
C.	Control - 3.6 inch overlay with fabric
D.	Crack and seat, seated with rubber tired roller, 3.6 inch overlay
E.	Crack and seat, seated with a vibratory sheepsfoot roller, 3.6 inch overlay

F.	Crack, not seated, 3.6 inch overlay
G.	Crack and seat, seated with a vibratory sheepsfoot roller, 3.6 inch overlay

The C&S cost was $1.60 per square yard.

(1) In the control section (Section A; no C&S, no fabric), 100 percent of the transverse joints had reflected through the overlay with low severity cracks.

(2) In the other control section (Section C; no C&S, with fabric), approximately 50 percent of the transverse joints had reflected through with low severity cracks.

(3) Sections B, D, E, F, and G all involved C&S and exhibited no reflective cracking.

(4) All of the cracking exhibited (Sections A & C) was in the right lane only. All cracks extended no further than the lane joint with an intersecting short longitudinal reflective crack at the joint, forming a "T." This was probably due to the different pavement age and base type.

(5) Deflection testing indicated generally higher deflections after the seating operation than just after cracking. A 13-ton roller was used with 10 passes.

In summary, after 3 years the C&S sections were exhibiting no reflective cracks and were outperforming both of the control sections.

d. Others

A number of other C&S projects were reviewed. Because there was not a true control section for comparison purposes and there was no distress evidenced on either the C&S or the normal overlay portions, these projects are summarized in one discussion.

Route	County	ADT (% Trucks)	Built	Total Overlay Thickness	C&S Cost Per Sq. Yard
I-5	Shasta	25,600 (23)	6/83	5.4 inches	0.75
I-580	Alemeda	56,000 (16)	3/84	4.2 inches	0.80
I-680	Contra Costa	152,000 (4.9)	11/83	4.8 inches	0.55
I-680	Contra Costa	157,000 (4.6)	10/83	3.4 inches	0.85
I-680	Contra Costa	69,000 (6.7)	11/83	4.2 inches	0.60

The projects consisted of 8-inch JPCP on 4-inches of CTB. All of these projects used a fabric interlayer between AC overlay courses and used the same specifications for C&S calling for 4- by 6-foot cracking pattern. These projects only called for C&S in the outer lane(s).

2. Michigan

a. US-10 in Clare County

The original pavement opened to traffic in the mid-1930's was a widened edge (9"-7"-9") jointed reinforced concrete pavement (JRCP). Joints were undoweled with a 60-foot spacing. The original PCCP was overlaid with approximately 4 inches of AC in 1960.

The 8-mile rehabilitation project, completed in October 1983, consisted of milling off the existing bituminous overlay, C&S the pavement, and overlaying with approximately 2 1/4 inches of AC. The pavement was cracked into 18- by 18-inch pieces and seated with a 50-ton vibratory steel wheel roller. The type of breaker was not specified. The C&S cost was $0.20 per square yard. Longitudinal edgedrains were added in select locations. A control section was not built. The current ADT is 1410 with an average of about 120 ESAL's/day since the rehabilitation.

(1) Nearly all transverse joints had reflected through the 2 1/4-inch overlay. The reflective cracks are primarily medium in severity. In addition, intermediate transverse cracks have also reflected.

(2) Less than 5 percent of the longitudinal lane joint has reflected through.

(3) Some minor rutting (1/4 inch) of the asphalt surface is evident.

(4) The ride quality on this project was very good.

b. US-23 in Monroe County

This was an experimental C&S project of approximately 1 1/4 miles within an overall 8-mile long overlay project. US-23 is a 4-lane freeway section with an original 9-inch JRCP with 99-foot doweled joint spacing.

The C&S experimental project, completed in 1983, consisted of 24-, 36-, and 48-inch cracking patterns plus control sections (no cracking), and two overlay thicknesses of 440 and 660 pounds per square yard (approximately 4 and 6 inches). A whip hammer was used to crack the pavement and a 50-ton rubber- tired roller was used to seat the pavement. The C&S cost was $0.19 per square yard. The current ADT is 11,350 with a daily loading of about 3,800 ESAL's per day.

(1) In all four of the comparisons (three different crack patterns and control section) the 660 pounds per square yard overlay (6 inches) had less reflective cracking than the 440 pounds per square yard (4 inches) overlay.

(2) Generally, the least amount of reflective cracking within the C&S sections occurred in the section with the 48-inch crack pattern.

(3) The test section with the least cracking (best condition) was the 660 pounds per square yard control section (no C&S) followed closely by the section with 660 pounds per square yard and the 48-inch crack pattern.

(4) The project showed no signs of distress, other than low severity reflective cracks.

3. Minnesota

a. T.H. 169, Scott County

This project is on T.H. 169 from 0.55 miles south of Belle Plaine's city limits to County Road 66. The original project was constructed in 1956 and consisted of a widened edge (9"-7"-9") non-reinforced PCCP. The joints were undowled with 20-foot spacing.

The rehabilitation project, completed in 1982, consisted of three 1,000-foot sections. One section had a 3-foot crack spacing with no crack closer than 5 feet from a joint or existing transverse crack, one section was cracked at 1 1/2-foot intervals, and the other section was not cracked. A spade type breaker was used to crack the pavement. A 30-ton rubber-tired roller was used to seat the pavement.

The three sections were overlaid with 5 3/4 inches of AC. The C&S cost was $50 per road station ($0.18 per square yard). The current ADT is 10,627.

The section with 3-foot crack spacing was exhibiting random reflective cracks at the joint and minor raveling. The section with the 1 1/2-foot crack pattern and the control section had low severity reflective cracks.

b. T.H. 60 and T.H. 169, Blue Earth County

This project is on T.H. 60 and T.H. 169 near the city of Mankato. The original project was constructed in 1961 and consisted of an 8-inch reinforced PCC pavement over 5 to 9 inches of aggregate base. The pavement was 25 feet wide and the joints were doweled with a 40-foot spacing.

The rehabilitation project, completed in 1982, consisted of eight 1,000-foot test sections. Test sections 1, 2, 5, and 6 were cracked with a spade type breaker. Test section 8 was cracked with a roller breaker. All the sections were seated with a 30-ton pneumatic-type roller. Each section was overlaid with a 6-1/4 inches of AC. The C&S cost was $55 per road station ($0.21 per sq. yd.) The current ADT is 8,454. A summary of the test sections follow:

Test Section	Rehabilitation
Section 1	3-foot crack spacing and edgedrains
Section 2	3-foot crack spacing, no edgedrains
Section 3	No cracking, no edgedrains
Section 4	No cracking, edgedrains
Section 5	1.5-foot crack spacing, edgedrains
Section 6	3-foot crack spacing, no edgedrains,
Section 7	Edgedrains, saw cut construction
Section 8	Edgedrains, cracked with pavement roller breaker

To date there has been very little difference in the performance of the test sections. Each section exhibited reflective cracks approximately every 40 feet (at each joint).

c. T.H. 71, Kandiyohi County

This project was the first C&S project in Minnesota and was completed in 1976. The original roadway structure was a widened edge (9"-7"-9") non-reinforced concrete pavement 22 feet wide with a continuous longitudinal centerline joint and undoweled transverse joints constructed every 15 feet. The surface had spalled at some of the joint locations and maintenance crews had patched these areas with bituminous mixture.

The rehabilitation called for a 6-inch thick AC overlay with the thickness being increased to 7 1/2 inches at some locations. The in-place PCC panels were cracked with a vehicle-mounted spade type breaker at the mid and quarter points thereby reducing the size of the PCCP to pieces about 3 3/4 by 11 feet. A control section of uncracked in-place PCCP with a 7 1/2-inch overlay was constructed to use as a comparison to the broken section. The overlay consisted of 3/4-inch plant-mixed AC wearing course, 1 1/2-inch plant-mixed AC binder course, and either 3 3/4 or 5 1/4 inches of plant-mixed AC base course depending on the location of the overlay. The C&S cost was $70 per road station ($0.26 per square yard). The current ADT is 3,974.

The 1981 final report by the Minnesota Department of Transportation(1) states, "the cracking of the in-place PCCP did reduce the amount of reflective cracking in comparison to similar sections where the PCCP was not cracked."

However, during our review, there were reflective cracks throughout the project. Thus, it appears that C&S did delay reflective cracks for the first 5 years, but after 10 years there was little or no difference in the performance of the C&S section and the control section.

4. Wisconsin

a. I-94, Eau Claire County

The original pavement, constructed in 1967, consisted of 9 inches of reinforced concrete with a 6-inch aggregate base and a 12-inch granular subbase. The joints were doweled with 80-foot spacing.

The rehabilitation project was completed in 1983. A pile drive hammer was used to crack the pavement with a maximun pattern of 18 inches. A 50-ton vibratory roller was used to seat the cracked pavement. The C&S cost was $0.30 per square yard. The current ADT is 16,000. The project consisted of the following:

Section	Overlay Thickness	Performance
Control	4 inches	Reflective cracks every 80 feet, some edgeline cracks
C&S #1	5½ inches	Random centerline reflective cracks
C&S #2	7 inches	Very few small reflective cracks
C&S #3	4 inches	Random edgeline and centerline reflective cracks

The C&S sections with the 5 1/2-inch and the 7-inch overlays were performing slightly better than the C&S section with the 4-inch overlay and the control section.

b. USH 14, Dane and Rock Counties

This was the first C&S project in Wisconsin and was completed in 1980. The original 9-inch non-reinforced PCCP pavement on a 9-inch aggregate base was constructed in 1952. The joints were undowled with 20-foot spacing.

The rehabilitation project, completed in 1980, was 6 miles in length. The pavement was cracked with hydro-hammer type breaker into pieces not exceeding 1 square yard in area. The cracked pavement was then rolled with a 50-ton pneumatic roller and overlaid with 4 1/2 inches of AC. The control section was not cracked and had a 4 1/2 inch AC overlay. The C&S cost was $0.45 per square yard. The current ADT is 4,000.

There were reflective cracks throughout the project and there was no difference in the performance of the C&S section and the control section.

c. STH 140, Rock County

The original project, a 9-inch non-reinforced PCCP with a 9-inch aggregate base, was constructed in 1931. The joints were undoweled with 20-foot spacing.

The C&S project, completed in 1982, required the pavement to be broken into pieces having a maximum dimension of 12 inches with a pile drive hammer and seated with a 50-ton vibratory roller. The control section and the C&S section were each overlaid with 4 inches of AC. The C&S cost was $0.35 per square yard. The current ADT is 2,000.

There were reflective cracks throughout each section with no difference noted in the performance.

5. South Dakota

a. US Route 18, Lincoln County

The original project consisted of mesh reinforced PCCP that was a widened edge (9"-6"-9") section, 20 feet wide, with a 6-inch aggregate base. The joints were undoweled with 20-foot spacing. The original construction was completed in 1930.

This rehabilititation project was completed in 1982. A total of 3.89 miles east and west of Canton was C&S and the 2-mile section through the town of Canton was just overlaid. A spade type breaker was used to crack the pavement at 5 foot intervals and a vibratory steel wheeled roller was used to seat the cracked pavement. The C&S section was overlaid with 3 1/2 inches while the non-C&S section had a 2-inch AC overlay. The C&S cost for this project was $4,000 per mile ($0.20 per square yard). The current ADT is 3,466 with 8.8 percent trucks.

There were reflective cracks about every 40 feet throughout the project. However, there were a few more cracks in the non-C&S section which is expected since it received 1 1/2 inches less AC.

b. US Route 50, Clay and Union Counties

The original project consisted of a mesh reinforced PCCP with a widened edge (9"-6"-9") section on a 6-inch aggregate base that was 20 feet in width. The original construction was completed in 1938. The joints were not doweled.

The C&S project was completed in 1980. It consisted of breaking the 40-foot panels at the quarter points with a spade type breaker, seating the pavement with a vibratory steel wheeled roller, and overlaying with a total of 4 1/2 inches of AC. There was no control section on this project. The C&S cost was $4,000 per mile ($0.20 per square yard). The current ADT is 1,492 with 8.8 percent trucks.

Approximately 90 percent of the project had centerline cracks. There were also random transverse and longitudinal cracks throughout the project.

c. US Route 14, Beadle County

The original construction consisted of a 22-foot wide, 8-inch thick mesh reinforced PCCP on a 6-inch aggregate base that was constructed in 1947. The panels were 15 feet long and the joints were not doweled.

The C&S project was completed in 1979. The 15 foot panels were cracked at 5-foot intervals with a hydro-hammer. A loaded scraper was used to seat the cracked pavement. A 500-foot section of the pavement was left uncracked to serve as the control section. The C&S and the control sections were overlaid with 4 1/2 inches of AC. The cost of C&S on this project was $3,258.90 per mile ($0.23 per square yard). The current ADT is 2,122 with 13.4 percent trucks.

There were random cracks observed at the joints throughout the project with little or no difference noted between the control and the C&S sections.

6. Florida

a. I-4, Hillsbourough County

The original pavement was a 9-inch plain jointed undoweled (except near expansion joints) PCCP with a 20-foot joint spacing on 12-inch cement stabilized base.

The rehabilitation project was completed in 1979. Four test sections were set up to evaluate C&S and two types of fabric to reduce reflective cracking. A drop hammer was used to crack the pavement into 36-inch maximum size pieces. Vibratory compacting equipment or traffic rollers weighing at least 15 tons were specified as equipment to seat the cracked pavement. All sections were overlaid with a 100 pound per square yard (approximately 1 inch) AC leveling course, 2 inches of AC binder, and a 5/8-inch open graded friction course. All sections also received underdrains.

The following is a breakdown of the performance of each section made by the Florida Department of Transportation in March 1986.

Section	Description	Percent Reflected Joints: Rt. Edge	Lt. Edge	Center Longitudinal	Transverse
A	Control with underseal No Fabric	100	50	0	94
B	Crack and Seat No Fabric	87	10	0	22
C	Control with underseal and fabric	100	80	35	72
D	Control with underseal and fabric	80	36	35	35

7. Indiana

I-74 Montgomery/Boone County, a length of 12.4 miles.

The original pavement was a 10-foot reinforced (welded wire) and doweled PCCP on about 6 inches of open graded aggregate subbase. Contraction joints were spaced at 40-foot intervals. Longitudinal edgedrains were provided in the original construction. The pavement was very deteriorated prior to the rehabilitation with 100 percent of the slabs having intermittent cracking at a rate of about 45 feet of cracking per 100 square feet of pavement and about 22 breakups per 100 square feet. Every joint was "D" cracked.

This rehabilitation project was completed in 1984 and consisted of the following sections:

Sections	Description
A.	Asphalt underseal with 4 1/4 inch asphalt overlay
A.1	Same as A with fiber reinforced asphalt base layer
A.2	Same as A with fiber reinforced asphalt base and binder layers
B.	Cracked and seated with 5 1/2 inch AC overlay
B.1	Same as B with fiber reinforced asphalt base layer
B.2	Same as B with fiber reinforced base and binder layers
C.	Cracked and seated with 6 1/2 inch AC overlay
D.	Cracked and seated with 8 1/2 inch AC overlay

The C&S sections used two types of pavement breakers, a whip hammer and a drop hammer. The cracks were required to be mainly transverse, spaced 18 to 24 inches apart. A 50-ton rubber-tired roller was used to seat the pavement. The C&S cost was $0.64 per square yard.

Since the overlay thickness of the "control" does not match the C&S, a direct comparison is not possible. The performance results of the 5-inch overlay in the C&S section are compared below with the 4 1/4 inch "control" overlay.

a. There were no reflective cracks in the 6 1/2- and 8 1/2-inch overlaid C&S pavements. (sections C&D)

b. Only a couple of reflective cracks were observed in the 5-inch overlaid pavements (sections B, B1, B2) which amounted to about 1 percent of the joints.

c. About 40 percent of the transverse joints in the 4 1/4-inch "control" pavements (sections A, A1, A2) had reflected through.

d. All cracks observed were medium in severity and followed a "jagged line pattern" across the pavement at the joint.

e. There were isolated "blow-up" areas in both the control and C&S sections.

f. There was one area about 1/2-mile long of the 4 1/4-inch overlay control sections that showed no reflective cracking. The lack of reflective cracking in this one area could not be readily explained and is not indicative of the "control" sections in the project.

The 1986 Initial Construction and Interim Performance Report from the Indiana Department of Highways(2) concludes in part... "the drop hammer was the most effective machine for producing regular transverse cracks in the pavement. Cracking reduced the strength of the concrete slab without decreasing the subbase support. Rolling with the 50-ton roller decreased both the slab strength and subbase support. Therefore, a heavy roller should not be used as it does not seat the pavement, but rather unseats it."

8. Tennessee

SR-5 Bypass, Madison County

The existing pavement was a 9-inch PCCP on a 6-inch CTB, with no dowels and a 25-foot joint spacing.

The C&S with overlay was completed in November 1983. It consisted of cracking the slab from 18- to 24-inch pieces, seating with a 50-ton pneumatic-tired roller, and overlaying with 5 3/4 inches of AC. The control section had undersealing with fly ash/cement grout, full-depth joint repair, joint resealing, and a 5 3/4-inch overlay. The existing pavement was in fair condition with less than 5 percent of the slabs with cracks. The C&S cost was $0.40 per square yard.

a. About 20 percent of the transverse joints had reflected through the control section overlay with primarily low severity cracking.

b. About 3 percent had reflected through in the C&S section.

c. There were a few isolated locations where longitudinal cracking appeared in the wheel paths of the C&S section.

F. DISCUSSION

1. Of the 22 projects reviewed, only four projects showed appreciably less reflective cracking in the C&S sections than in the control sections. To quantify the benefits of C&S, a measure of the difference in the percent of transverse joints which had reflected through the overlay was employed. Observations made during this review coupled with previous State condition surveys, where available, indicated a reduction in the percent transverse joints reflecting through the overlay during the first few years when C&S is applied. However, after 4 to 5 years the C&S sections generally have approximately the same cracking as the control sections. Therefore, it can be concluded that overall, C&S appears to

provide benefits under some conditions by delaying, not eliminating, reflective cracking.

2. The two projects where the C&S sections performed best were:
   a. SR-99 near Bakersfield, California
   b. I-4 near Tampa, Florida

   Because of the notable difference in the percent of transverse joints reflecting through between the C&S and the control sections on these projects, similarities were investigated. It is believed that the following combination of conditions had the greatest impact on the success of these two projects.

   a. Strong base (cement-treated)
   b. Small changes in seasonal temperatures
   c. Non-reinforced pavement

   These similarities tend to indicate that C&S works best under the same limited conditions as other methods used to reduce reflective cracking (pavements that tend to have little vertical and horizontal movement). Small changes in seasonal temperatures understandably result in less thermal movement in the slab, thereby reducing tensile stress in the AC overlay and the possibility of reflective cracking. A strong base should help in reducing the vertical shear stresses in the overlay. With non-reinforced pavements, the aggregate interlock of the crack interface is the controlling factor in resisting differential deflection or vertical movement. A strong base helps maintain uniform support and should minimize differential deflections of the individual pavement pieces.

In addition, non-reinforced pavements should provide better performance since the presence of reinforcing steel in a slab will tend to inhibit the development of cracks which penetrate all of the way through the slab. Even when the pavement is cracked full depth the steel will tie the sections together concentrating the thermal movement at the original joints which should result in reflective cracking. Non-reinforced pavements generally have shorter slab lengths than reinforced pavements. This reduces the thermal movement at the joints and, therefore, should reduce reflective cracking.

3. The reduction of the structural capacity of the existing pavement appears to be an undesirable feature of C&S. The size of the cracked sections have a direct relationship to structural capacity.

The 1986 AASHTO Guide for Design of Pavement Structures includes a methodology for overlay of C&S pavements. Using this methodology, the suggested structural layer coefficients (indication of carrying capacity per inch of pavement) of the C&S pavement are as follows:

Crack Spacing	Structure Layer Coefficient
1 Foot	0.25
2 Feet	0.35
3 Feet	0.45

A research report(5), completed for the National Asphalt Pavement Association, concluded through back calculation of deflection testing performed on Minnesota's C&S projects that the structural layer coefficients for the C&S project test sections ranged from 0.21 to 0.53. The crack spacing and degree of cracking appeared to be related to the structural layer coefficients. This tends to support and verify the values used in the AASHTO Guide.

Since the structural capacity of the existing pavement is reduced by cracking, more overlay thickness is required to maintain the same structural number as the non-cracked pavement. Using an overlay analysis such as AASHTO would typically result in the need for up to 3 inches to maintain equivalent structural capacity.

The additional cost of: 1) the additional overlay thickness; 2) the cracking and seating; and 3) other required work such as shoulder and guardrail raising, must be evaluated to determine if these costs are justified.

Based on this review and the limited field performance data available to date, it appears these extra costs may not be justified since the condition of the C&S and control sections seemed to be the same after some period of time on most of projects reviewed.

One project where this type of comparison is possible is on US 23 in Michigan. This project had two overlay thicknesses, 440 pounds per square yard and 660 pounds per square yard (approximately 4 and 6 inches) on both the C&S and the control. The extra 2 inches alone has given added performance life because the amount of reflective cracking is much less for the thicker overlay. The C&S with the thicker overlay is performing no better than its control section which indicates no benefit can be seen at this point.

Other C&S projects where various overlay thicknesses were constructed are:

Wisconsin I-94:

	Section	Overlay Thickness
a.	Control	4 inches
b.	C&S #1	5 1/2 inches
c.	C&S #2	7 inches
d.	C&S #3	4 inches

During the review, 3 years after construction, it was observed that the sections b. (C&S-5½") and c. (C&S-7") were performing slightly better than sections d. (C&S-4") and a.(Control-4").

Indiana I-74:

	Sections	Overlay Thickness
a.	Control	4 inches
b.	C&S	5 inches
c.	C&S	6 1/2 inches
d.	C&S	8 inches

At the time of the review, 2 years after construction, there were no reflective cracks in Sections c and d indicating more time is bought by the additional AC thickness.

4. Very little deflection testing has been performed on C&S projects. Only two of the projects reviewed had completed research in this area. The following is a general description of the results of that research.

<u>Indiana, I-74</u>: A Dynaflect was used to measure deflections. Deflection measurements were made before cracking, immediately after cracking, and after the seating operation. The effectiveness of the seating operation was tested after three passes of a 50-ton rubber tired roller as required in the specifications. Test data was also obtained on seven subsections after a variable number of passes of the roller. The average increase in deflection per pass of the seating roller was:

2.3×10^{-5} inch/pass for No. 1 sensor

0.8×10^{-5} inch/pass for No. 5 sensor

Since the deflection increased with each pass of the roller for both sensors, the concrete slab and the subbase lost strength with each pass. The research report states "... the heavy roller caused the slab pieces to unseat rather than to seat as was originally intended. This means that the heavy roller should not be used to attempt to seat the cracked slab pieces."

California, SR-99: Deflection testing was done with the Benkleman Beam and an 18 kip axle load. Deflection measurements were taken before C&S, after cracking, and after seating. The results of the testing are summarized below.(4) Rolling was performed with a 13-ton roller.

After Breaking/Before Seating

Change in Deflection	Number of Joints	Amounts
Reduced	36 of 39 (92%)	Average = 0.006 inches
Increased	1 of 39 (3%)	Average = 0.001 inches
Unchanged	2 of 39 (5%)	-----

After Seating

Change in Deflection	Number of Joints	Amounts
Reduced	9 of 35 (26%)	Average = 0.001 inches
Increased	14 of 35 (40%)	Average = 0.001 inches
Unchanged	12 of 35 (34%)	-----

The results of these two projects cast doubt on the need for seating after cracking. More research is needed in this area.

5. A review of the two projects where a direct comparison of the cracking pattern is possible, Michigan U.S. 23 and Minnesota 60/169, reveals that the larger crack spacing generally performed better than the smaller crack spacing. This would

be expected since for the same overlay thickness, the larger crack spacing is structurally superior to the smaller crack spacing.

Figure 1 shows the results of specific research by Michigan and Minnesota which compared performance of different cracking patterns. In both cases, the larger crack patterns performed better than the smaller crack patterns. Line "0" on Figure 1 is the performance of the control section. Any value above "0" indicates better performance and values below "0" means worse performance.

CRACK PATTERN COMPARISON

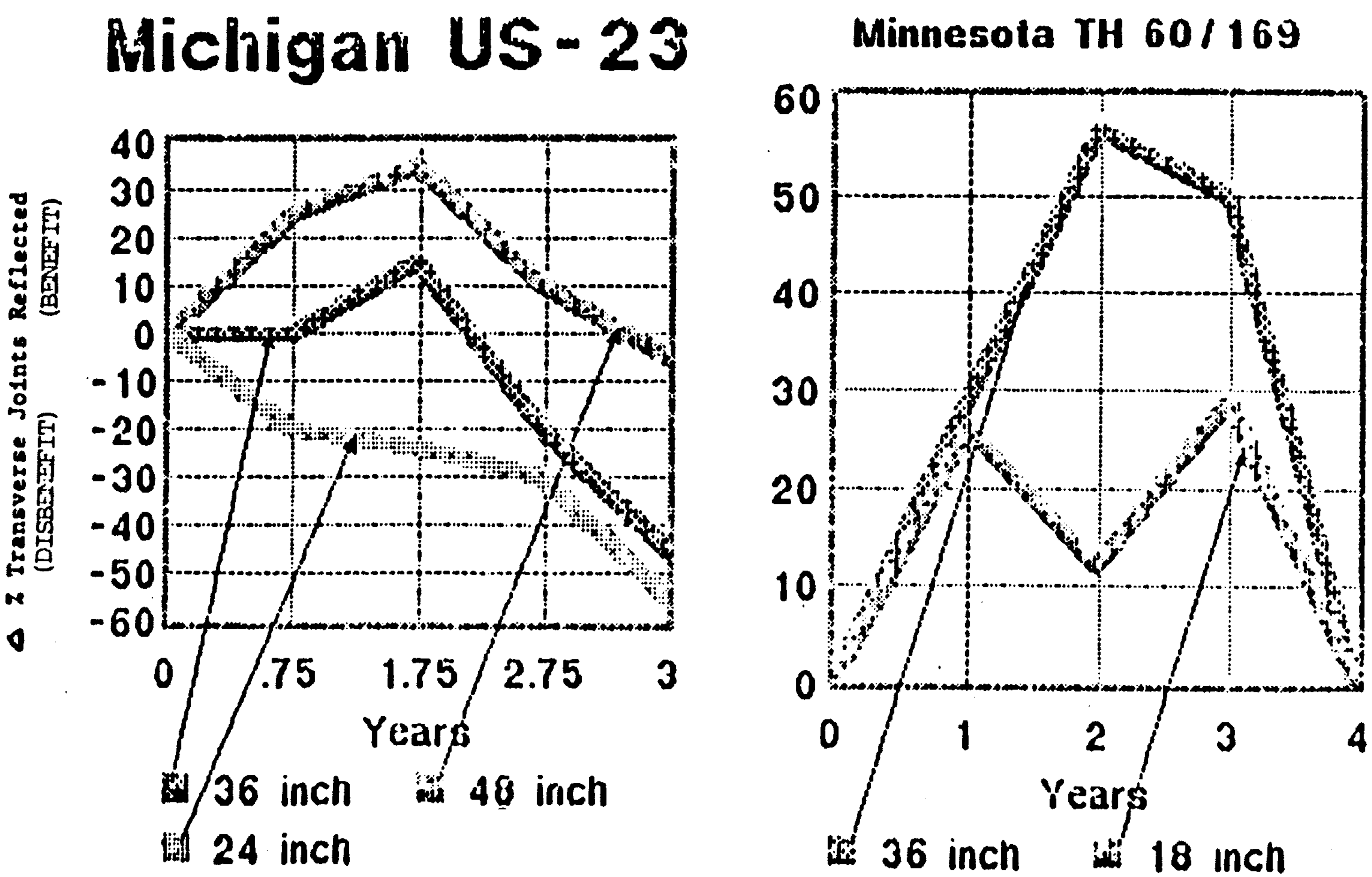


Figure 1

STATE	PROJECT	REINFORCE	DOWELS	JOINT SPACING	PAVEMENT THICKNESS	BASE TYPE	BREAKER TYPE	BREAK PATTERN	ROLLER	ROLLER WEIGHT	OVERLAY (INCHES)	YEARS	COST	REMARKS
FLORIDA	I-4	NO	NO	20	9"	CTB	DROP HAMMER	36" MAX	OPTIONAL	15T	3 5/8	7	-	C&S PERFORMING BEST
CALIFORNIA	SR-99	NO	NO	-	9"	CTB	DROP HAMMER	4' X 6'	VIB & RUBBER	13T	5	3½	1.60	NO CRACKS IN C & S
CALIFORNIA (ALEMEDIA & CONSTRA COSTA)	I-80	NO	NO	-	8	CTB	PILEDRIVE HAMMER	3' X 4'	VIBRATORY	12T	3½, 5	4	12.50	NO DIFFERENCE IN PERFORMANCE
CALIFORNIA (YOLO COUNTY)	I-80	NO	NO	15	9	AC	HYDRAULIC STAMPER	2' X 2'	VIBRATORY	12T	4.8	4	0.75	NO DIFFERENCE
TENNESSE	SR-5	NO	NO	25	9"	CTB	PILE DRIVER	18"-24"	RUBBER	50T	5 3/4	3	0.40	C&S SLIGHTLY BETTER
INDIANA	I-74	YES	YES	40	10	AG	WHIP & DROP HAMMER	18"-24"	RUBBER	50T	5	2	0.64	C&S MARGINALLY BETTER
SOUTH DAKOTA	U.S. 18	YES	NO	20	9-6-9	AG	SPADE	5 FEET	VIBRATORY	-	3½	4	0.20	3½" C&S SLIGHTLY BETTER THAN 2" CONTROL
SOUTH DAKOTA	U.S. 50	YES	NO	40	9-6-9	AG	SPADE	10 FEET	VIBRATORY	-	4½	6	0.20	NO CONTROL SECTION
SOUTH DAKOTA	U.S. 14	YES	NO	15	8	AG	HYDRO-HAMMER	5 FEET	LOADED SCRAPER	-	4½	7	0.23	NO DIFFERENCE BETWEEN C&S AND CONTROL
MINNESOTA	TH 169	NO	NO	20	9-7-9	AG	SPADE	18", 36"	RUBBER	30T	5 3/4	4	0.18	LOW SEVERITY CRACKS IN EACH SECTION
MINNESOTA	TH 60/169	YES	YES	40	8"	AG	SPADE	18", 36"	PNEUMATIC	30T	6½	4	0.21	NO DIFFERENCE IN PERFORMANCE
MINNESOTA	TH 71	NO	NO	15	9-7-9	-	DROP HAMMER	3 3/4'	RUBBER	-	6, 7½	10	0.26	AFTER 10 YEARS PERFORMANCE THE SAME
WISCONSIN	I-94	YES	YES	80	9	AG	PILEDRIVE HAMMER	18"	VIBRATORY	50T	4,5½,7	3	0.30	THICKER OVERLAYS PERFORMING SLIGHTLY BETTER
WISCONSIN	USH14	NO	NO	20	9	AG	HYDRO-HAMMER	"36 X 36"	PNEUMATIC	30T	4½	6	0.45	NO DIFFERENCE IN PERFORMANCE
WISCONSIN	STH140	NO	NO	20	9	AG	PILEDRIVE HAMMER	12"	VIBRATORY	50T	4	4	0.35	PERFORMANCE IS THE SAME
MICHIGAN	U.S.10	YES	NO	60	9-7-9	-	NOT SPECIFIED	"18 X 18"	VIBRATORY	50T	2½	3	0.20	NO CONTROL SECTION REFLECTIVE CRACKS IN C&S
MICHIGAN	U.S.23	YES	YES	99	9	SELECT MATERIAL	WHIPHAMMER	24, "36", "48"	RUBBER	50T	4,6	3	0.19	6" C&S & CONTROL PERFORMING THE SAME

G. References

1. Minnesota Department of Transportation, Research and Development, Crack Reflectançe on Bituminous Overlaid PCC Pavement (August 1981).

2. Indiana Department of Highways, Division of Research and Training, Initial Construction and Interim Performance Report (September 1986).

3. State of Florida Department of Transportation, Memorandum Inspection of Asphalt Over Concrete, Test Section Located on I-4 in Hillsboro County, (March 1986).

4. California Department of Transportation, Memorandum, Report of Construction, "Effects of Slab Breaking and Seating on Differential Vertical Movement at PCC Slab Joints and Cracks."

5. Midwest Pavement Management, Inc., Structural Evaluation of Crack and Seat Overlay Pavements, (in Minnesota) (September 1986).

Memorandum

U.S. Department of Transportation

Federal Highway Administration

Washington, D.C. 20590

Subject: Technical Paper - Saw and Seal Pavement Rehabilitation Technique

Date: FEB - 2 1988

From: Chief, Pavement Division

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Division Administrators
(Pavement Specialists)

The purpose of the attached technical paper is to provide practicing pavement technologists with a brief summary of experience on the technique of saw and seal.

The sawing and sealing technique involves the marking of the existing transverse joints of a PCC pavement on the surface of the AC overlay. Next, a saw is used to cut a joint into the asphalt surface, directly over the existing transverse joint. This produces a straight, neat joint in the overlay, which establishes a stress relief plane. The joint is then sealed and maintained as a normal pavement joint.

Currently underway is a FHWA Research and Development Administrative contract, "Performance/Rehabilitation of Rigid Pavements." The contractor has recently completed the field survey to evaluate performance of saw and seal projects. These projects range in age from 3 to 10 years old and are located in 6 States. The preliminary review of the data gathered during the field survey indicates that projects incorporating the saw and seal technique out perform those not using the technique. The research contractor is now entering the field survey data into a data base for further analysis. A more comprehensive report, including a chapter for the Pavement Rehabilitation Manual is expected at the end of the year.

Because of the good performance observed from the saw and seal technique, we are providing the attached technical paper as interim state of the practice information. We can anticipate minor modifications to the current procedures upon completion of detailed analysis of the research data and further experimental usage.

We recommend that you and your staff promote the further usage of this technique during the forthcoming construction season. The Pavement and Demonstration Projects Divisions would welcome any data as it becomes available. If needed, some limited funding is available, on a first come first serve basis, for the evaluation and reporting on the performance of recently completed or planned saw and seal projects under Experimental Project No. 9, Pavement Rehabilitation Techniques.

Norman J. Van Ness

Norman J. Van Ness

TECHNICAL PAPER 88-01 — Saw and Seal Pavement Rehabilitation Technique

I. BACKGROUND

For the purposes of this technical paper, reflection cracking can be defined as fractures in an asphalt concrete overlay that are the result of, and reflect, the joint pattern in the underlying Portland Cement Concrete pavement, and may be either environmental or traffic induced.

The basic mechanisms generally assumed to lead to reflection cracking are the vertical and horizontal movements of the pavement being resurfaced. Vertical movements are differential movements at the joint in the underlying pavement and are caused by moving loads; horizontal movements are due to expansion and contraction caused by temperature change and/or moisture change. The horizontal movement of slabs under an asphalt overlay causes high tensile stresses in the immediate area over the joint. Particularly during lower temperature, the AC surface stiffens and can withstand only small temperature-induced stresses. In addition to temperature changes in the underlying slab, total movement at the joint is affected by slab length, moisture changes, friction or bonding to the base, and the stiffness properties of the overlying material.

The problem of reflective cracking is one of the most perplexing facing the pavement engineer. There do not appear to be any treatments which can prevent the eventual reflection of existing cracks. One treatment, the sawing and sealing of joints in the overlay above existing joints and cracks, has been demonstrated to effectively control severity and extend the service life of the overlay.

The sawing and sealing technique involves the marking of existing transverse joints on the surface of the overlay. Next, a saw is used to cut a joint into the asphalt surface, directly over the existing transverse joint. The joint should be continued through the shoulders, from outside edge to outside edge. This produces a straight, neat joint in the overlay, which establishes a stress relief plane. The joint is then sealed and maintained as a normal pavement joint.

II. DETAILS OF THE TECHNIQUE

Accurately locating joint -

The most critical step in sawing and sealing the overlay is the process of locating the transverse joints in the existing pavement. Experience has shown that as little as one inch deviation from the existing joint location can cause the joint to reflect through the overlay at its location rather than at the sawed joint. Therefore, extreme care must be taken in locating the existing transverse joints.

sealing operation should follow the sawing. They found that at moderate temperatures the joints did not close in four days, but at higher temperatures, the shoving by traffic did close up the joints.

A practical recommendation would be that the overlay should be sawed before any occurrences of sub-freezing temperatures and that the sealing take place as soon as possible or at least before traffic is allowed on the overlay.

Pre overlay treatments -

The effectiveness of sawing and sealing depends greatly on the condition of the underlying pavement. To obtain the full benefit, only concrete pavements with relatively good joints and no surface deterioration should be selected. Joints wider than 3 inches make it difficult to control reflective cracks. Concrete pavements with numerous full-depth and surface patches, misaligned slabs, and midslab cracking are not candidates for this technique. Consequently, there should be a minimum of pre-overlay treatments.

New York specifications include a requirement that if a full depth patch is wider than 0.5 feet, then an additional saw cut shall be made at the patch interface.

There have been recent applications of saw and seal technology on projects requiring significant joint repair. In one instance, the joints were D-cracked. Consequently, the D-cracked material was milled out 2 inches deep over the joints and backfilled with AC prior to the overlay.

In another instance, the joints were spalled. Again the joints were milled 3 inches deep and backfilled with AC. Both of these installations are relatively new and no significant performance data is yet available.

III. APPLICATIONS AND LIMITATIONS

Jointed reinforced PCC pavements -

All of the saw and seal projects have been on jointed reinforced PCC pavements, with relatively long joint spacing. This raises the question as to the cost effectiveness of the saw and seal technique on plain jointed PCC pavement with a lesser joint spacing.

Northeastern States -

Most of the States using saw and seal are located in the northern tier of the country where there is a potential for sizable temperature related slab movement. Connecticut, Massachusetts, New Jersey, New York, and Pennsylvania have the most experience with the saw and seal technique.

New Jersey - This test section was constructed on US 22 in 1977. The original pavement was a 9 inch jointed PCC. It was overlaid with 2 inches of AC. The joints were sawed 3/8 inch wide and 1/2 inch deep and sealed with rubberized joint sealer.

New Jersey - This test section was constructed on I-80 in 1977. The original pavement was a 9 inch jointed PCC with 78 foot joint spacing. It was overlaid with a 2 inch AC overlay. The joints were sawed 3/8 inch wide by 5/8 inch deep and sealed with hot poured elastic (ASTM D 1190).

New York - This test section was constructed on State Route 5 in 1980. The original pavement was a 9 inch jointed PCC with 90 foot joint spacing. It was overlaid with 2 1/2 inches AC. The joints were sawed 1/2 inch wide by 5/8 inch deep and sealed with hot poured sealant (ASTM D 3405).

New York - This test section was constructed on I-81 in 1984. The original pavement was a 9 inch jointed PCC with 63 foot joint spacing. It was overlaid with 3 1/2 inches of AC and sawed at the joints 1/2 inch wide and 5/8 inch deep and sealed with hot poured sealant (ASTM D 3405).

New York - This test section was constructed on I-87 in 1984. The original pavement was a 9 inch jointed PCC with 60 foot joint spacing. It was overlaid with 4 1/2 inches of AC and sawed at the joints 5/8 inches wide and 5/8 inches deep and sealed with hot poured sealant (ASTM D 3405).

Connecticut - This test section was constructed on I-91 in 1978. It was overlaid with 2 3/4 inches of AC and sawed at the joints 3/8 inch wide and 1/2 inch deep and sealed with hot poured elastic sealant (AASHTO T 187).

Connecticut - This test section was constructed on I-84 in 1982. It was overlaid with 3 inches of AC and the joints were sawed 3/8 inch wide and 1/2 inch deep and sealed with hot poured elastic sealant (AASHTO T 187).

Indiana - This test section was constructed on I-80 in 1986. It was overlaid with 5 1/2 inches of AC. The joints were sawed 1/8 inch wide and 2 inches deep. This was followed by routing 1/2 inch wide by 1 inch deep and sealed with a single component hot poured elastomeric polymer.

V. SAMPLE SPECIFICATIONS

Sample specifications from New York and Indiana are provided as examples of comprehensive specifications on saw and seal projects.

VI. COST DATA

There is limited cost data available, however, reported costs range between $1 and $4 a linear foot for the sawing and sealing technique.

TO:	ENGINEERING INSTRUCTION NEW YORK STATE DEPARTMENT OF TRANSPORTATION
	SUBJECT: SAWING AND SEALING JOINTS IN BITUMINOUS CONCRETE OVERLAYS Subject Code:

Distribution: 31 Main Office 33 Regions 34 Special	Code: 85-43
APPROVED: M. Tegza M. TEGZA, Final Plan Review Bureau	Date: 9/12/85 Supersedes:

On April 15, 1985 EI 85-25 was issued implementing item 18403.2501 for all asphalt overlays effective with the lettings of August 8, 1985. Since then the Materials Bureau has discovered that sometimes the concrete joints are milled to a depth of 3" or more. This results in a total overlay thickness greater than 4½" over the joint.

The current note 1 on page 4 would require a 1/8" wide sawcut over these milled joints but the wording may allow a Contractor to avoid constructing the 1/8" wide sawcut because of the reference to the T&L course.

The new note 1 requires the 1/8" wide sawcut be included whenever the total thick of asphalt concrete over the existing joint exceeds 4½ inches. This change sho eliminate problems interpreting when to sawcut. Also, the minimum depth has been increased to 2½" minimum.

The new item number will be 18403.2502. This will be effective with the letting of January 30, 1986.

ITEM 18403.2502 - SAWING AND SEALING JOINTS IN BITUMINOUS CONCRETE OVERLAYS

If the top course is to be placed the following Spring, due to seasonal paving limitations, all underlying courses shall receive a 1" deep by 1/8" wide sawcut to facilitate and control reflective cracking as well as to provide a means of properly referencing the sawcut to eventually be made in the top course. These sawcuts shall be made in all underlying courses within seven (7) days after the underlying courses are placed and before any evidence of reflective cracking has developed. Sealing of these sawcuts will not be required. Payment for sawcutting all underlying courses shall be included in the unit bid price for sawing and sealing.

Sawcutting of Transverse Joints. The contractor shall sawcut transverse joints to the appropriate dimensions shown in Figure I, based on the existing pavement slab length and new overlay depth. Full depth patches adjacent to joints in the underlying concrete shall have separate sawcuts in the overlay over the patch/slab interface. See Figure II. Sawcuts over patch interface shall conform to Figure I. The sawcut joints shall be directly over the existing portland cement concrete pavement joints and shall be accurately located by a method employing pins and stringline. The pins shall be accurately located prior to paving. Details of the method for locating the sawcuts shall be subject to the approval of the Engineer.

The blade or blades shall be of such size and configuration that the desired dimensions of the sawcut can be made with one pass. Either dry or wet cutting will be allowed. No spacers between blades will be allowed.

The transverse sawcut joints shall normally extend the full width of the pavement and shall extend into the asphalt shoulder to a distance three (3) feet beyond the edge of the underlying portland cement concrete pavement edge, unless otherwise detailed on the plans or in the proposal. Existing transverse joints that are offset at the longitudinal joint by more than 1 inch, measured between the centers of the joint cavities, shall require separate sawcuts terminating at the longitudinal joints.

Cleaning. Dry sawed joints shall be thoroughly cleaned with a stream of air sufficient to remove any dirt, dust or deleterious matter adhering to the joint walls or remaining in the joint cavity. Wet sawed joints shall be thoroughly cleaned with a water blast (50 psi minimum) immediately after sawing to remove any sawing slurry, dirt, or deleterious matter adhering to the joint walls or remaining in the joint cavity. Wet sawed joints shall be blown with air to provide dry joint surfaces prior to sealing.

All sawing slurry from the wet sawing process shall be immediately flushed from the pavement surface. Dry dust and material from the dry sawing process shall be blown or brushed off the pavement surface.

The contractor shall be required to provide protective screening, subject to the approval of the Engineer, if his cleaning operations are capable of causing damage to or interference with traffic in adjacent lanes.

DETAILS FOR TRANSVERSE JOINTS IN ASPHALT CONCRETE OVERLAYS

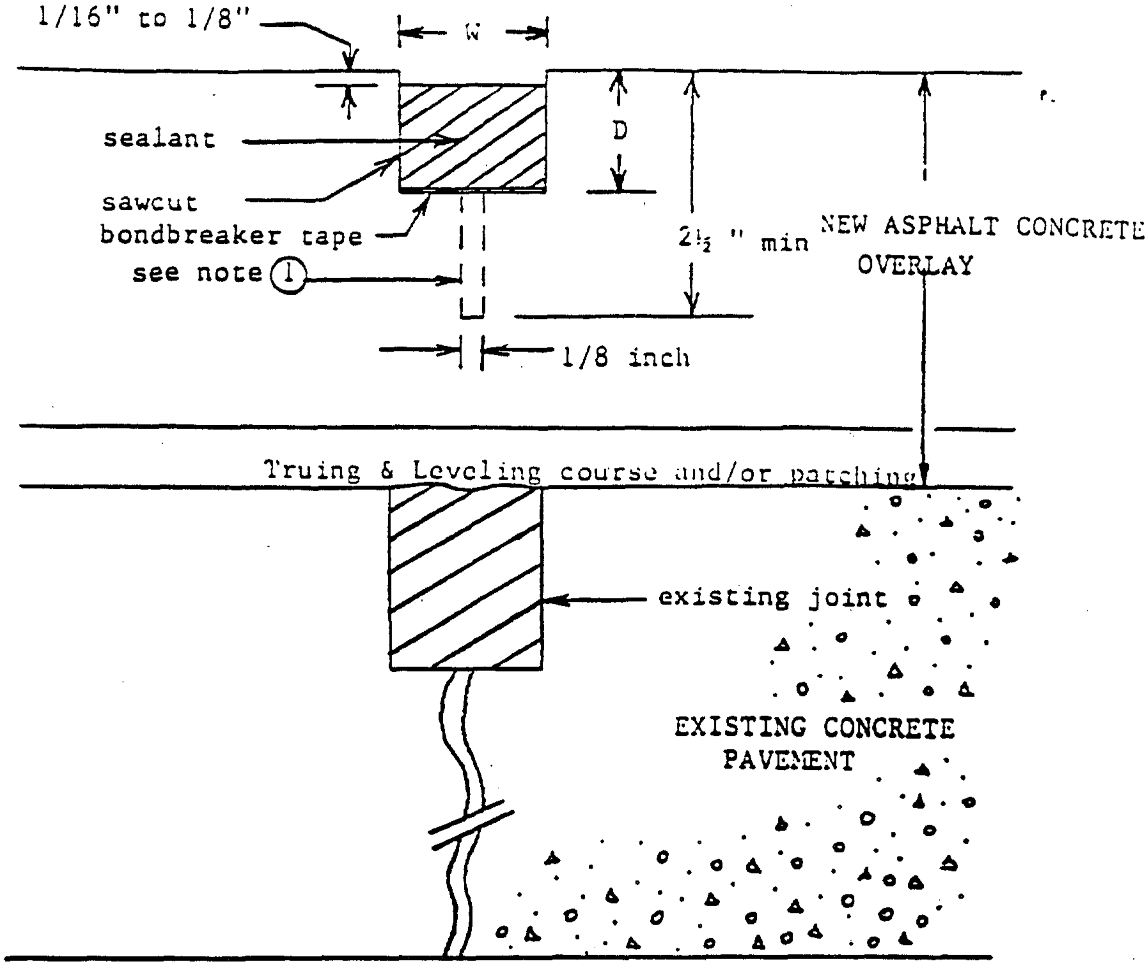
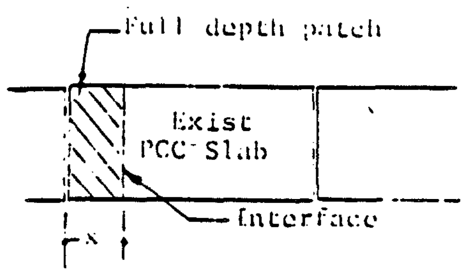


FIGURE I

SAWCUT DIMENSIONS

SLAB LENGTH	W	D
50 Ft or less	1/2 in	5/8 in
51 to 62 Ft	5/8 in	5/8 in
63 to 75 Ft	3/4 in	5/8 in
76 to 87 Ft	7/8 in	3/4 in
88 to 100 Ft	1 in	7/8 in



If "x" is greater than 0.5 ft. a sawcut is req'd at patch/slab interface in the overlay.

FIGURE II PLAN VIEW

NOTE 1

When the total thickness of asphalt concrete over the existing joint exceeds 4½ inches, an 1/8 inch wide sawcut shall be included in the joint geometrics to a minimum depth of 2½ inches.

3. CONSTRUCTION -

(a) General. Locate and reference the location of each existing transverse joint prior to placement of any bituminous courses. Make all saw cuts directly above the existing transverse joints.

Do not perform saw cutting until the bituminous course has thoroughly cooled. Perform saw cutting within 7 days after placement of the wearing course. Perform this work on all finished overlay areas prior to discontinuing of work due to seasonal paving limitations.

Extend the saw cuts the full width of the pavement including any widening. Provide separate saw cuts in each lane when existing transverse joints are offset more than 1 inch.

If the wearing course is to be placed the following construction season due to seasonal paving limitations, provide a 1 inch deep, 1/8 inch wide saw cut in the last placed bituminous concrete course.

(b) Sawing. When the total depth of overlay exceeds 4½ inches, not including scratch or leveling courses, make a 1/8 inch wide saw cut to a minimum depth of 2 inches or 1/3 of the total overlay thickness.

Saw a reservoir, in the wearing course having a width of ½ inch and a depth of 1 inch. If wet sawing is used, immediately flush the reservoir with water.

(c) Sealing. Do not place sealing material unless the reservoir faces are thoroughly clean and dry. Do not place on the same day as wet sawing. Clean the reservoir by using compressed air immediately before placing sealing material. Use compressed air free of oil, moisture, or any other substance that would prevent bonding of sealing material to the reservoir faces.

Do not place sealing material when the air temperature is less than 40 F. Use heating equipment of an indirect heating type, constructed as a double boiler. Provide positive temperature control and mechanical agitation.

Obtain the safe heating temperature and recommended pouring temperature from the manufacturer's shipping container. Place the material within this temperature range, but as close as possible to the recommended pouring temperature. Maintain a safe heating temperature. Maintain a single material batch at the pouring temperature for no more than 4 hours. Heat material only once.

Fill the reservoir with sealing material to a level 1/8 inch plus or minus 1/16 inch below the pavement surface. Do not allow sealing material to spread over the pavement surface.

4. MEASUREMENT - Linear Foot.

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

SUBJECT	FHWA NOTICE
HOT AND COLD RECYCLYING OF ASPHALT PAVEMENTS	N 5080.93 October 6, 1981

1. PURPOSE

To present the Federal Highway Administration's (FHWA) position on recycling of asphalt pavements.

2. CANCELLATION

The FHWA Technical Advisory T5040.9 dated February 16, 1979, Hot Recycling of Asphalt Pavement Materials, is cancelled.

3. BACKGROUND

The pressing need to conserve energy and minimize costs in highway construction requires that special effort be made to identify and make the maximum use of procedures that will result in reduced energy usage and minimum cost. Because recycling of asphalt pavements has the potential to be an effective method of conserving energy and materials and reducing costs, it is FHWA's policy that recycled asphalt concrete, defined as asphalt concrete containing salvaged paving materials including the use of suitable reclaimed material from other projects, be allowed for use on all projects. States with insufficient experience to properly evaluate the reuse of these materials should take immediate steps to initiate experimental projects.

4. DEFINITIONS

a. Recycled hot asphalt concrete is an asphalt concrete mix, processed hot in a central plant, which consists of sized salvaged asphalt material, new asphalt, and/or recycling agents and new and/or salvaged aggregates, and meets all standard material and mix specifications for the type of mix being produced.

b. Recycled cold mix is an asphalt concrete mix, processed in a central plant or on the grade which consists of sized salvaged asphalt material, some type of stabilizing agent and new and/or salvaged aggregates. This material meets specifications of an asphalt aggregate base and generally requires that an asphalt surface course or surface seal be used.

5. PAVEMENT DESIGN

a. Recycling should be one of the options considered at the design stage of all rehabilitation projects. Material testing of the old pavement may be necessary to determine that recycling is a practical option. The decision to recycle or to overlay should be based on cost and performance on a life cycle basis rather than initial cost and should be specified by the contracting agency. It is emphasized that alternate bids between recycling and overlay are not recommended.

b. Cracks and material deficiencies in the overlaid pavement will cause reflective cracks and points of weakness to occur in an overlay. Cracks can be eliminated and material deficiencies can be corrected by recycling.

c. Recycled mixes placed experimentally as base layers, top structural layers, and wearing surfaces are still being evaluated and it would be premature to offer definite conclusions on life cycle performance. However, the earliest of those pavements are 5 years old or older and are performing as well as pavements constructed with new materials. While there is limited experience with recycled mixes, it appears that reasonable performance can be obtained.

d. It is reasonable to assume that a recycled layer is structurally equivalent to an equal thickness of new hot mix pavement provided the mix meets all of the laboratory design criteria for a new mix intended to perform the same functions.

e. Only proven methods and materials with which there has been adequate experience to assure success should be used on large projects with high traffic or heavy loading.

6. MIX DESIGN

a. Recommendations for detailed mix design procedures are contained in NCHRP Report 224. Gradation and other material requirements should be the same for a recycled mix as those developed for mixes using all new materials for the same type of pavement.

b. Distress observed on a few projects is directly attributable to improper or poor mix designs represented by low stabilities, uncorrected aggregate stripping problems, and low job achieved densities. These problems emphasize the need for proper mix design and construction control. Research results indicate that testing for water susceptibility is especially important for recycled mixes.

c. Variation in material properties of the pavements to be salvaged should be identified by sampling and a sufficient proportion of new material provided to reduce the variation to an acceptable level. Major changes in mix characteristics for various sections along the same route usually demand separate mix designs.

d. Removal and sizing of salvaged pavement materials have at times created additional minus 200 sieve material. The amount depends on the type and operation of the sizing process and aggregate properties. Final mix design should always be corrected to final properties of the material processed by the actual equipment used on the project. Large amounts of minus 200 sieve material or other gradation deficiencies can be compensated for by limiting the amount of salvaged material used in the recycled mix and varying the gradation of the added new material. Experience has indicated that in most cases crushing the recycled material to a maximum particle size of 2 inches is adequate for hot mix. Additional crushing may result in excess fines.

e. A soft asphalt alone has been used successfully to restore the penetration and viscosity of the reclaimed asphalt binder. A number of commercial recycling agents have also been used when salvaged asphalt binder in the salvaged material was severely hardened. Any proposed softening agent should be tested with the salvaged asphalt for the specific project, in the ratio to be used, to assure the desired properties of the combination are realized.

7. REMOVAL AND SIZING

The type and degree of deterioration in a pavement to be constructed and/or the type of material underlying the pavement will usually determine whether a full or partial depth removal technique is utilized. Full-depth pavement removal and sizing can be accomplished using standard construction equipment such as dozers and loaders and portable or stationary crushers or by milling machines. The latter process, although generally more expensive, allows removal of one lane without disturbing traffic movement on adjacent lanes. Excessive dropoffs can be minimized by milling successive levels to a specific depth. While milling machines usually are specified for partial depth removal, the choice of the method used for full-depth removal will be influenced by economics and maintenance of traffic through construction.

8. EQUIPMENT

Virtually all equipment manufactured today for the production of asphalt concrete can be built to produce acceptable recycled mixes and meet all air quality standards. Existing equipment can be modified at reasonable cost. In hot mix recycling, batch plants are generally limited to the reuse of a maximum amount of 50 percent salvaged asphalt material in a recycled mix, while an upper limit of approximately 70 percent is attainable in some drum plants.

9. SAVINGS

Materials savings are realized from the reduction in new asphalt and aggregate. Energy savings result primarily from reduced aggregate haul and drying, and asphalt transportation. Cost savings are greatly influenced by length of aggregate haul and distance from the plant to the job site. Other factors which have a major influence on bid prices are the degree to which contractors in the area are familiar with and equipped for recycling, the size of the State's present and projected recycling program, and State contract procedures.

10. RECOMMENDATIONS

a. Allow the contractor the use of salvaged asphalt materials and aggregates in the production of asphalt concrete.

b. Allow the contractor to determine the source and amount of salvaged material to be used as long as the mix produced meets all standard material and mix specifications called for in the contract.

c. Require that a revised mix design be submitted and approved prior to changing either the source or amount of salvaged material originally approved.

d. Serious consideration should be given to transferring ownership of all material to be removed to the contractor. This allows the owner agency to receive instant credit, in the form of lower bids, for the value of the salvaged material removed.

e. Do not specify how to remove and size a pavement scheduled for full-depth reconstruction; what type of hot mix plant (batch, continuous or drum) to use; the use of recycling agent--but allow it to be used; and what percentage of salvaged material to be used. All of these will be determined by economics resulting from the competitive bidding process.

f. Recycled hot asphalt concrete should be paid for on the basis of a bid price per ton regardless of the percentage of salvaged material used. This price per ton is also to include the costs of all new additional asphalt, recycling agent, and aggregate.

11. DISCUSSION OF RECOMMENDATIONS

These recommended practices will allow the production of recycled asphalt concrete, if economically feasible, at any time in any location. Because no restrictions are placed on percentages of used salvaged material, a batch plant owner can economically compete with owners of drum plants. If across the board use of salvaged materials is allowed in the production of asphalt concrete, the contracting industry can better justify gearing up for such production and write off the additional plant modification cost over a much larger tonnage basis over a longer period of time than on only one or two projects.

Transferring ownership of all removed salvaged material to the contractor encourages recycling because surplus material can be used in private work at additional savings to the contractor.

12. EVALUATION

Most highway agencies have successfully constructed one or more hot recycling projects and are continuing to develop new projects. These projects have been constructed under NEEP Project 22, Pavement Recycling, distributed by Notice N 5080.64 dated June 3, 1977. Many projects have also been constructed with technical and financial assistance from the Demonstration Projects program. It is recommended that the evaluation of these projects be continued to validate long-term performance projections. Broad participation is needed to provide the data base necessary to require additional projects to be programmed experimental. The projects under evaluation should be representative of recycling procedures adopted by a State which have become routine. When a significantly new or innovative feature is contemplated, or when a project is in a significantly different environment, the highway agency should be urged to designate the project as experimental.

A recycling data bank is being developed under a contract through the FHWA Office of Research that will provide a means of long term evaluation of pavement recycling. The contract is scheduled to be completed in 1982.

R. D. Morgan
Associate Administrator for
Engineering and Traffic Operations

Attachments

REPORT FROM WISCONSIN DIVISION, April 13, 1981

WISCONSIN

1981 RECYCLING PROGRAM

During the first 6 months of 1981, Wisconsin Department of Transportation (WISDOT) let 42 contracts involving recycling of the existing bituminous pavement. The dollar amount of these contracts totaled $40.4 million and included 52 Federal-aid projects. Contractor competition for these contracts has been good with only 1 out of the 26 successful contractors having more than three contracts.

The contracts let to date have provided 696,700 tons of recycled bituminous pavement for paving 418 lane miles. The average bid price for this recycled bituminous pavement has been $8.84 per ton. This is significantly less than the $14.24 per ton average for virgin bituminous concrete pavement. When the savings in asphalt and shoulder aggregate are considered, the savings are almost $8.00 per ton.

In addition to the above tonnage, eight contracts totaling 34,800 tons of single aggregate bituminous surface have been let with the contractor having the option to use recycled or virgin aggregate. Most (6) of these projects were relatively small and provided less than 3,500 tons of bituminous pavement per project. The two larger projects provided 8,800 and 13,350 tons of single aggregate bituminous surface. Five of these contracts with optional recycling were in Milwaukee County.

The contracts let in FY 1981 have also provided for salvaging 496,000 tons of existing bituminous pavement. The average cost of salvaging bituminous pavement has been $4.41 per ton.

In addition to the "normal" recycling type of project, Wisconsin's 1981 recycling program has included three contracts that provide for recycling as part of a sulfur extended asphalt pavement. The cost of the sulfur for these projects has averaged $149 per ton.

One of the major accomplishments in WISDOT's recycling program is the savings in energy, natural resources, and cost. It is estimated that the energy savings this fiscal year is equivalent to 915,000 gallons of gasoline; the aggregate savings is 574,700 tons of aggregate, and the cost savings is $4.8 million.

Reports Dealing with Recycling

NCHRP Synthesis of Highway Practice 54, "Recycling Materials For Highways," 1978.

NCHRP Report 224, "Guidelines for Recycling Pavement Materials," 1980.

American Society of Testing Materials, STP 662, 1976.

Association of Asphalt Paving Technologists, Volume 46, 1977; Volume 48, 1977; Volume 49, 1980.

Proceedings of the National Seminar on Asphalt Pavement Recycling, Dallas-Ft. Worth, Texas, 1980 - Transportation Research Record 780.

The above reports are available at a charge from:

The National Technical Information Service (NTIS)
Springfield, Virginia 22161

Evaluation of Selected Softening Agents used in Flexible Pavement Recycling, FHWA-TS-79-204, 1978.

Hot Recycling - Minnesota - Modified Dryer Drum, FHWA-TS-80-233, 1980

Hot Recycling - Wyoming Dryer Drum, FHWA TS-80-234, 1980.

The above reports are available free of charge from:

Federal Highway Administration
Region 15
Demonstration Projects Division (HDF-15)
1000 North Glebe Road
Arlington, Virginia 22201

REPORTS PREPARED
FOR
DEMONSTRATION PROJECT NO. 39
RECYCLING ASPHALT PAVEMENTS

FHWA-DP-39-1 - IN-PLACE RECYCLING OF ASPHALT PAVEMENT - REPUBLIC COUNTY, KANSAS - CONSTRUCTION REPORT - Clarence W. Smith - August 1978 - 30 pages

FHWA-DP-39-2 - SURFACE RECYCLING ASPHALTIC CONCRETE PAVEMENT - MC ALLEN, TEXAS - CONSTRUCTION REPORT - Wade D. Barnes and Jack T. Trammell - September 1877 - 58 pages

FHWA-DP-39-3 - WASHINGTON STATE DEPARTMENT OF TRANSPORTATION'S FIRST ASPHALT CONCRETE RECYCLING PROJECT - ELLENSBURG, WASHINGTON - CONSTRUCTION REPORT - R. V. LeClerc, R. L. Schermerhorn, and J. P. Walter - July 1978 - 52 pages

FHWA-DP-39-4 - RECYCLING OF ASPHALT CONCRETE-OREGON'S FIRST HOT MIX PROJECT - WOODBURN, OREGON - INTERIM REPORT - James Dumler and Gordon Beecroft - November 1978 - 56 pages

FHWA-DP-39-5 - PAVEMENT SURFACE RECYCLING ON PARKS HIGHWAY BETWEEN LITTLE SUSITNA RIVER AND WILLOW CREEK - ANCHORAGE, ALASKA - INTERIM REPORT - John W. Henry - February 1978 - 31 pages

FHWA-DP-39-6 - BLEWETT PASS RECYCLING PROJECT - BLEWETT PASS, WASHINGTON - PRELIMINARY REPORT - September 1979 - 57 pages

FHWA-DP-39-7 - MILLING BITUMINOUS SURFACE - ELLENDALE, NORTH DAKOTA - CONSTRUCTION REPORT - September 1978 - 32 pages

FHWA-DP-39-8 - EVALUATION OF RECYCLED BITUMINOUS PAVEMENTS - ELKHART COUNTY, INDIANA - FINAL REPORT - Barry L. Elkin - August 1978 - 60 pages

FHWA-DP-39-9 - RECYCLING OF ASPHALTIC CONCRETE PAVEMENTS - LARAMIE, WYOMING - INITIAL REPORT - Wyoming State Highway Department, Materials Division - February 1979 - 89 pages

FHWA-DP-39-10 - EVALUATION OF RECYCLED ASPHALT CONCRETE PAVEMENTS - KOSSUTH COUNTY, IOWA - CONSTRUCTION REPORT - Richard P. Henely - February 1979 - 52 pages

FHWA-DP-39-11 - RECYCLING ASPHALTIC CONCRETE PAVEMENT - ROSCOE, TEXAS CONSTRUCTION REPORT - Bobby R. Lindley - March 1979 - 142 pages

FHWA-DP-39-12 - EXPERIMENTAL PROJECT SURFACE RECYCLING OF ASPHALT CONCRETE PAVEMENT - NATCHEZ, MISSISSIPPI - PROGRESS REPORT - James D. Webb, Gayle E. Albritton, and Thomas L. Chance

FHWA-DP-39-13 - COLD RECYCLING - MENOMINEE INDIAN RESERVATION WISCONSIN - CONSTRUCTION REPORT - Steve Beckett and Roy J. Calbo - February 1979 - 45 pages

FHWA-DP-39-14 - EVALUATION OF RECYCLED ASPHALTIC CONCRETE - CHESTER, VIRGINIA - CONSTRUCTION REPORT - C. S. Hughes - August 1977 - 26 pages

FHWA-DP-39-15 - INTERIM REPORT ON HOT RECYCLING - Douglas J. Brown - April 1979 - 99 pages (English or Spanish)

FHWA-DP-39-16 - PAVEMENT RECYCLING PROJECT - GILA BEND, ARIZONA - CONSTRUCTION REPORT - Arizona Department of Transportation Research Division - October 1978 - 59 pages

FHWA-DP-39-17 - RECYCLING ASPHALT CONCRETE ON INTERSTATE 80 - GOLD RUN, CALIFORNIA - INTERIM REPORT - R. N. Doty and T. Scrimsher - April 1979 - 134 pages

FHWA-DP-39-18 - RECYCLING OF BITUMINOUS SHOULDERS - FERGUS FALLS, MINNESOTA - INTERIM REPORT - Ronald H. Cassellius and Roger C. Olson - March 1979 - 31 pages

FHWA-DP-39-19 - RECYCLING OF ASPHALT CONCRETE PAVEMENTS - PALM BEACH COUNTY, FLORIDA - INITIAL REPORT - Charles F. Potts and Kenneth H. Murphy - January 1980 - 35 pages

FHWA-DP-39-20 - COLD RECYCLING ASPHALT PAVEMENT - SHERVURNE, VERMONT - INITIAL REPORT - R. I. Frascoia and D. N. Onusseit - January 1979 - 42 pages

FHWA-DP-39-21 - SURFACE RECYCLING OF ASPHALT CONCRETE PAVEMENT - OHIO - PROGRESS REPORT - Willis B. Gibboney - November 1979 - 23 pages

FHWA-DP-39-22 - SURFACE RECYCLING OF ASPHALT CONCRETE PAVEMENTS - FORT MYERS, FLORIDA - INITIAL REPORT - Charles F. Potts and Kenneth H. Murphy - September 1979 - 62 pages

FHWA-DP-39-23 - RECYCLING OF ASPHALT CONCRETE PAVEMENTS - PANAMA CITY, FLORIDA - INITIAL REPORT - Charles F. Potts and Kenneth H. Murphy - December 1979 - 53 pages

FHWA-DP-39-24 - COLD RECYCLING OF PAVEMENT USING THE HAMMERMILL PROCESS - MAINE - FINAL REPORT - David W. Rand - December 1978 - 41 pages

FHWA-DP-39-25 - COWHERD ROAD COLD ASPHALT RECYCLING PROJECT - JACKSON COUNTY, MISSOURI - CONSTRUCTION REPORT - Kirk Phillips - November 1979 - 99 pages

FHWA-DP-39-26 - COLD BITUMINOUS PAVEMENTS RECYCLING - WIBAUX, MONTANA CONSTRUCTION REPORT - John J. Wright - May 1979 - 75 pages

FHWA-DP-39-27 - COLD RECYCLING OF A SOIL-ASPHALT ROADWAY - BEAVER COUNTY, OKLAHOMA - INTERIM REPORT - Jack C. Stewart - April 1980 - 52 pages

FHWA-DP-39-28 - HOT MIX RECYCLING - DURANGO, COLORADO - INTERIM REPORT - Robert F. LaForce - May 1980 - 61 pages

FHWA-DP-39-29 - BITUMINOUS CONCRETE PAVEMENT RECYCLING - INTERIM REPORT - Edgar J. Hellriegel - NORTH BRUNSWICK, NEW JERSEY - July 1980 - 61 pages

FHWA-DP-39-30 - HOULTON - LITTLETON HOT RECYCLING PAVING PROJECT - HOULTON, MAINE - PRELIMINARY & CONSTRUCTION REPORT - D. W. Rand - March 1980 - 61 pages

FHWA-DP-39-31 - HOT RECYCLING OF ASPHALTIC CONCRETE PAVEMENT - BEAVER, UTAH - INTERIM REPORT - Wade B. Beteson - October 1980 - 170 pages

FHWA-DP-39-32 - 1980 PAVEMENT RECYCLING PROGRAM - SPRINGFIELD, MISSOURI - INTERIM REPORT - prepared by Anderson Engineering, Inc. - January 1981 - 75 pages

OTHER RELATED RECYCLING REPORTS

DEMONSTRATION PROJECT NO. 39 - RECYCLING ASPHALT PAVEMENTS - PROJECT STATUS REPORT - February 1979 - 66 pages

RECYCLING OF ASPHALTIC CONCRETE - ARIZONA'S FIRST PROJECT - James A. McGee and A. James Judd - 28 pages

MINNESOTA HEAT TRANSFER METHOD FOR RECYCLING BITUMINOUS PAVEMENT - REPORT ON MAPLEWOOD, MINNESOTA, RECYCLING PROJECT - Richard C. Ingberg, Richard M. Morchinek, and Ronald H. Cassellius - 1977 - 43 pages

EVALUATION OF AIR POLLUTION CONTROL DEVICES FOR ASPHALT PAVEMENT RECYCLING OPERATIONS - PROGRESS REPORT - Richard P. Henely - December 1977 - 47 pages

RECYCLING ASPHALT CONCRETE PAVEMENT - DEPARTMENTAL RESEARCH REPORT NO. 524-1-F - DHT 1-9-76-524-1F - Charles H. Hughes - August 1977 - 145 pages

COLD RECYCLING OF ASPHALT CONCRETE PAVEMENT - EXPERIMENTAL PROJECTS - REPORT NO. 613-1 - B. R. Lindley - October 1975 - 27 pages

RECYCLED ASPHALTIC CONCRETE PAVEMENT - SR-26, SR-100 TO HOLDEN RS-0303(3) - Wade B. Betenson - February 1979 - 94 pages

COLD RECYCLING OF PAVEMENT BY HAMMERMILL PROCESS - INTERIM REPORT - David W. Rand - August 1977 - 82 pages

RECYCLING OF SUBSTANDARD OR DETERIORATED ASPHALT PAVEMENTS - A GUIDELINE FOR DESIGN PROCEDURES - Donald D. Davidson, William Canessa, and Steven J. Escobar - February 1977 - 51 pages

FHWA-DP-PC-1000-1 - PRODUCTION EFFICIENCY STUDY ON PAVEMENT PLANING EQUIPMENT - INTERIM REPORT - David R. Lewis - March 1979 - 58 pages

HOT RECYCLING IN HOT-MIX BATCH PLANTS - National Asphalt Pavement Association - 5 pages

PRODUCING A BITUMINOUS WEARING COURSE BY DRUM MIX RECYCLING (MICHIGAN) - R. B. Moore and R. A. Welke - January 1979 51 pages

BATCH PLANT RECYCLING (MICHIGAN) - John E. Norton - April 1979 - 30 pages

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MIXED-IN-PLACE STABILIZATION OF HIGHWAY BASE AGGREGATES AND PULVERIZED BITUMINOUS SURFACING USING ASPHALT STABILIZERS (MICHIGAN) - J. H. DeFoe - March 1977 - 39 pages

RECYCLING OF BITUMINOUS MAINLINE AND SHOULDERS (MINNESOTA) - Roger C. Olson - February 1979 - 26 pages

RECYCLING OF ASPHALTIC CONCRETE PAVEMENTS NO.2 (WYOMING) - Materials Division of Wyoming State Highway Department - 86 pages

RECYCLED COLD-MIX ASPHALT BASE CATOCTIN MOUNTAIN PARK (REGION 15, FHWA) - William F. Bensing - December 1978 - 34 pages

HOT MIX RECYCLING GEORGE WASHINGTON MEMORIAL PARKWAY (REGION 15, FHWA) - Reynaldo Cortez - 31 pages

EXPERIMENTAL TEST SECTION NEAR COVE FORT (UTAH) - Uath Department of Transportation - 59 pages

RECYCLING ASPHALTIC CONCRETE PAVEMENT (TEXAS) - FINAL REPORT (I-20 PROJECT) - Bobby R. Lindley - January 1980 - 4 pages

EVALUATION OF RECYCLED ASPHALT CONCRETE PAVEMENTS (KOSSUTH COUNTY, IOWA) - FINAL REPORT - Richard P. Henely - 30 pages

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: Use of Recycled Concrete in Portland Cement Concrete Pavements

Date: JUL 25 [redacted]

From: Chief, Pavement Division
Washington, D.C. 20590

Reply to Attn. of: HHO-12

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

A Pavement Design and Rehabilitation Team Review was recently made of pavements, located in one State, reconstructed with recycled portland cement concrete (PCC) pavement. The purpose of the review was to analyze causes for the transverse working cracks which were developing in many of the slabs.

The pavements were reconstructed in 1984 and 1985 using the recycled existing PCC pavement. The design called for a 10-inch reinforced PCC pavement with plain PCC shoulders and an open-graded granular (unstabilized) subbase. The mainline pavement joints were spaced at 41 feet.

Numerous intermediate cracks with spalling and faulting were observed. Typically there were two cracks per slab, occurring at the third points. These cracks were significantly more severe in the driving lane. A few slabs had also developed one or two additional cracks, some of which showed signs of staining.

Observation of the concrete which was removed from the pavement indicated that some of the recycled material, used as large aggregate in the reconstructed pavement, was mainly mortar with very little if any aggregate.

The team made the following recommendations to this specific State:

1. Based on recent findings, it is our recommendation that recycled PCC pavement not be used as aggregate in reinforced PCC pavements. The wire mesh reinforcement in reinforced pavements is intended to hold cracks close together so that load transfer can be obtained through aggregate interlock. However, the recycled concrete aggregate does not appear to provide adequate aggregate interlock for two reasons. The first is due to its fairly small size; on these projects the top size was 1 inch.

The second is due to portions of this larger aggregate being merely lumps of mortar, which easily grind smooth with pavement deflections caused by traffic loading. Since the recycled pavement does not provide sufficient aggregate interlock, the wire mesh reinforcement is subjected to excessive shearing forces. As a result, the wire mesh ruptures and the crack begins to function as a working joint. Plain PCC pavements are designed not to crack, so aggregate interlock is not a factor, providing dowel bars are properly installed at the joints.

2. If the decision is made to use recycled PCC pavements as aggregate in reinforced pavements, it is recommended that a 3-sized aggregate mix be used, with the recycled PCC pavement serving as the middle-sized aggregate. A larger-sized coarse material should be added to provide the necessary aggregate interlock.

3. The high absorptive level of the recycled aggregate (approximately 6 percent) may have resulted in high drying shrinkage of the concrete. This in turn could cause the cracks to open wider than normal, further reducing load transfer through aggregate interlock. When recycled concrete pavement is used as aggregate, consideration should be given to moistening the aggregate prior to adding it to the mix.

Based on the performance of the recycled concrete in reinforced pavement which was observed in this State, we believe a review of reinforced concrete pavements constructed with recycled concrete is warranted. We would appreciate your assistance in identifying reinforced pavements, both jointed and continuously reinforced, containing recycled concrete as aggregate. These pavements should have carried traffic at least 2 years and be located on the Interstate or a route carrying moderate to high volume truck traffic.

We would also like to receive information on any investigations the State may have undertaken to evaluate the load transfer at cracks in reinforced pavements with recycled aggregates.

We are planning to complete this review during September and October so the results will be available to the States for use in developing their 1990 projects. Mr. John Hallin will be performing the review. Please contact him at 366-1323, if you have any questions. To expedite the review, please advise him, by phone, of the projects which are available for review.

Louis M. Papet

The Use of Recycled PCC as Aggregates in PCC Pavements

Stephen W. Forster
Construction, Maintenance and
Environmental Design Division
Office of Engineering and Highway
Operations Research and Development

February 1985

The Use of Recycled PCC as Aggregates in PCC Pavements

Table of Contents

The Use of Recycled PCC as Concrete Aggregate

1. Introduction

1.1 The Incentive to Recycle. Economic considerations are the primary reasons for recycling, although often there are also environmental benefits to be derived. In some areas of the country there are no available supplies of virgin aggregates and recycling is the only viable economical solution. In other areas available sources of new rock are inaccessible, either because the value of the land is too high, or because zoning-type constraints prevent the opening of pits or quarries to obtain the material. In some instances, such as highly developed urban areas, economic incentive comes from the inability to properly dispose of the wasted material, and hence, it is less expensive and more environmentally acceptable to re-use it. Therefore, when a PCC pavement will be removed prior to replacement with a new pavement, the project is a prime candidate for recycling, thereby serving as a source of aggregate in the new concrete and eliminating the need and expense of disposing of the material removed. Further, if the project is large enough to set up an aggregate plant on site, additional savings can be realized by the elimination of much of the materials' transportation costs.

1.2 Recycling History. Results of a 1971 survey conducted by the Texas State Highway Department and the Texas Transportation Institute (ref.1) indicate that at that time little consideration was being given by most States to recycling existing pavement material other than as unstabilized base courses. PCC removed from a roadway was normally disposed of in landfills, or at best as erosion control in drainage ditches. This attitude has changed, as the use of natural resources and energy has had increasing economic impact.

Proposals to use recycled PCC as concrete aggregate material generated a number of questions. First, what would the quality of the new concrete containing the recycled material be, compared to the old concrete and also to new concrete made with natural aggregate?

Would the crushed concrete make good aggregate? How could the reinforcing be easily removed? Would recycling for this purpose (aggregate) be an economically viable alternative? These questions, and many others, concerning the recycling of PCC have now been substantially answered by subsequent work. This report will deal particularly with the use and properties of the recycled material as aggregates in PCC.

1.3 FHWA Involvement. The Federal Highway Administration (FHWA) initiated Demonstration Project No. 47 (DP47), Recycling Portland Cement Concrete Pavements, in May, 1978, and it is still active. The initial report under this project was the reprinting of an Iowa Department of Transportation report on an early recycling project, which is summarized later in this report. A number of other States have since conducted recycling projects under DP47 and States continue to show interest in participating in DP47.

FHWA also established project 22 on pavement recycling under its National Experimental and Evaluation Program (NEEP) in June, 1977. Both asphalt and portland cement concrete recycling were included in NEEP 22. Throughout its duration 42 States participated in the project which has now been integrated into either DP 47, mentioned above, or DP39, Asphalt Pavement Recycling.

FHWA sponsored a national seminar on PCC recycling and rehabilitation in September, 1981, which was conducted by the Transportation Research Board (TRB). Many of the details given is this report are from the proceedings volume (ref. 2) and the summary volume (ref. 3) for this meeting.

1.4 Definitions. Recycling as applied to PCC pavements many be grouped into 3 categories. First is surface recycling, which includes milling or grinding the surface (approximately the top inch (25 mm)) of the pavement to remove surface deterioration, restore rideability, and improve surface friction. The material removed is usually quite fine and in relatively small quantities, so it is normally not used as concrete aggregate. A second type of recycling is in-place recycling in which the old pavement is crushed and combined with the existing base or subbase material to form a base for support of a new pavement. The third type of recycling may be called plant recycling, in which the existing PCC pavement is broken up, removed from the roadway to a crushing operation, crushed and sized. The aggregate material thus produced is incorporated in a new PCC mixture for placement on the job. It is this use of the old concrete as aggregate in new PCC which will be the major topic of this report.

2. Properties of Recycled PCC Aggregate

2.1 Aggregate Tests. A number of laboratory studies have compared the properties of aggregate material made from crushed PCC with the properties of natural aggregates. Early work in this area was done by Alan Buck of the U.S. Army Engineers Waterways Experiment Station (WES), (ref. 4). Buck examined the properties of aggregate made from crushed concrete containing chert gravel (coarse) and natural sand (fine) and a second aggregate made from crushed concrete containing limestone (coarse) and natural sand (fine). These manufactured aggregates were tested and compared with natural aggregate and then incorporated into new concrete mixes for further comparisons. Results of absorption and specific gravity tests are shown in Table 1.

Visual inspection of the crushed concrete indicated a good particle shape. The fine aggregate as produced did not meet the normal gradation requirements, but was used as produced in the concrete mixes.

Results of studies conducted by WES, the Iowa Department of Transportation, Massachusetts Institute of Technology, The Minnesota Department of Transportation, the Michigan Department of Transportation and FHWA are summarized by Yrjanson (ref. 1). He found the following points of agreement:

> The aggregate particles produced by crushing concrete have good shape, high absorptions and low specific gravity compared to natural mineral aggregates.

The Michigan Department of Transportation (ref. 5) conducted a laboratory investigation of a series of crushed concrete materials for comparison with natural aggregate. Table 2 shows their test results. Michigan also tested a concrete material which had been recycled twice. Its specific gravity was still lower (2.11) and the absorption event higher (8.36 percent). These results are predictable since with each successive recycling the amount of natural aggregate decreases when expressed as a percent of the aggregate material and the amount of lighter, more absorptive cement paste increases. Interestingly, the soundness loss of the recycled material was less (0.9-2.0) then that of the natural aggregate (3.9).

2.2 Concrete Tests. Buck (ref. 4) made all his recycled concrete mixes with a water cement ratio of 0.49, a target air content of 6 ± 1/2 percent, and a slump of 2 1/2 ± 1/2 inches (63 ± 13 mm). He found that concrete made with recycled concrete as both coarse and fine aggregate had lower slumps and higher cement content than comparable mixes made with either all natural aggregate or recycled coarse aggregate and natural sand fine aggregate. He also noted that the concrete with recycled aggregate had compressive strengths 300-1300 psi (2068-8962 kpa) less than the control concrete throughout the period of testing (up to 180 days of age). Freeze-thaw test results differed depending on the original aggregate type. Recycled concrete containing freeze-thaw susceptible coarse aggregate performed better as aggregate in a new concrete than concrete containing that stone as coarse aggregate (although whether the improvement is sufficient to bring performance to an acceptable level would have to be judged on a case by case basis). Conversely, new concrete made with recycled concrete containing an originally freeze-thaw resistant aggregate performed somewhat worse than the control mix with the natural coarse aggregate, although both mixes performed acceptably well. Finally, Buck found that volume change in response to temperature changes or increased moisture was similar for the recycled concrete mixes and the controls.

Yrjanson (ref. 1) presented the following conclusions about the recycled concrete in his report:

1. The use of crushed concrete as coarse aggregate had no significant effect on mixture proportions or workability of the mixtures compared to the control mixes.

2. When crushed concrete was used as fine aggregate the mixture was less workable and needed more water and therefore more cement. Substitution of natural sand for up to 30% of the recycled fine aggregate improves workability to the approximate levels of a conventional mix.

3. The frost resistance of the concrete made from recycled aggregates was usually much higher than that made with natural aggregates.

4. The use of recycled aggregate did not have any significant effect on the volume response of concrete specimens to temperature and moisture changes.

5. The use of low strength recycled concrete as aggregate need not be detremental to the concrete's compressive strength.

6. The use of water reducing admixtures to lower the water content is effective in increasing strengths of concrete mixtures that contain recycled concrete as aggregate.

Fergus (ref. 5) reported that the Michigan Department of Transportation used various percentages of recycled PCC in the fine aggregate to determine its effect on the mixture. They also used various percentages of recycled bituminous concrete in the mixture to simulate contamination which would occur in practice. They made their mixtures with a water cement ratio of 0.43, a cement factor of 6 sacks/cuyd (7.8 sacks/m^3) and an entrained air percentage of 5.5 $\pm$ 1.5. The results of this research agrees with the findings of

Buck and Yrjanson. The slump of the recycled concrete mixtures was less than that of the control mixture due to differences in workability. Compressive and flexural strengths of the recycled concrete were slightly less than those of the control mixture made with a gravel aggregate, but still exceeded the Michigan Department of Transportation minimum specifications for pavement concrete. Those recycled materials with crushed bituminous concrete (patches, unremoved overlay spots, etc.) included as a small percentage of the aggregate were not detrimentally affected unless there was an inclusion of crushed bituminous fines. These fines are almost totally bitumen coated and therefore act essentially as voids in any strength test of the new concrete. The recycled concretes exhibited durability factors superior to that of the control mix.

3. Special Concerns for Recycled PCC

3.1 Recycled "D" Cracked Pavement. The possible use of crushed "D" cracked pavement as an aggregate material presents an additional concern. The question posed is whether the recycled material will continue to promote "D" cracking, or will the problem be alleviated (at least to the level of economically available natural aggregate material) by the crushing which takes place during the recycling process.

Prior to carrying out a recycling project using a "D" cracked PCC pavement, the Minnesota Department of Transportation conducted a laboratory study (ref. 6) to determine the behavior of recycled "D" cracked material when used as aggregate in new concrete. For the laboratory work a three foot (0.98 m) section, full width of the candidate pavement, was removed and crushed for testing in the laboratory. Four initial mixture designs contained: 1) 100 percent recycled aggregate; 2) recycled coarse aggregate and natural sand fine aggregate; 3) the same as 2, except fly ash was substituted for 10 percent of the cement; 4) the same as 2, except 20 percent fly ash was substituted for 15 percent of the cement. They also made a control with all natural aggregate and 20 percent fly ash substituted for 15 percent of the cement. Like other investigators, they found that the recycled material passing the No. 4(4.75mm) sieve was very angular and that this increased the water demand substantially (to provide acceptable workability). Mix 1 (recycled fine aggregate) required 333 lbs/yd^3 (197.5 kg/m^3) of water versus 250-260 lbs/yd^3 (48.3 to 15.42 kg/m^3) for the control. This higher water demand also increased the cement requirement. Compressive strengths were at or above conventional mixtures and they had no problem entraining the necessary air. Based on these results, three more trial mixes were made. The recycled aggregate all passed the 3/4 inch (19 mm) sieve and 0-5 percent passed the No. 4(4.75 mm) sieve. One of the mixes had no fly ash, one had 10 percent of the cement replaced by fly ash, and the third had 15 percent of the cement replaced by 20 percent fly ash. To evaluate the "D"-cracking susceptability, these mixes were subjected to freeze-thaw testing. In comparison with concrete containing the "D" cracking natural aggregate, the concrete with the recycled concrete aggregate was somewhat more resistant to freeze- thaw action, and the mixtures with 10-20 percent substituted fly ash had a greatly reduced "D" cracking potential. The fly ash also acted as a plasticizer, thereby lowering the amount of water necessary to make the mix workable.

Based on these laboratory results, the State went ahead and reconstructed U.S. 59 using the recycled concrete as coarse aggregate. The specific gravity of the recycled coarse aggregate was 2.41 and its absorption was 4.4 percent. Natural sand was used as the fine aggregate and 20 percent fly ash was substituted for 15 percent of the cement. Average core strength on the concrete was 4590 psi (31.6 MPa) after 60 days. The minus number 4 (4.75 mm) recycled material was used in the base course as a stabilizing material.

3.2 Salt Content of Recycled Pavement. As part of the Michigan study summarized above (ref. 5), they examined the NaCl content of the recycled PCC aggregate material, since large amounts of rock salt are used as a deicer on their highways. They found that the recycled material contained less than 2 lbs/yd^3 (1.2 Kg/m^3) compared to their critical NaCl level of 4 lbs/yd^3 (2.4 Kg/m^3) used for bridge decks. They concluded that no restrictions were necessary on the use of the material based on its salt content. Further, since the recycled material is used as only the aggregate portion, the overall level of chloride in the new concrete would be even less (the amount in the recycled PCC times the fraction of the new concrete which is recycled material).

In preparation for a recycling project, Connecticut (ref. 7) examined the total chloride content of recycled PCC material. They found 12 lbs/yd^3 (7.1 kg/m^3) at the 1.5 in (38 mm) level, 0.96 lbs/yd^3 (0.57 kg/m^3) at the 4in (102 mm) level, and 0.27 lb/yd^3 (0.16 kg/m^3) at the 6.5in (166 mm) level. The new mixture with the recycled concrete aggregate contained 1.93 lb/yd^3 (1.14kg/m^3) total chloride.

To summarize, it would be advisable to check the NaCl content of any recycled material which may have excessive salt, and based on the findings calculate what the salt content would be for the new mix. Based on the results, a decision could be made as to whether any additional steps (reinforcement coating, etc.) would be necessary to avoid problems.

3.3 Alkali - Aggregate Reactivity. Three things are necessary to cause damaging alkali - aggregate reactivity: 1) an aggregate with sufficient amounts of reactive constituents that are soluble in highly alkaline aqueous solutions; 2) enough water soluble alkali from some source (usually the cement) to drive the pH of the liquid in the concrete up to 14-15 and hold it there so that swelling alkali - silica gel is produced; 3) sufficient water to maintain the solutions and provide moisture for the swelling of the gel.

The consequences of using recycled PCC material which has suffered from alkali - aggregate reaction as an aggregate in a new concrete have not been throughly studied. In this special case of PCC recycling, several questions must be answered. How severe is the extent of the reaction and the resultant distress at the time of recycling? Has the reaction gone to completion - that is, has the reactive mineral matter been used up? If petrographic or other examination seems to indicate this, it may be safe to go ahead and use the material. On the other hand, merely the use of a low alkali-cement in the new concrete may not prevent further alkali-aggregate reaction with the recycled material because the reaction may continue within the recycled material between the old mortar and aggregates. Probably the only safe way to screen materials with this potential problem is to do long term mortar bar expansion tests (ASTM C-227) with the recycled material in cements with various alkali contents to determine what level of alkali is acceptable. If reaction is taking place between the recycled materials, it may be that no level of alkali - in the cement will be low enough to prevent the reaction. It has been speculated that the addition of limestone aggregate in the mix may reduce the probability of alkali-aggregate reactivity (ref. 8) but this is not yet proven. Reduction in recycled aggregate size may also be helpful in controlling the reaction problem. The question of recycling alkali-aggregate reactive materials needs additional investigation, and work is currently underway in a cooperative study in Colorado.

4. Field Projects with Recycled PCC

As a result of field projects incorporating recycled PCC as aggregate in the mixture, several facts were learned which should aid in the planning and conduct of future recycling projects. Iowa (ref. 9) had one of the early recycling projects on U.S. Rte 75 in 1976. They stockpiled the entire crushed recycled PCC from the secondary crusher (1 1/2 inch (38 mm) minus) in a single stockpile and found that segregation problems resulted as well as inconsistent feed through the automatic bin gates of the batching plant. They therefore went to splitting the material on the 3/8 in. (9.5 mm) sieve on subsequent projects, which alleviated the problem. Using recycled material for both coarse and fine aggregate produced a harsh mix which was nearly unworkable, so 15 percent concrete sand was added which made the mixture much easier to work. It was found that less air entraining agent was needed to reach the desired air content than would have been true with a conventional mix. The amount of contaminants in the recycled material must be controlled because they often have an effect on the air content of the new concrete. They found that approximately 75-80 percent of the old pavement is recovered as crusher product. Using the experience gained in the initial project, Iowa conducted two additional projects in 1977. As was found in the first project, the crusher product was low in fine material (22-24 percent passing the number 4 (4.75mm) sieve). A three aggregate blend (coarse and fine recycled, plus concrete sand) controlled segregation of the recycled material and made for a workable mixture. Washing the recycled material was found to be unnecessary if proper removal and processing practices were followed.

Minnesota (ref. 6) conducted a recycling project on U.S. 59 in the southeastern part of the State in 1980. This was a "D" cracked pavement and the results pertaining to that particular problem are discussed in section 3.1. However, several conclusions reached as a result of this project are applicable to recycling projects in general. As in Iowa, Minnesota found that the crushed material passing the number 4 (4.75mm) sieve is very angular and results in increased water demand and cement content when used in the mix. To avoid this situation, Minnesota removed the minus number 4 (4.75mm) material from the crushed concrete and used it as a stabilizer in the base material. They found that even in this use it needed constant watering to achieve target densities. They calculated that they would have enough recycled material for coarse aggregate in the mix if they had an aggregate blend of 60 percent coarse aggregate and 40 percent natural sand. The actual yield proved to be very close to this estimate.

5. Specifications

Several States (Iowa, for example, ref. 10) have developed specifications for removal, crushing, storing, and incorporating recycled materials in new PCC. These specifications cover all phases of the construction, and the reader is referred to them for this information. The discussion of specifications here will be limited to items directly effecting the recycled aggregate material.

5.1 Removal and Contamination. Some limit should be set on the amount of allowable contamination in the material recycled, either from any asphalt overlay, patch, joint sealant or subbase material. It has been found that some amount of adhering asphaltic concrete is allowable and not detrimental to the mixture.

5.2 Crushing and Stockpiling. Maximum size of material should be specified and may vary depending on the use of the concrete, however, typically top size is specified as 100% less than 1 1/2 inches (38 mm). The maximum size specified may have to be reduced (100% less than 3/4in (19 mm)) if the material being recycled is a "D" cracked pavement. Standard good stockpiling techniques should be followed, and the plus 3/8 in. (9.5 mm) and minus 3/8 in. (9.5 mm) should be stored separately to avoid segregation. Washing is not normally necessary, however this would be dictated by individual job conditions. Provision should be made to limit the amount of minus 200 (.075 mm) material to some maximum percentage.

5.3 Mix Proportions. Crushed recycled material may be used for both the coarse and fine aggregate, however use of 15-30% natural sand in the fines may be specified to improve workability and finishability of the mix. Mix proportions should be determined based on trial mixes made in the laboratory. An effort should be made to proportion use of the coarse and fine recycled material in the same ratio as it is produced by the crusher.

Cement factor will be determined according to the strength desired, as with a conventional mix. Water shall be used in a ratio which will provide acceptable workability and finishability without being so high that excessive cement is required to maintain strength. To this end, addition of natural fine aggregate (as noted above) may be specified to improve these characteristics while holding the water content at a reasonable level. Water reducing admixtures may also be considered for the specification to maintain the water cement ratio at an acceptable level. Air entrainment will also increase workability.

5.4 Durability. The durability of the concrete produced should be required to be checked in the laboratory according to ASTM C-666 or some equivalent method. If alkali-aggregate reactive material is being recycled, the expansive characteristics of the new concrete may also be checked by ASTM C-227 or equivalent to determine if it will perform adequately.

5.5 Air Entrainment. Air content may be specified and obtained using the addition of an approved air entraining agent as with a conventional mix. If the recycled material is air entrained, the specified air for the new concrete may have to be set higher than normal since the measured air will include the newly entrained air plus the air content of the recycled material. When the air content of the recycled material is subtracted from the measurement obtained on the new plastic concrete, the residual will then provide a measure of the amount of air in the new mortar. The presence of organic contaminants may cause high air contents and therefore de-air entraining agents may be needed.

6. Summary & Conclusions

This report is an assemblage of the current knowledge on the use of recycled PCC as aggregate in new concrete construction. The following points highlight its contents.

1. Recycling PCC is a viable alternative to using natural aggregate in concrete construction in many instances, particularly those in which the natural aggregate would have to be transported some distance and there is a problem disposing of the old concrete removed.

2. FHWA continues to encourage States to try recycling projects through its Demonstration Project 47, Recycling Portland Cement Concrete Pavements.

3. The recycled material may be tested using many of the same tests used for natural aggregate material. Recycled PCC tends to have a higher absorption and lower specific gravity than natural aggregates. The crushed material has a good particle shape.

4. The use of recycled concrete as the aggregate in a new mixture has several effects. If the recycled material is used for the fine aggregate, its harshness decreases the workability of the mixture. This may be compensated for by substituting some natural aggregate fines for the recycled material, increasing the water (and therefore the cement) content, adding a water reducing admixture, or some combination of the three.

5. The freeze-thaw resistance of the new concrete is generally better than that of a comparable concrete made with natural aggregates.

6. The durability of recycled "D" cracked concrete is greatly improved over that of the original concrete, and may be improved still more if necessary by specifying a reduced maximum size for the recycled material. Fly ash appears to decrease the tendency for "D" cracking in the recycled concrete mix.

7. Compressive and flexural strengths of recycled concretes tend to be slightly less than those of comparable mixes with natural aggregates, however strengths above the minimum normally required are still easily obtained with proper mix design.

8. In the few studies examining the possible problems of recycling salt contaminated concrete, the NaCl levels were not high enough to promote distress. More work needs to be done to determine the level at which salt content in the recycled material becomes determinantal to the new mix, particularly since the recycled material is usually

used in pavements rather than bridges which present different corrosion conditions.

9. The use of recycled PCC suffering from alkali-aggregate reactivity in a new concrete has not been adequately addressed. The surest approach at this time is to subject any suspected material to the mortar bar expansion test (ASTM C-227) to evaluate of its behavior. Further research is needed in this area.

10. Specifications for recycled PCC aggregate material should have the performance requirements which are generally applied to natural aggregates. Attention must be paid to the recycled material's effect on the workability of the new mix and the various ways to improve it. Depending on the condition and distress of the recycled pavement, statements may have to be included in the specification to require testing for durability, expansion, permeability and strength.

11. <u>Recommendations and Extensions</u>

The recycling of PCC as aggregate in a new concrete mix is a viable alternative to the use of natural aggregates in many instances. Experience has shown that with proper planning, testing, and construction techniques, quality concrete can be made using recycled PCC as aggregate.

There are a number of recycling situations where additional study still needs to be done to determine long term effects. The recycling of concrete which has suffered from alkali-aggregate reaction still involves some unknownsas to the long term behavior of the recycled aggregate material. A cooperative study is currently being done in Colorado to determine the effects of fly ash on new mixtures using this type of recycled aggregate.

The presense of chlorides in the recycled concrete is another area of concern. We know pretty well what levels of chloride content are critical in causing corrosion of bridge deck reinforcement. However, in pavement concrete there is usually much less steel which has a greater cover of concrete. The effect of having the chloride concentrated in the aggregate initially is also unknown.

A synthesis study on recycling of PCC pavement is included in the National Cooperative Highway Research Program for fiscal 1985. This will summarize current knowledge and practices in this subject area. Recycling of PCC will also be one of the subjects addressed by the Strategic Highway Research Program, now in the planning stage.

References

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2. Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, December 1981, 196 p.

3. Pavement Recycling: Summary of two Conferences, FHWA-TS-82-208, April 1982, 66 p.

4. Buck, A.D., Recycled Concrete, in Utilization of Waste Materials and Upgrading of Low-Quality Aggregates, HRR 430, 1973 pp 1-8.

5. Fergus, J.S., Laboratory Investigation and Mix Proportions for Utilizing Recycled Portland Cement Concrete as Aggregate, in Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, December 1981, pp 144-160.

6. Halverson, A. D., Recycling Portland Cement Concrete Pavements, FHWA-DP-47-3, May 1981, 66 p.

7. Lane, K. R., Construction of a Recycled Portland Cement Pavement, Connecticut Department of Transportation Report No. 646-1-80-12, September, 1980, 47p.

8. Heck, W.J., Study of Alkali-Silica Reactivity Tests to Improve Correlation and Predictability for Aggregates, in Cement, Concrete, and Aggregates, Vol 5, no 1, Summer 1983, pp 47-53.

9. Bergren, J.V. and R.A. Britson, Portland Cement Concrete Utilizing Recycled Pavement, FHWA-DP-47-1, January 1977, 35 p.

10. Huisman, C.L. and R.A. Britson, Recycled Portland Cement Concrete "Specifications and Quality Control", in Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, FHWA-TS-82-208, December 1981, pp 140-143.

	Recycled Material Chert Concrete	 Limestone Concrete	Natural Material Chert Gravel	 Crushed Limestone
Coarse Friction Absorption SSD Sp. Gravel	 4.0 - 4.3 2.43-2.44	 3.9 2.52	 2.6 2.52	 0.8 2.67
			Sand	
Fine Fraction Absorption SSD Sp. Gravel	 7.6 - 9.0 2.36	 - -	 0.4 2.63	

Table 1. Properties of Crushed Concrete and Natural Aggregates (After Buck, 1973).

	Recycled Material Once Recycled	 Twice Recycled	Natural Material Gravel
Coarse Fraction Absorption Bulk Sp. Gravel	 3.43 - 5.0 2.31 - 2.40	 8.36 2.11	 1.02 2.67
Fine Fraction Absorption Bulk Sp. Gravel	 7.17 - 8.31 2.15 - 2.23	 - -	 1.38 2.60

Table 2. Properties of Crushed Concrete and Natural Aggregates (After Fergus 1981).

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: Technical Paper - An Overview of Surface Rehabilitation Techniques for Asphalt Pavements

Date: APR 6 1992

From: Chief, Pavement Division

Reply to Attn of: HNG-42

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

During the past year, the Pavement Division, in conjunction with the Office of Technology Applications, has been involved in a comprehensive effort to develop an information base on existing and emerging surface rehabilitation techniques for asphalt pavements. Examples of techniques we are evaluating include: (1) cold mixtures such as slurry seals and micro-surfacing; (2) single and multiple chip seals; and (3) open and dense graded thin hot-mix overlays. The use of modified binders and fibers in these applications will also be examined. This project will provide information on the usage, design, construction, cost, and anticipated performance of these techniques when applied as a functional improvement to a structurally sound higher volume roadway pavement. Further, this project will complement and expand on the information gained from the Strategic Highway Research Program's specific pavement studies (SPS-3) experiment.

Attached are copies of the technical paper entitled, "An Overview of Surface Rehabilitation Techniques for Asphalt Pavements," (FHWA-PD-92-008). You may wish to provide copies of this paper to your division offices. This paper summarizes known preventative maintenance and surface rehabilitation techniques based on our literature search and some limited field work. During the coming months, we will be visiting several existing and new projects to gather additional related information on various applications. Your staff assistance in this regard will be appreciated.

If you have any questions on our effort or like to arrange for a presentation on this subject, please call Messrs. Hassan Raza at FTS 366-1338 or James Sorenson at FTS 366-1333.

Louis M. Papet

Attachments

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: ACTION: Distribution of Publication

Date: July 12, 1994

From: Director, Office of Engineering
Director, Office of Technology Applications

Reply to Attn of: HNG-42

To: Regional Administrators
Federal Lands Highway Program Administrator

The attached publication, State of the Practice Design, Construction, and Performance of Micro-surfacing (FHWA-SA-94-051) provides a comprehensive discussion on an emerging surface rehabilitation technology. Sufficient copies of this publication are attached for your use and further distribution to the division offices and States within your region. Copies have also been distributed to each of the LTAP Technology Transfer Centers. Additional copies are available in limited supply from the Research and Technology Report Center, HRD-11, 6300 Georgetown Pike, McLean, Virginia 22101-2296 (telephone 703-285-2144).

Micro-surfacing consists of polymer-modified asphalt emulsion, crushed aggregate, mineral filler, water, and field-controlled additives as needed. Micro-surfacing is primarily used to seal existing surfaces, improve surface friction, and fill wheel ruts on both moderate and high volume roads. When properly designed and constructed, micro-surfacing has shown promising results with several years of service life. This surface rehabilitation technique has also been used effectively on portland cement concrete pavements to improve surface friction or address mechanical wear in the wheel paths.

This state-of-the-practice paper is a result of a joint effort by the offices of Engineering and Technology Applications, and the industry to develop information on existing and emerging surface rehabilitation techniques for asphalt pavements. The first product of this effort, An Overview of Surface Rehabilitation Techniques for Asphalt Pavements (FHWA-PD-92-008) was developed and distributed in April 1992. Presentation slides for both of the above papers will be available later this fall.

FEDERAL HIGHWAY ADMINISTRATION 1893 1993

In a related effort, an Office of Engineering memorandum dated June 24 announced the availability of warranty guide specifications for micro-surfacing projects on the National Highway System under Experimental Project 14. If you have any questions or would like to request technical support in the surface rehabilitation area, please call Hassan Raza at 202-366-1338.

William A. Weseman

Ray G. Griffith
FOR: Director, Office of
Technology Applications

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject INFORMATION: SP204 - Retrofit Load Transfer

Date FEB 10 1994

From Chief, Pavement Division
Chief, Engineering Applications Division

Reply to Attn of HNG-42
HTA-21

To Regional Federal Highway Administrators
Division Federal Highway Administrators
Federal Lands Highway Program Administrator

Attached are the following documents for your use and information:

1. Current status report - **SPECIAL PROJECT 204** - Retrofit Load Transfer and December 27, 1993 report Retrofit Load Transfer in Jointed Concrete Pavements
2. TRB Preprint 940247, Linda M. Pierce, PCCP Rehabilitation in Washington State (A Case Study)
3. Inspection report by Lynn Porter and Cathy Nicolas on Washington State Load Transfer Retrofit Project
4. Report by Roger Larson of load transfer restrofit field visits in Puerto and Indiana

Until recently, load transfer retrofit had been used only experimentally in the continental United States. In the last ten years, an estimated 300 lane Km of faulted or cracked undoweled jointed plain concrete pavement (JPCP) has been successfully rehabilitated in Puerto Rico. Based on the generally good performance of previously constructed load transfer retrofit experimental sections in the U.S. and the outstanding performance in Puerto Rico, SP-204 was initiated to encourage the development of equipment to construct multiple slots in each wheelpath to increase the production rate for this technique and to reduce the construction cost and road user delays.

Attachment 1 describes the current status and background of this effort. Attachment 2 describes the preliminary engineering and experimental test section construction that led to the 53 km project now underway in Washington State. Attachment 3 describes the major Washington State project currently underway involving 53 km (about 24 km now complete) of retrofit load transfer on eastbound I-90. Attachment 4 describes field visits to Puerto Rico to observe the long term performance of retrofit load transfer projects and to Indiana to observe a demonstration of the feasibility of using carbide milling technology to construct multiple slots in jointed reinforced concrete pavement (JRCP).

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Based upon the recent construction of 24 lane km of retrofit dowels (JPCP) in the project currently underway in Washington State and the successful demonstration of milling three slots per wheelpath in one pass on working cracks in a JRCP ramp in Indiana, equipment is now available to economically construct retrofit load transfer at joints or cracks in existing jointed concrete pavements. The bid price to construct retrofit load transfer devices in Washington State was $34.50 per dowel installed (62,000 38 mm dowels in 64 mm wide slots). The average bid price in Puerto Rico is $20 per dowel installed (25 mm dowel in 40 mm wide slots) where this has been done routinely for ten years (slots sawed individually).

This technique should be used with other concrete pavement restoration techniques to rehabilitate existing jointed concrete pavement before serious deterioration is present. Perhaps the most cost-effective initial application of this technique would be to restore load transfer at working cracks developing in under-reinforced JRCP in other wise good condition. If performed early, it would also provide a cost-effective extension of the service life at the joints on undoweled JPCP and at transverse cracks without serious deterioration in either doweled or undoweled JPCP. If serious deterioration is present, full depth patching and/or selective slab replacements should be performed instead.

When properly applied, this technique will result in a cost-effective extension of the service life of existing jointed concrete pavements in good to fair condition. This technique would also be a very effective routine and preventive maintenance technique to reduce the cost and user delays during repairs of working cracks shortly after they develop and before full depth patches or slab replacements become necessary.

If you have comments or questions, please contact Mr. Roger Larson, the project manager of SP 204, at (202) 366-1326. A Technical Working Group will be formed shortly to update guidance reflecting the new equipment developments and other critical technical issues to help ensure success of this promising technique.

Theodore R. Ferragut　　　　Louis M. Papet

4 Attachments

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: ACTION: ISTEA Section 6005
Thin Bonded Overlay and Surface Lamination
Pavements and Bridges
Reply due: October 31, 1994

Date: July 1, 1994

From: Director, Office of Engineering

Reply to Attn of: HNG-32
HNG-42

To: Regional Federal Highway Administrators

We are requesting applications for additional projects for the Thin Bonded Overlay and Surface Lamination (TBO) Program, which is part of the Applied Research and Technology (ART) Program established by Section 6005 of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991. A summary of the TBO program and the application procedures are described in Attachment A. The application form is included as Attachment B. A summary of information on technologies is included in Attachment C. A listing of bridge deck and pavement overlay projects and TBO technologies previously approved is included in Attachment D and the evaluation plans developed for these projects are included in Attachment E.

Additional projects are being sought for available fiscal year (FY) 1994 and 1995 funding. Projects proposed for construction in FY 1996 and 1997 are also encouraged. There may be no future solicitations for ISTEA TBO projects if enough candidate projects are available for selection from responses to this request. Please contact the States in your region for candidate projects for the TBO program. Candidate projects proposed by the State highway agencies must be submitted on the application form (Attachment B) and sent with any supporting information to the appropriate Federal Highway Administration Division Office by October 14, 1994, for forwarding to this office by October 31. The Section 6005 funding provided (100 percent for reporting and evaluation and 80 percent for construction and an equal amount of obligation authority for projects approved as a part of this solicitation) is in addition to the individual State's regular Federal-aid. Please also note that priority for funding will be given to the technologies listed in the *New Projects Sought* section of Attachment A.

Your cooperation and attention are greatly appreciated. If you have any questions or comments, please contact Mr. Vasant Mistry, HNG-32, (202) 366-4599 or Mr. Roger Larson, HNG-42, (202) 366-1326. General questions on the ART Program should be addressed to Mr. Richard A. McComb, HTA-2, (202) 366-2792.

for William A. Weseman

5 Attachments

CHAPTER 8

SURFACE AND OTHER CONSIDERATIONS

8.1 Rideability Specifications, December 17, 1987.

8.2 A Selection of Measuring Equipment Used to Monitor and Enforce Rideability Specifications, Technical Paper 88-03, May 24, 1988.

8.3 TA 5040.17, Skid Accident Reduction Programs, December 23, 1980.

8.4 TA 5140.10, Texturing and Skid Resistance of Concrete Pavement and Bridge Decks, September 18, 1979.

8.5 TA 5040.31, Open-Graded Asphalt Friction Course, December 26, 1990.

8.6 Automatic Profile Index Computation, February 21, 1991.
- Analysis and Recommendations Concerning Profilograph Measurements in South Dakota, November 1990.

8.7 Measurements, Specifications, and Achievement of Smoothness for Pavement Construction, NCHRP No. 167, 1990.

8.8 A Half Century with the California Profilograph, Report Number FHWA-AZ-SP9102, February 1992.

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: Rideability Specifications

Date: DEC 17 1987

From: Director, Office of Highway Operations

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Direct Federal Program Administrator

Smoothness has become the primary measure by which the traveling public determines and evaluates the quality of both newly constructed and rehabilitated pavements. In the spring of 1987, the Pavement Division assisted the Rideability Task Force of the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Construction by developing and compiling a survey on rideability specifications being used by the States. We have attached a copy of the draft survey summary report for your use.

Based on the survey results, the task force proposed a revised PCC rideability specification and a new AC rideability specification at the Subcommittees mid-year meeting. These specifications were balloted on in the fall of this year and received a two-thirds majority approval. They were subsequently approved by both the Standing Committee on Highways and the Executive Committee of AASHTO at their December meeting and will be incorporated into the new 1988 AASHTO Guide Specifications for Highway Construction when published. Copies of the specifications as balloted are attached for your information.

Key elements of both proposed guide specifications include:

1. The measurement device is limited to only the California-type profilograph.

2. Methods for the evaluation of both profiles and bumps are specified as California Test 526.

3. The acceptance level for the profile index was revised from 12 to 10 inches per mile using the California type profilograph with a 0.2 inch blanking band.

4. The minimum day's paving length for which a profile is taken is established as 0.1 mile, consistent with the evaluation length.

5. The size of the "must grind" bump was increased to 0.4 inch in 25 feet.

6. Two pay adjustment schedules are provided: 1) establishes disincentives only on a sliding scale to a maximum reduction of 10 percent at 15 ipm. 2) an optional schedule, establishes both incentives and the disincentives on a sliding scale of a maximum reduction of 10 percent at 15 ipm and a maximum increase of 5 percent at 3 ipm.

We believe that adopting rideability specifications pays significant dividends to all elements of the highway industry in not only providing smoother pavement surfaces, but providing a higher quality product. In addition, it has been demonstrated through the AASHTO design equations that there is a significant direct relation between initial pavement smoothness and design life. Therefore, we strongly recommend that you and your staff work with your respective divisions and States in developing and implementing rideability specifications for both Portland Cement Concrete and Asphalt Concrete. The specifications should generally follow the new AASHTO Guide Specifications. The Pavement Division is available to provide assistance at your request.

David S. Gendell

401.03 Asphalt Concrete

Surface Test. Method #1. The surface will be tested with a 10-foot straightedge at locations selected by the Engineer. The variation of the surface from the testing edge of the straightedge between any two contacts, longitudinal or transverse with the surface shall not exceed inch (3/16 to 1/8 suggested). Irregularities exceeding the specified tolerance shall be corrected by and at the expense of the Contractor by removing the defective work and replacing it with new material or by an overlay (not patching), or by grinding/cold milling as directed by the Engineer. Following correction, the area shall be retested to verify compliance with the specified tolerances.

Profilograph Surface Test. Method #2. The smoothness of the pavement will be determined by using a profilograph over each designated lane. The surface of mainline pavement where the design speed will be 40 miles per hour (MPH) or higher will be tested and shall be corrected by the Contractor to a smoothness as follows.

If the final surface course is a friction course or other special purpose pavement layer, this specification, including corrective actions and pay adjustments, shall be applied to the pavement layer placed prior to the final surface course. The Contractor shall place the final surface course so the profile index of the final surface course is less than or equal to the profile index of the preceding pavement layer.

Equipment - The profile index will be determined using a California type profilograph furnished and operated by the Department. The profilogram is recorded on a scale of 1 inch, or full scale, vertically. Motive power may be manual or by a propulsion unit attached to the assembly. The profilograph will be moved longitudinally along the pavement at a speed no greater than 3 MPH to minimize bounce. The results of the profilograph tests will be evaluated as outlined in California Test 526.

Surface Test - The Contractor shall furnish paving equipment and employ methods that produce a riding surface having a profile index of 10 inches per mile or less, except as provided f: , in subsequent paragraphs. Initial profiles up to 15 inches per mile may be accepted with applicable Price Adjustments. The profile will terminate 15 feet from each bridge approach pavement or existing pavement that is joined by the new pavement.

Pavement profiles will be taken 3 feet from and parallel to each edge of pavement for pavement placed at a 12-foot width or less. When pavement is placed at a greater width than 12 feet, the profile will be taken 3 feet from and parallel to each edge and from the approximate location of each planned lane marking. Additional profiles may be taken only to define the limits of an out-of-tolerance surface variation.

During the initial paving operations, either when starting up or after a long shut-down period, the pavement surface will be tested with the profilograph as soon as the final rolling has been completed. Initial testing will be used by the Contractor and the Engineer to evaluate the paving methods and equipment.

If the initial pavement smoothness, paving methods, and paving equipment are acceptable to the Engineer, the Contractor may proceed with the paving operation. After initial testing, profiles of each day's paving will be run prior to continuing paving operations on prior to opening the pavement to public traffic.

A daily average profile index will be determined for each day's paving. A day's paving is defined as a minimum of 0.1-mile of full-width pavement placed in a day. If less than 0.1-mile is paved, the day's production will be grouped with the next day's production. If an average profile index of 15 inches per mile is exceeded in any daily paving operation, the paving operation will be suspended and will not be allowed to resume until the Contractor takes corrective action. In the event that paving operations are suspended as a result of the average profile index exceeding 15 inches per mile, subsequent paving operations will be tested in accordance with the initial testing procedures.

For determining pavement section where corrective work or pay adjustments will be necessary, the pavement will be evaluate in 0.1 mile sections using the profilogram. Within each 0.1-mile section, all areas represented by high points having deviations in excess of 0.4 inches in 25 feet or less shall be corrected by the Contractor. After correcting individual deviations in excess of 0.4 inches in 25 feet, corrective action shall be made to reduce the profile index to 10 inches per mile or less.

In addition, any 0.1 mile section having an initial profile index in excess of 15 inches per mile shall be corrected to reduce the profile index to 10 inches per mile or less.

On those sections where corrections are made the pavement will be tested to verify that corrections have produced a profile index of 10 inches per mile or less.

Corrective actions shall be made at the Contractors expense. All corrective work shall be completed prior to determining the pavement thickness. Corrections made by cold milling, by diamond grinding, by overlaying, or by removing and replacing, shall be as directed by the Engineer in accordance with the following:

(1) Cold Milling/Grinding

Cold Milling/grinding shall be performed by the Contractor until the required surface tolerances are achieved. Cold milling/grinding shall be performed so a uniform cross-section is produced. All milled areas shall be neat and of uniform surface appearance.

(2) Overlaying

Asphaltic concrete pavement overlays shall meet all the requirements specified in the Contract. The overlay lift shall extend the full width of the underlying pavement surface and have a finished compacted thickness sufficient to correct the roughness and produce a final surface meeting specified surface tolerances.

If the overlay does not meet the longitudinal smoothness requirement, a second overlay will not be allowed. The repairs to an overlay not meeting smoothness requirements shall be made by the Contractor as directed by the Engineer

(3) Removing and Replacing

Corrections made by removal shall be replaced by asphalt concrete pavement materials meeting the requirements specified in the contract.

Price Adjustments - When the profile index does not exceed 10 inches per mile per 0.1 mile section, payment will be made at the contract unit price for the completed surface course. When the profile index exceeds 10 inches per mile per 0.1 mile section but does not exceed 15 inches per mile per 0.1 mile section, the Contractor may elect to accept a contract unit Price Adjustment in lieu of reducing the profile index. Contract unit Price Adjustments will be made in accordance with the following schedule.

Profile Index Inches per mile per 0.1-mile section	Contract Unit Price Adjustment Percent of pavement unit bid price
Less than (10)	100
Over 10 to 11	98
Over 11 to 12	96
Over 12 to 13	94
Over 13 to 14	92
Over 14 to 15	90
Over 15	Corrective work required

This unit bid Price Adjustment will apply to the total theoretical tonnage representing the total thickness of the asphaltic pavement structure of the 0.1-mile-long section for the lane width represented by the profilogram.

The above Price Adjustment schedule will apply to pavement sections where corrective work has been completed.

Pay adjustments with incentives. Method #3. When the profile index is greater than 7 inches per mile but does not exceed 10 inches per mile per 0.1 mile section, pavement will be made at the contract unit price for the completed surface course. When the profile index exceeds 10 inches per mile per 1.0 mile section but does not exceed 15 inches per mile per 0.1-mile section, the Contractor may elect to accept a contract unit Price Adjustment in lieu of reducing the profile index. When the profile index is less than or equal to 7 inches per mile, the contractor will receive an incentive payment.

Contract unit price adjustments will be made in accordance with the following schedule.

Profile Index Inches per mile per 0.1-mile section	Contract Unit Price Adjustment Percent of pavement unit bid price
3 or less	105
Over 3 to 4	104
Over 4 to 5	103
Over 5 to 6	102
Over 6 to 7	101
Over 7 to 10	100
Over 10 to 11	98
Over 11 to 12	96
Over 12 to 13	94
Over 13 to 14	92
Over 14 to 15	90
Over 15	Corrective work required

Pay adjustments for incentives will only be based on the initial measured profile index, prior to any corrective work. The Price Adjustment schedule for 100 percent pay or pay reductions apply to pavement sections where corrective work has been completed.

This unit bid Price Adjustment will apply to the total theoretical tonnage representing the total thickness of the asphaltic pavement structure of the 0.1-mile-long section for the lane width represented by the profilogram.

501.03 Portland Cement Concrete

Surface Test. Method #1. The surface will be tested using a 10-foot straightedge at locations selected by the Engineer. The variation of the surface from the testing edge of the straightedge between any two contacts, longitudinal or transverse with the surface, shall not exceed 3/16 inch. Irregularities exceeding the specified tolerances shall be corrected by and at the expense or the Contractor with an approved profiling device or by other means as directed by the Engineer. Following correction the area will be retested to verify compliance with the specified tolerances.

Profilograph Surface Test. Method #2. The smoothness of the pavement will be determined by using a profilograph over each designated lane. The surface finish of mainline pavement where the design speed will be 40 miles per hour (MPH) or higher shall be tested and corrected to a smoothness as follows:

Equipment - The profile index will be determined using a California type profilograph finished and operated by the Department. The profilogram is recorded on a scale of 1 inch, or full scale, vertically. Motive power may be manual or by a propulsion unit attached to the assembly. The profilograph will be moved longitudinally along the pavement at a speed no greater than 3 MPH to minimize bounce. The results of the profilograph tests will be evaluated as outlined in California Test 526.

Surface Test - The Contractor shall furnish paving equipment and employ methods that produce a riding surface having a profile index of 10 inches per mile or less, except as provided for in subsequent paragraphs. Initial profiles up to 15 inches per mile may be accepted with applicable Price Adjustments. The profile will terminate 15 feet from each bridge approach pavement or existing pavement that is joined by the new pavement.

Pavement profiles will be taken 3 feet from and parallel to each edge of pavement for pavement placed at a 12-foot width or less. When pavement is placed at a greater width than 12 feet, the profile will be taken 3 feet from and parallel to each edge and from the approximate location of each planned longitudinal joint. Additional profiles may be taken only to define the limits of an out-of-tolerance surface variation.

During the initial paving operations, either when starting up or after a long shut-down period, the pavement surface will be tested with the profilograph as soon as the concrete has cured sufficiently to allow testing. Membrane curing damaged during the testing operation shall be repaired by the Contractor as directed by the Engineer. Initial testing will be used to aid the Contractor and the Engineer to evaluate the paving methods and equipment.

If the initial pavement smoothness, paving methods, and paving equipment are acceptable to the Engineer, the Contractor may proceed with the paving operation. After initial testing, profiles of each day's paving will be run prior to continuing paving operations.

A daily average profile index will be determined for each day's paving. A day's paving is defined as a minimum of 0.1-mile of full-width pavement placed in a day. If less than 0.1-mile is paved, the day's production will be grouped

The unit bid adjusted price will be computed using the planned thickness of portland cement concrete pavement. This unit bid adjusted price will apply to the total area of the 0.1-mile section for the lane width represented by the profilogram.

The above Price Adjustment Schedule will apply to pavement sections where corrective work has been completed.

Pay adjustments with incentives. Method #3. When the profile index is greater than 7 inches per mile but does not exceed 10 inches per mile per 0.1-mile section, payment will be made at the Contract unit price for the completed pavement. When the profile index exceeds 10 inches per mile per 0.1-mile section but does not exceed 15 inches per mile per 0.1-mile section, the Contractor may elect to accept a contract unit adjusted price in lieu of reducing the profile index. When the profile index in less than or equal to 7 inches per mile, the Contractor is entitled to an incentive payment. Contract unit Price Adjustments will be made in accordance with the following schedule in those cases when the Contractor is entitled to incentive payments or elects to accept contract unit Price Adjustments in lieu of reducing the profile index.

Index Profile Inches per mile per 0.1-mile section	Price Adjustment Percent of pavement unit bid price
3 or less	105
Over 3 to 4	104
Over 4 to 5	103
Over 5 to 6	102
Over 6 to 7	101
Over 7 to 10	100
Over 10 to 11	98
Over 11 to 12	96
Over 12 to 13	94
Over 13 to 14	92
Over 14 to 15	90
Over 15	Corrective work required

Pay Adjustments for incentives will only be based on the initial measured profile index, prior to any corrective work. The Price Adjustment schedule for 100 percent pay or pay reductions apply to pavement sections where corrective work has been completed.

The unit bid adjusted price will be computed using the planned thickness of portland cement concrete pavement. This unit bid will apply adjusted price to the total area of the 0.1-mile section for the lane width represented by the profilogram.

TECHNICAL PAPER 88-03

A SELECTION OF MEASURING EQUIPMENT USED TO MONITOR AND ENFORCE RIDEABILITY SPECIFICATIONS

I. INTRODUCTION —

There has been a large amount of information provided concerning the use and benefits of rideability specifications recently. This paper is to help FHWA engineers in the field become more familiar with the various types of equipment available to enforce these specifications. Consequently, the following information is provided to give the engineers an overview of the equipment characteristics, operational and calibration methods, and costs. We have tried to provide this information for the most commonly used devices and the inclusion or omission of any particular device should not be construed as an endorsement or disapproval.

Comparability of measurements from the different pieces of equipment is always a question. There have been a number of research studies done, and several underway to address this question. So far, it is sufficient to say that correlations have been developed between several of the following equipment types. Many of these studies contain restrictions and/or limitations which should be understood before using the correlations.

II. PROFILOGRAPHS —

Traditional California Type Profilograph — characterized by an aluminum truss frame 25 feet long. At each end is mounted a wheel assembly consisting of six caster wheels. In the middle of the frame a profile wheel is mounted directly under and connected to a graphic recorder. See figure 1. The truss breaks down into three major components plus the graphic recorder. This allows easy transportation in a standard sized pickup truck or van. Assembly requires two persons about 15 minutes. The cost of the traditional California Type Profilogragh ranges from about $12,000 to over $25,000 for a model which includes a microprocessor for data reduction. This type profilograph is manufactured by James Cox & Sons, Inc., Mc Cracken Concrete Pipe Co., Mc Beth Engineering Corp., and Thompson-Quill Assoc.

Ames California Type Profilograph — this device is characterized by an aluminum box beam 25 feet long as the reference plane. Mounted at each end of the beam is a wheel assembly the same as the traditional California type device, with six caster wheels. The Ames profilograph has both a bicycle wheel and the graph recorder assembly mounted at the rear of the device. This wheel drives the graph recorder and is not involved in recording the vertical deviations. The Ames device uses a six-inch caster wheel mounted mid beam as the profile measuring wheel. See figure 2. As with the traditional California type this device also brakes down into three major components plus the graph recorder assembly for easy transportation. The type of profilograph costs about $7,000 and is manufactured by Ames Profilograph.

Rainhart Profilograph — this device is characterized by an aluminum frame which has four triangular subframes mounted equidistant along it. Each of the

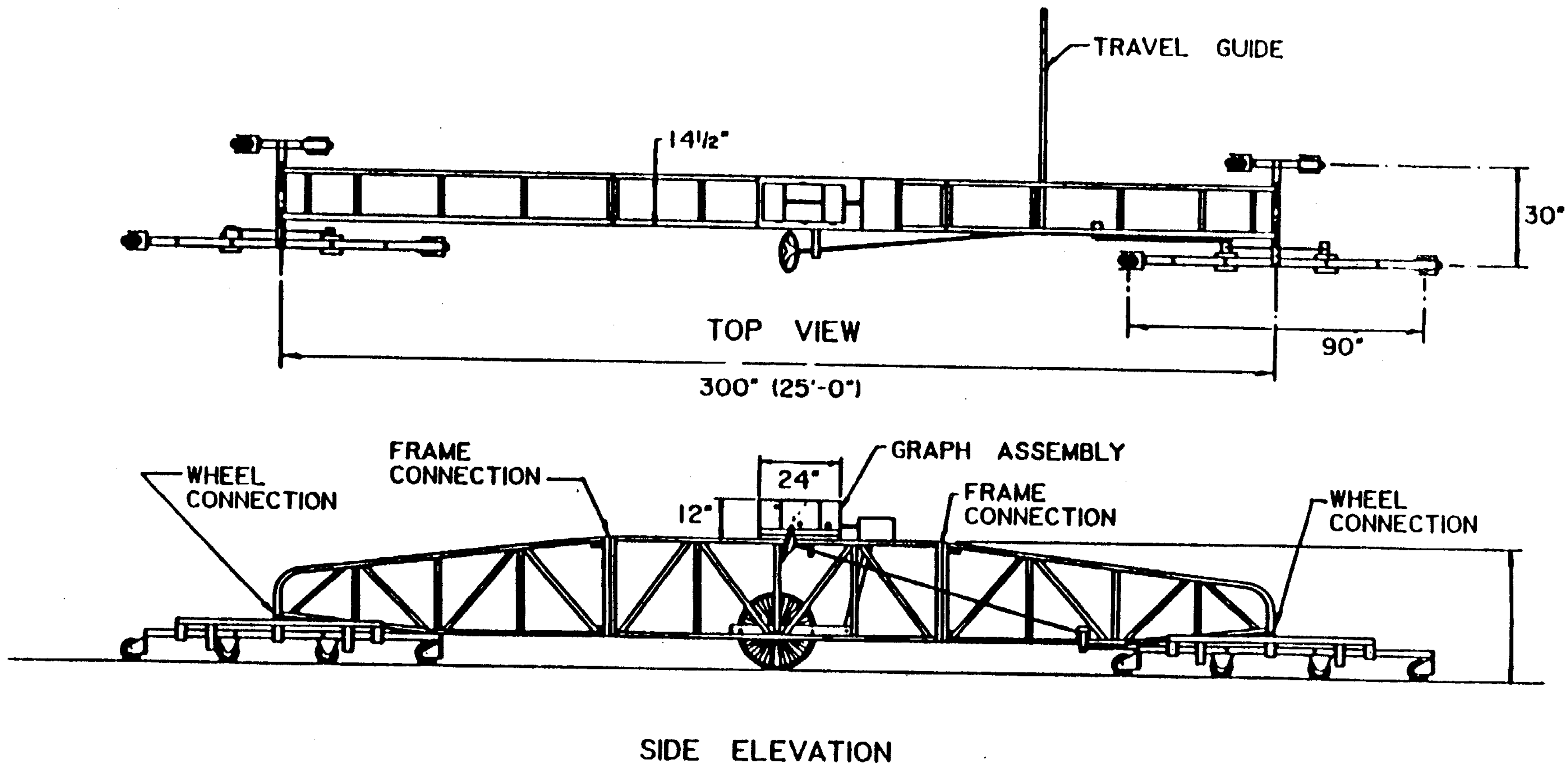
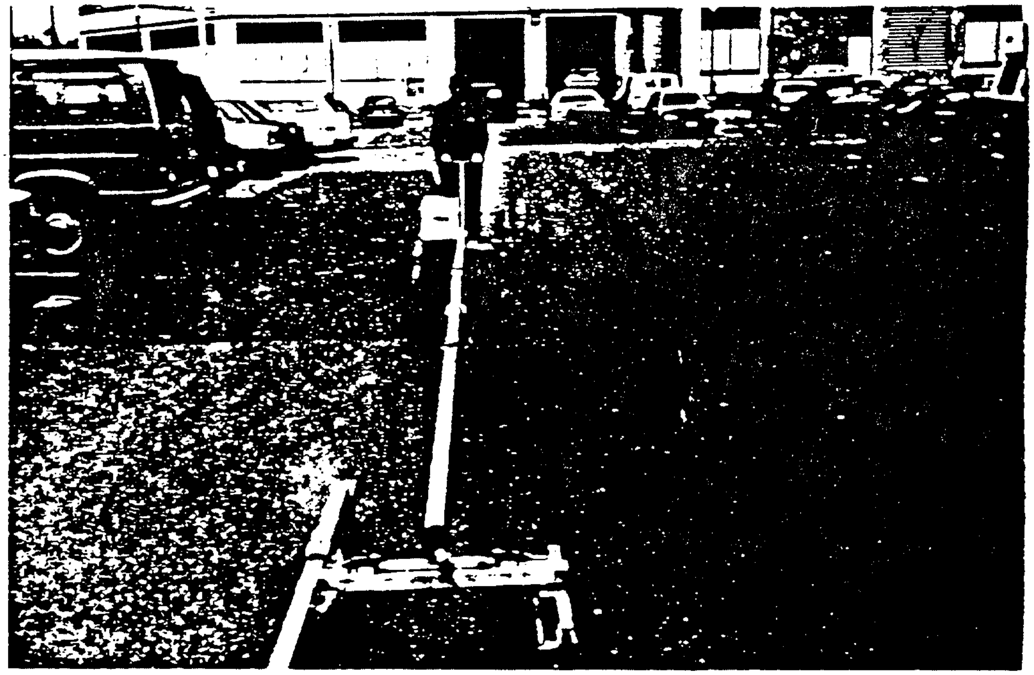
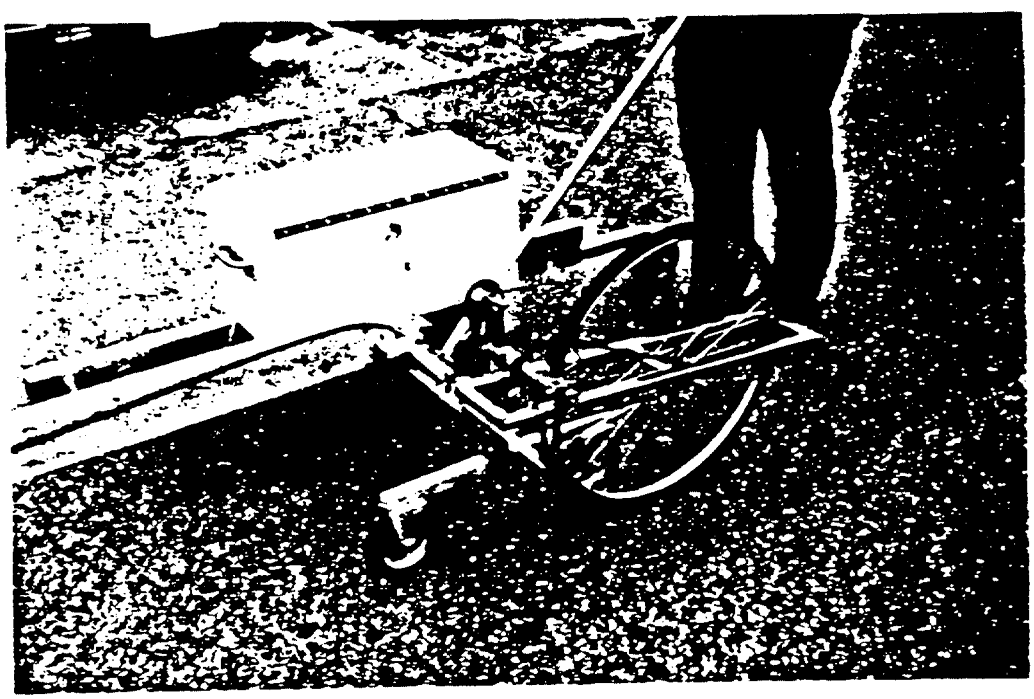


Figure 1. Schematic of California type profilograph.

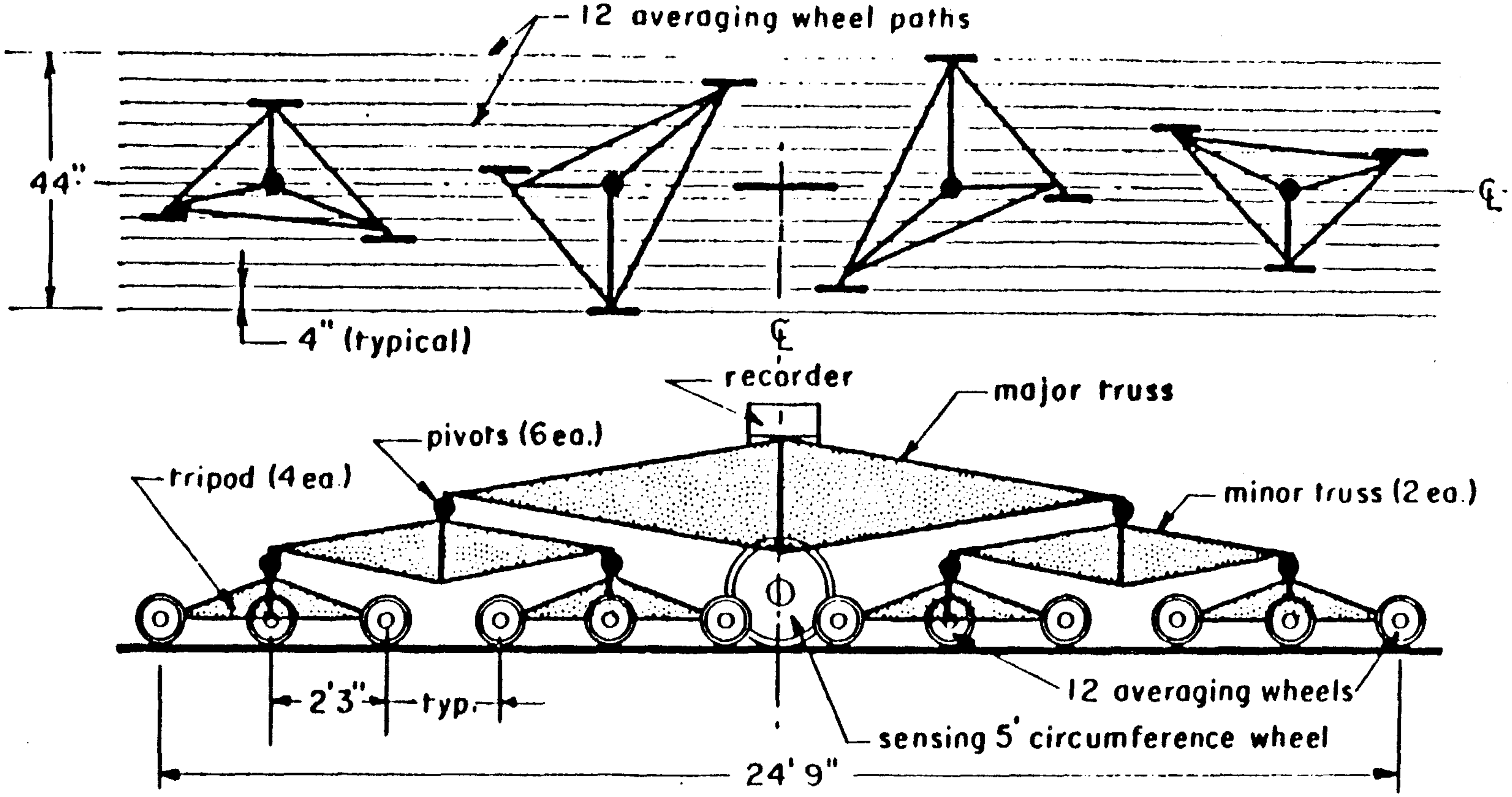


Front View Ames California Profilograph



View of recording box
Ames California Profilograph

Figure 2



Any averaging wheel lifted 1" = tripod apex up $\frac{1"}{3}$ = minor truss ℄ up $\frac{1"}{6}$ = major truss ℄ (recorder) up only $\frac{1"}{12}$ (0.083")

Figure 3 Schematic of Rainhart Profilograph subframes has three wheels mounted such that no two wheels follow the same path. The Rainhart device has a profile wheel mounted directly under and connected to a graphic recorder in the middle of the frame. See figure 3. The Rainhart Profilograph does not break down into subassemblies for transportation as do the other profilographs. Instead, the Rainhart provides an auxiliary wheel assembly which locks down making the device into a trailer for ready transportation. This device is manufactured by the Rainhart Company and costs about $11,000.

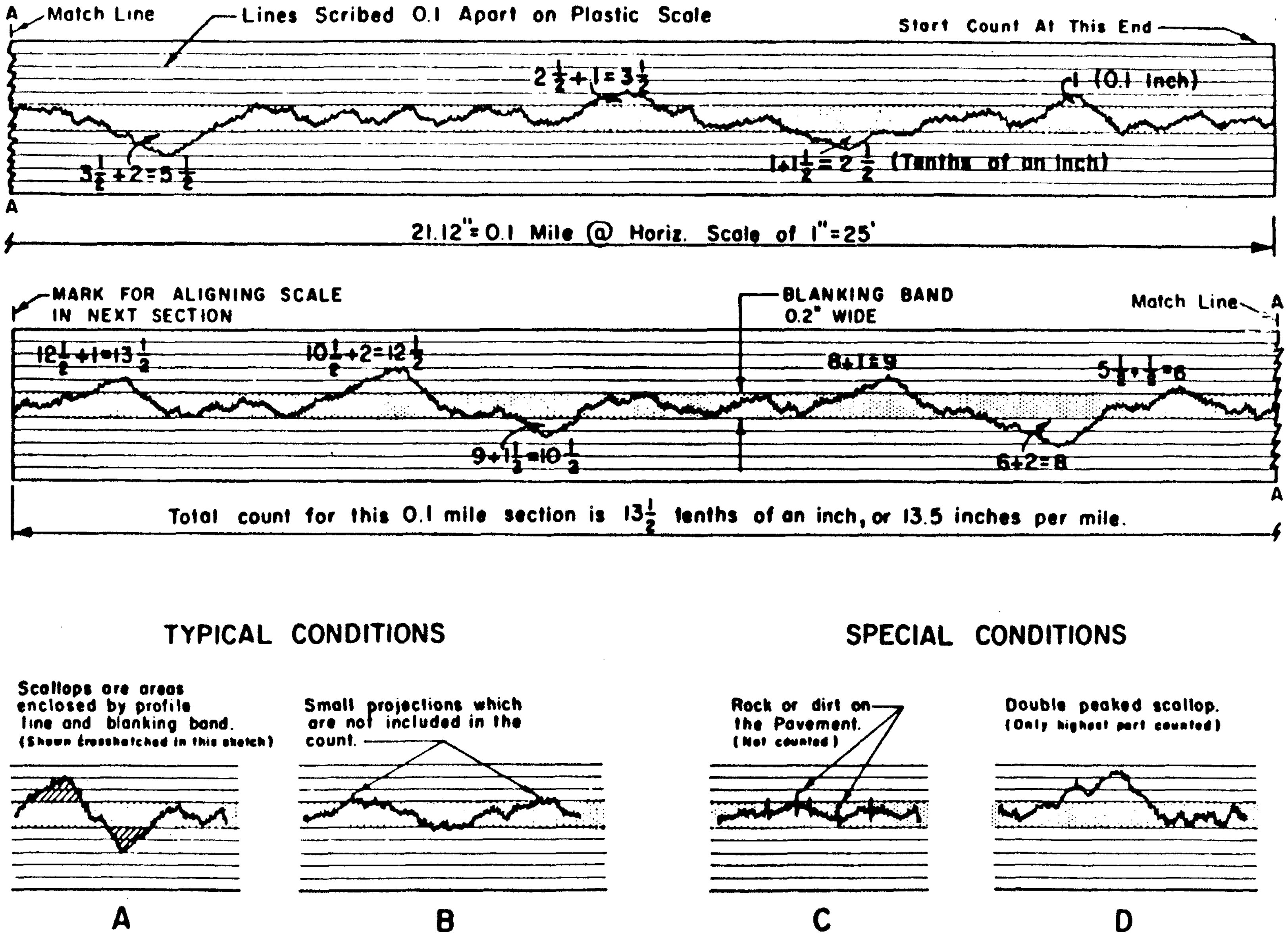
The calibration, operation, and methodology for reducing the data is generally the same for all the profilographs, we will address the details generically.

Calibration of the profilograph is relatively simple because they are simple machines where little can go wrong. There are two basic calibrations. One is the horizontal distance and the other is the vertical deviations. The horizontal distance is measured by the bicycle tire rolling along the pavement. This movement is transferred to the graph recorder by a standard bicycle chain. The only adjustments are in tire pressure, tire trueness, and a some models allow adjustments to the gearing at the graph recorder. To calibrate horizontally, a known distance should be measured with the profilograph. If the measured distance does not correspond to the known distance, adjustments should be made. Vertical deviations in the pavement are measured by the wheel mounted in the center of the profilograph raising and lowering. This movement is transferred to the pen in the graph recorder by cables. There is usually little adjustment available to the cable mechanism. To calibrate vertically, blocks of known height are placed under the recording wheel and the pen in the graph recorder checked to ensure a like height is recorded.

A two person crew is required to measure a profile of the roadway. One person pushes and steers the profilograph while the other marks events on the graph recorder and ensures the profile is being properly recorded. Because these devices are only able to operate at low speeds, i.e., 2-3 miles per hour, extreme care must be exercised while operating in traffic. Some form of traffic control must be an element of the normal operation if the roadway is open to traffic. Most specifications require that the profilograph measure the roughness in the wheel paths, with the measurements of the two wheel paths being averaged for the lane. Events, such as mileposts, structures etc. and the direction of travel should be recorded on the graph.

The data reduction process or trace evaluation is an activity which requires training in order to ensure repeatability. A special blanking band is placed over the trace so that a maximum amount of trace is covered by the 0.2 (0.1 recommended for the Rainhart profilograph) inch opaque band. A nearly equal amount of deviations will show above and below the opaque band when the blanking band is positioned correctly. See figure 4. The height of each deviation is determined to the nearest 0.05 inch. The value is recorded above each deviation. Multi-peaked deviations are considered only once with the highest peak measured. Deviations with no width (spikes) represent chunks of mortar, rocks, texture or wheel bounce and are not counted. The sum of the deviation heights is counted and divided by the distance. The profile index is expressed in inches per mile. Profile Index = total count (inches)/length of profile (miles).

EXAMPLE SHOWING METHOD OF DERIVING PROFILE INDEX FROM PROFILOGRAMS



ıre 4

Studies have shown that the traditional California type and the Ames device produce a tracing which is virtually identical. Because of the different wheel configuration on the Rainhart, it produces a different looking tracing when compared to the other two devices. The results of the California type profilographs cannot be directly correlated to that of the Rainhart device.

The American Society of Testing and Materials (ASTM) is in the process of developing a standard test method of measuring pavement roughness using a profilograph. We will provide copies of the standard when available.

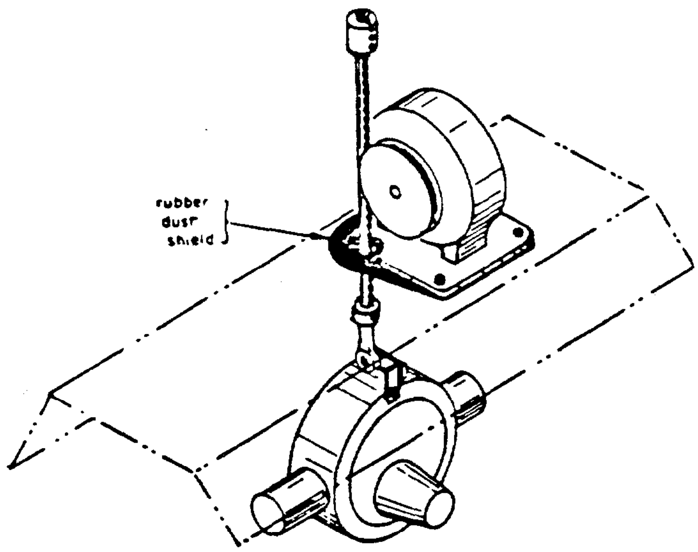
III. RESPONSE TYPE ROAD ROUGHNESS MEASURING (RTRRM) DEVICES —

Response type road roughness measuring devices operate at highway speeds. These devices are mounted in a vehicle (trailer) and measure the response (bounce) of the vehicle to the road roughness. It is therefore not a true measurement of roughness. Included in this category of equipment are devices that measure the relative axle-body motion and devices that measure the acceleration of the axle or the vehicle body.

Mays Ride Meter — this device determines the roughness of the roadway by measuring the displacement between the axle housing and the body of the test vehicle. The method actually measures the relative motion of a sprung mass system in response to traveled surface roughness where the mass is supported by automotive type suspension and tires. Their are other types of response devices, but the Mays Ride Meter is by far the most commonly used today.

The major components of the Mays Ride Meter is the rotary transducer, the pavement condition recorder, and the distance measuring instrument. The rotary transducer converts the axle/body movement to an electrical signal. The distance measuring instrument is an electronic odometer. The pavement condition recorder is a microprocessor which accepts input from the rotary transducer, the distance measuring instrument, and a keyboard processes the various signals into an output. This output is commonly in the form of accumulated inches of relative motion over a distance.

The Mays Ride Meter can be mounted in either a standard passenger car or a trailer. Experience has shown that many characteristics in a passenger car can affect the roughness reading. Consequently, it is now common to mount the Mays Ride Meter components in a trailer. See figure 5. However, there are still items within the trailer which can cause potential inaccuracy. These items include: shock absorbers, tire roundness, tire balance, tire pressure, condition of springs, loose wheel bearings, play in tow/sway bar assembly, and wind. It has also been found that the roughness readings are temperature sensitive, therefore readings should not be made at temperature extremes. The trailer must receive preventative maintenance on a regular basis. The tires must be trued and balanced. Wheel bearing should be checked and periodically repacked. Tire pressure should be checked several times a day and maintained at a constant hot pressure. It is also very important to assure that the shock absorbers be maintained in good condition and replaced only with a standard type. Also new shock absorbers should be subjected to a break in period and should be warmed up daily for a prescribed distance. Another factor affecting



Detail of Mays Ride Meter

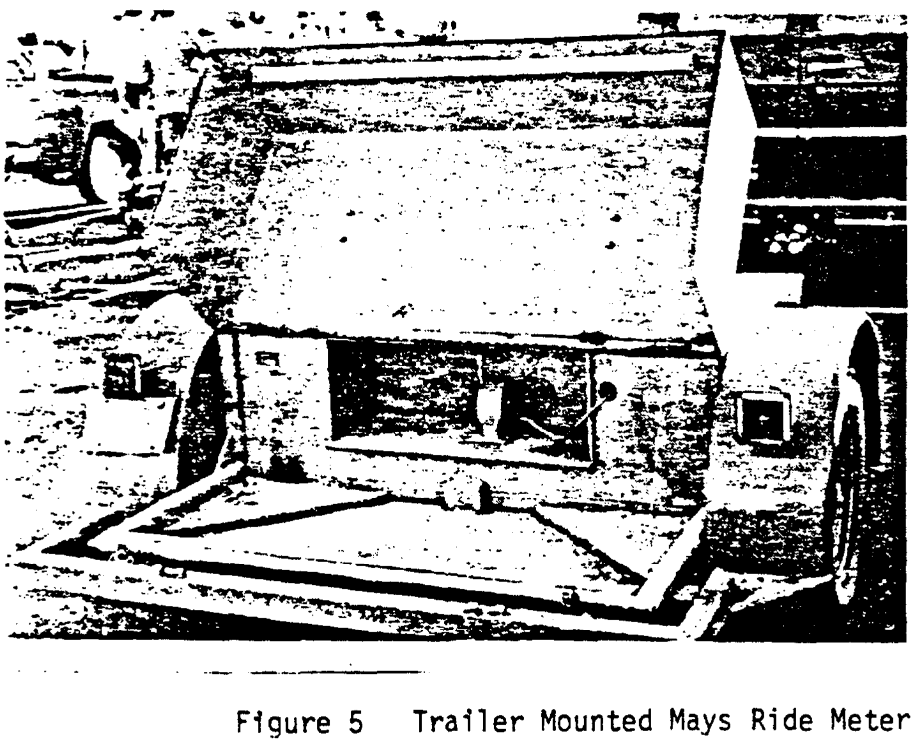


Figure 5 Trailer Mounted Mays Ride Meter

repeatability is the speed at which the test is run. It is very important to maintain the vehicle at a constant speed. A speed of 50 miles per hour is commonly used in rural areas, but the device can be operated at slower speeds. Calibration tests must be done at all speeds at which roughness measurements will be recorded.

Most users of the Mays Ride Meter use extensive calibration procedures to increase the potential for repeatability of each Ride Meter and the comparability between two different Ride Meters. One method of calibration (Highway Performance Monitoring System Field Manual) consists of both a periodic control check to assure the components of the device are operating properly and an annual calibration based on a number of control sections. These control sections should be approximately 0.2 mile in length and be relatively straight and level with a low ADT. The low ADT is important because to use in-service control sections the assumption has to be made that the roughness measurements do not significantly change during a years time. It is extremely important to recalibrate the device anytime a component is replaced or repaired. Further information is available in Appendix J of the HPMS Manual.

The cost of the May Ride Meter Trailer and associated strip chart recorder manufactured by the Rainhart Company is about $8,000. The price of the Pavement Condition Recorder is about $10,000. The operation requires a two man crew and costs about $40.00 per hour including the cost of the tow vehicle.

ASTM Standard Test Method E1082-85, Measurement of Vehicular Response to Traveled Surface Roughness is available to more fully discuss the details of the equipment and its operation.

IV. ROAD ROUGHNESS PROFILING DEVICES —

Profiling devices measure and record the longitudinal profile in one or both wheel tracks. In the United States the inertial type profiling devices are used. Devices in this category of equipment include the K.J. Law profilometer, the FHWA PRORUT system and the South Dakota profiling device.

Inertial Profilometer — these devices are capable of measuring and recording road surface profiles at speeds between 10 and 55 miles per hour. The devices utilize the inertial reference concept which was developed in the early 1960's at the General Motors Corporation Research Laboratories.

The profilometer measures and computes the longitudinal profile of the pavement through the creation of an inertial reference by using accelerometers placed on the body of the measuring vehicle. Relative displacement between the accelerometers and the pavement surface is measured with a non-contact light or acoustic measuring system mounted with the accelerometer on the vehicle body.

Operation requires a two person crew, one as a vehicle driver and the other as a system operator. The entire system is mounted in a full size van. See figures 6 and 7. The profile computer, data recording and other system components are all contained in the vehicle. The profilometer contains non-

Figure 6 K.J. Law 690 DNC Profilometer

Figure 7 FHWA PRORUT System

contact sensors for measuring road surface profile. The accelerometers establish the reference plane for the profilometer system's measurement by measuring the vertical accelerations of the vehicle body. The distance traveled by the system is measured with a distance encoder. This is usually a pulse type distance measuring device which is mounted to the front wheel of the vehicle.

The profile signal processing is performed by a digital computer which is mounted in the vehicle. Profile computations are performed in real time as the vehicle is driven down the road. Interface between the user and the profilometer system is provided through a system terminal and printer.

Vehicle response simulation programs for roughness index calculations are available with the profilometer system. The selected roughness index is normalized to read inches per mile and is printed out on the system printer. The roughness indices are simulations of standardized response type devices performed by the profilometer computer system and computed from the measured and recorded pavement profile data. The simulations can be used to calibrate response type equipment like the Mays Ride Meter or the roughness indices values can be used as the measured roughness statistic.

The cost of a Non-Contact Inertial Profilometer varies with the level of precision. The K.J. Law profilometer is the most precise device and is commercially available from K.J. Law Engineers for $250,000 to $300,000. The FHWA PRORUT system measures the profile in both wheel tracks and the average rut depth. To date, one prototype of the PRORUT has been constructed for a cost of $100,000 to $150,000. The South Dakota device us inexpensive ultrasonic sensors which are less precise. It measures the profile in one wheel path and the average rut depth. The South Dakota device is estimated to cost $50,000.

ASTM Standard Test Method E950-83, Measuring the Longitudinal Profile of Vehicular Traveled Surface with an Inertial Profilometer is available to more fully discuss the details of the equipment and its operation.

Additional information is available from Bruce E. Matzke, HHO-12, at 366-1342.

HHO-12/5-24-88

PARTIAL LISTING OF

EQUIPMENT MANUFACTURERS

Profilographs:

James Cox & Sons
PO Box 674
Colfax, California 95713

McCraken Concrete Pipe Company
PO Box 1708
Sioux City, Iowa 51102-1708

Rainhart Company
PO Box 4533
Austin, Texas 78765

Ames Profilograph
200 Rockwell Avenue
Ames, Iowa 50010

Response Type Road Roughness Measuring Devices:

Mays Ride Meter
Rainhart Company
PO Box 4533
Austin, Texas 78765

Road Roughness Profiling Devices:

K.J. Law Engineers, Inc.
23660 Research Drive
Farmington Hills, Michigan 48024

Mr. David Huff
South Dakota Department of Transportation
700 Broadway Avenue East
Pierre, South Dakota 57501-2586

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

SUBJECT	FHWA TECHNICAL ADVISORY
SKID ACCIDENT REDUCTION PROGRAM	T 5040.17 December 23, 1980

1. PURPOSE. To provide guidance for State and local highway agencies in conducting skid accident reduction programs.

2. BACKGROUND

a. This Technical Advisory provides a general overview of factors that should be considered as elements of any Skid Accident Reduction Program. This Technical Advisory supports current Federal Highway Administration (FHWA) policy and will be revised as appropriate to reflect changes in policy as they occur.

b. The existing requirements for skid resistance pavements are contained in several documents including Highway Safety Program Standard No. 12, Highway Design Construction and Maintenance (23 CFR 1204.4), Federal Highway Program Manual (FHPM) 6-2-4-7, Skid Measurement Guidelines for the Skid Accident Reduction Program. Other sources of technical advice are cited in the appropriate sections of this Technical Advisory.

c. Highway Safety Program Standard 12 (HSPS No. 12) states that every State shall have a program of highway design, construction, and maintenance to improve highway safety. This program shall provide that "there are standards for pavement design and construction with specific provisions for high skid resistance qualities." The HSPS No. 12 also requires that each State have a "program for resurfacing or other surface treatment with emphasis on correction of locations or sections of streets and highways with low skid resistance and high or potentially high accident rates susceptible to reduction by providing improved surfaces." In discharging the responsibilities of FHWA, the Division Administrator

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OPI: HHS-12

should determine the acceptability of specification requirements and construction practices for placing, consolidating, and finishing both asphalt concrete and portland cement concrete pavements. Such determinations will rely on the highway agency to research, evaluate, and document the performance of the various aggregates, mix designs, and construction practices used.

d. Even though the use of studded tires is beyond the control of most highway agencies, their use can cause significant wear on the pavement surface texture. For example, grooves sawed in concrete pavements have worn completely down in as short a time as 2 years. States are encouraged to ban or restrict the use of studded tires.

e. Legislative actions in recent years support a general duty of any highway agency to "... maintain the roadway in a reasonably safe condition. This would involve, in essence inspection, anticipation of defects, and conformity with generally accepted standards and practices."* The practical result is that highway agencies should have an organized system to identify and correct hazardous locations in a cost-effective manner, as well as a comprehensive pavement management program to design, construct, and maintain highways in conformance with reasonable standards. Such a systematic process is the best way to execute the highway agency's duty to maintain a reasonably safe roadway.

3. <u>SKID ACCIDENT REDUCTION PROGRAM</u>. Each highway agency is encouraged to develop and manage a skid accident reduction program to reflect the individual needs and conditions within the State. The purpose of a skid accident reduction program is to minimize wet weather skidding accidents through: identifying and correcting sections of roadway with high or potentially high skid accident incidence; ensuring that new surfaces have adequate, durable skid resistance properties; and utilizing resources available for accident reduction in a cost-effective manner. A program comprised of at least the following three basic activities, if faithfully implemented, should enable the highway agency to comply with HSPS No. 12.

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* Engineering and Government Liability, David C. Oliver, FHWA, an unpublished paper presented to the American Road and Transportation Builders Association Local Officials Meeting, St. Louis, Missouri, August 23, 1978.

a. The evaluation of pavement design, construction, and maintenance practices through its pavement management program to ensure that only pavements with good skid resistance characteristics are used.

b. The detection and correction of locations with a high incidence of wet weather accidents utilizing (1) the State and local accident record systems, and (2) countermeasures for locations with high wet weather incidences, to ensure that existing highways are maintained in a safe condition.

c. The analysis of skid resistance characteristics of selected roadway sections to:

(1) ensure that the pavements being constructed are providing adequate skid resistance,

(2) develop an overview of the skid resistance properties of highway systems,

(3) provide up-to-date information for the pavement management process, and

(4) provide data for use in developing safety improvement projects and the implementation of cost-effective treatments at appropriate locations.

4. PAVEMENT DESIGN, CONSTRUCTION, AND MAINTENANCE

a. Pavement Design

(1) Current pavement design practices should be evaluated to ensure that skid resistance properties are durable and suitable for the needs of traffic. Consideration of skid resistance levels, texture, aggregate availability, traffic volume, traffic speed, type of facility, rainfall, construction and maintenance practices, and accident experience are basic elements in such evaluations. Evaluations should document the compliance with the requirement for skid resistant surfaces and provide basic data for use in choosing corrective actions for locations with high wet weather accident rates.

(2) One principal result of the evaluations is the development of a performance history for each particular pavement used by each highway agency. The performance of the existing pavement designs

should be monitored and new designs should be evaluated to ensure that only skid resistant pavement surfaces are used. Information should be gathered as to the durability of a mix and the loss of skid resistance under traffic.

(3) The level of skid resistance needed for a particular roadway depends primarily on the traffic volume, traffic speed, type of facility, and climate with additional consideration warranted at special locations such as steep hills, curves, intersections, and other sites which experience high demands for pavement-tire friction. It is desirable to have one or more "skid resistant mixes" which have durable and higher than usual frictional properties for use in these special areas.

(4) A pavement surface may provide adequate skid resistance at low speeds, yet be inadequate for high speed conditions. Pavement surfaces, therefore, should be designed on the basis of properties at expected operating speeds.

(5) The American Association of State Highway and Transportation Officials (AASHTO) Guidelines for Skid Resistant Pavement Design, 1976, provide detailed information on the design of surfaces for both flexible and rigid pavements. The major considerations follow:

(a) Flexible Pavements

1 The skid resistance evaluation of bituminous pavements should include a determination that the aggregate used in the top layer of future pavements is capable of providing adequate skid resistance properties when incorporated in the particular mix and that the mix should be capable of providing sufficient stability to ensure the durability of the skid resistance.

2 A bituminous pavement surface should contain nonpolishing aggregates. It is essential for good skid resistance that a mix design be used which allows good exposure of the aggregates. This

requires that the pavement surface mixture be designed to provide as much coarse aggregate at the tire-pavement interface as possible.

3 The open graded asphalt friction course (OGAFC), with a large proportion of one size aggregate, provides excellent coarse texture and exposes a large area of coarse aggregate. Guidance for this mix can be obtained from FHWA Technical Advisory T 5040.13, Open-Graded Asphalt Friction Courses, January 11, 1980.

(b) Rigid Pavements

1 The evaluation of portland cement concrete (PCC) pavements should include a determination that the finishing procedures, mix design, and aggregates provide the initial texture and necessary surface durability to sustain adequate skid resistance.

2 In PCC pavements, the initial and early life skid resistance properties depend primarily on the fine aggregates for microtexture and on the finishing operation for macrotexture. Specifications for texturing concrete pavements should be carefully selected and enforced to ensure a macrotexture pattern appropriate to the type of facility.

3 Regardless of the finishing or texturing method used, adequate durable skid resistance characteristics cannot be attained unless the fine aggregate has suitable wear and polish resistance characteristics. Research by the Portland Cement Association indicates that the siliceous particle content of the fine aggregate should be greater than 25 percent.

4 If pavement evaluation studies indicate that the coarse aggregates will be exposed by the surface wear and have a significant effect on skid resistance of pavement, it too should have a suitable polish resistance characteristic.

5 Metal tines, preceded by burlap or another type of drag finish, are recommended as being the most practical and dependable method of providing texture in PCC surfaces. Additional guidance can be obtained from FHWA Technical Advisory T 5140.10, Texturing and Skid Resistance of Concrete Pavements and Bridge Decks, September 18, 1979.

b. Pavement Construction

(1) Highway agencies are encouraged to adopt a policy of "prequalifying" aggregates to be used in surface courses. Prequalifying is a method by which aggregates can be classified according to their friction, texture, wear, and polish characteristics. Classifications should reflect performance related to traffic volume, operating speed, percent trucks, climate, geometric design, and other appropriate factors. Design procedures should be established to ensure that aggregates can be selected for each project which are suitable to the needs of traffic.

(2) Prequalification may be accomplished by one of the following, or a combination of both:

(a) A systematic rating of all fixed sources of aggregates (e.g., a commercial quarry which obtains aggregate from the same location for many years). Ratings should be based on standardized laboratory tests such as the American Society for Testing and Materials (ASTM) D 3319, Recommended Practices for Accelerated Polishing of Aggregates Using the British Wheel, or ASTM D 3042 Test for Insoluble Residue in Carbonate Aggregates, combined with data obtained from skid resistance tests of pavements in service. Other tests may be added or substituted if shown to predict pavement performance.

(b) An evaluation and in-service history of the geologic or petrographic types of aggregates commonly used. Thus, when a new aggregate source is proposed, it can be accepted with minimum testing if an in-service history has been established for that type of aggregate.

(3) Based on prequalification of aggregates, construction plans and specifications should define the friction quality of aggregate which will be acceptable. The following steps should be followed to assure acceptability of the as-constructed pavement surface course:

(a) After the contractor has identified the particular aggregates and asphalt to be used on a project, it is recommended that a mix design be performed with the actual ingredients being used. Aggregates should be checked to determine if they are from prequalified sources or are an acceptable petrographic type.

(b) Macrotexture and void content are important considerations in asphalt mixes. Since asphalts are often blended from several sources of crude oil that vary in temperature-viscosity characteristics, the mixing temperature should be determined for each project after establishing the characteristics of the selected asphalt. Allowable tolerances for asphalt content, mixing temperatures, and gradation should be established for each asphalt mix.

(c) Job control of asphalt mixes should be designed to ensure that desired skid resistance properties are obtained. It should be recognized that small changes in aggregate gradation or asphalt content may significantly affect the macrotexture of finished surfaces.

(4) The frictional properties of pavement surface types should be randomly tested within 6 months after opening to traffic to verify that the anticipated characteristics are present. Evaluation tests should involve direct measures such as the skid tester (ASTM E 274), or an acceptable alternative, but may use surrogate measures such as those which evaluate texture (for example, ASTM E 303, Standard Method for Measuring Surface Frictional Properties Using the

British Pendulum Tester; and sand patch tests as described in the American Concrete Paving Association Technical Bulletin No. 19, Guidelines for Texturing Portland Cement Concrete Highway Pavements, Measurement of Texture Depth by the Sand Patch Method).

(5) In cases where the skid resistance properties of a pavement are found to be questionable or inadequate, appropriate warning signs should be placed immediately as an interim measure. A complete evaluation and any remedial action needed should be effected as soon as possible.

c. Pavement Maintenance. The same procedures and quality standards used in construction should be used in the maintenance operations.

5. WET WEATHER ACCIDENT LOCATION STUDIES. The purpose of this type of study is to identify locations with high incidence of wet weather accidents, determine corrective measures, and take appropriate actions in a timely and systematic manner. This activity should be conducted as part of the highway agency's safety improvement program and should make effective use of the agency's accident data file. Items to be considered for retrieval from the accident and traffic records are total accidents (rate), wet weather accidents (rate), and the wet/dry ratio.

a. Identification of Wet Weather Accident Sites

(1) Accident records, which are developed in compliance with Highway Safety Program Standard No. 9, Identification and Surveillance of Accident Locations, should be searched at least annually to identify sites which have a high incidence of wet weather accidents. It is essential to have a standardized highway location reference system for correlating data from different sources. Accident rates at a site will be of greatest value if:

(a) the traffic volume is relatively high (i.e., approximately 1,500 vehicles per day or greater),

(b) the period of accident data is at least two years, and

(c) rainfall data are available for the same period as the accident data.

(2) Rainfall patterns for the years in which skid resistance and accident data were compiled should be acquired for each area in the highway agency's jurisdiction. A suggested method is presented in Appendix A.

(3) There are several methods in use by highway agencies to evaluate wet weather accident locations. One such method is the Wet Safety Factor (WSF), which is presented in Appendix A.

b. Field Review. A list of all sites ranked in order of WSF or another appropriate measure should be prepared as the basic list of candidate sites for remedial treatments. The selected locations should then be skid tested and reviewed by a team representing various disciplines such as highway materials, design, construction, maintenance, traffic and safety. See Appendix B for skid testing procedures. The review team should determine probable reasons for the high incidence of accidents and recommend corrective actions. Once the review team has recommended appropriate corrective treatments, a priority list of projects can be prepared based on benefits and expected costs.

c. Priority Program. An assessment should be made of the benefits relative to the cost of providing remedial treatments for high priority projects. A number of highway agencies have their own methods for conducting benefit cost analyses of alternative remedial treatments. Some of these remedial methods are tied into traffic engineering or pavement management programs. A specific program for evaluating the benefits and cost of alternative treatments is presented in reference 1, Appendix C.

d. Evaluation

(1) Evaluation of completed projects as required in Highway Safety Program Standard No. 9 and FHPM 8-2-3, Highway Safety Improvement Program, should be well documented and should include a representative sample of completed projects. A sampling plan should be established, using accepted statistical methods, to evaluate projects with a range of such variables as classes of roadways, traffic volumes, types of countermeasures, pavements used, and other pertinent factors. On hazard elimination

projects, these data should be correlated with accidents and traffic exposure and other pertinent factors in before/after analysis. See reference 2 in Appendix C.

(2) The evaluation of completed safety projects should be a continuing process to ascertain the long-term performance of corrective actions such as skid resistant overlays. The evaluations should address at least:

(a) the overall effectiveness of the program in reducing accident rates at the corrected sites,

(b) the adequacy of the various materials, designs, or methods used, and

(c) recommendations for changes in the program, practices, or needed research and development.

(3) As a secondary benefit, the evaluation process should provide input to an overall pavement management process.

6. PAVEMENT SKID RESISTANCE TESTING PROGRAM

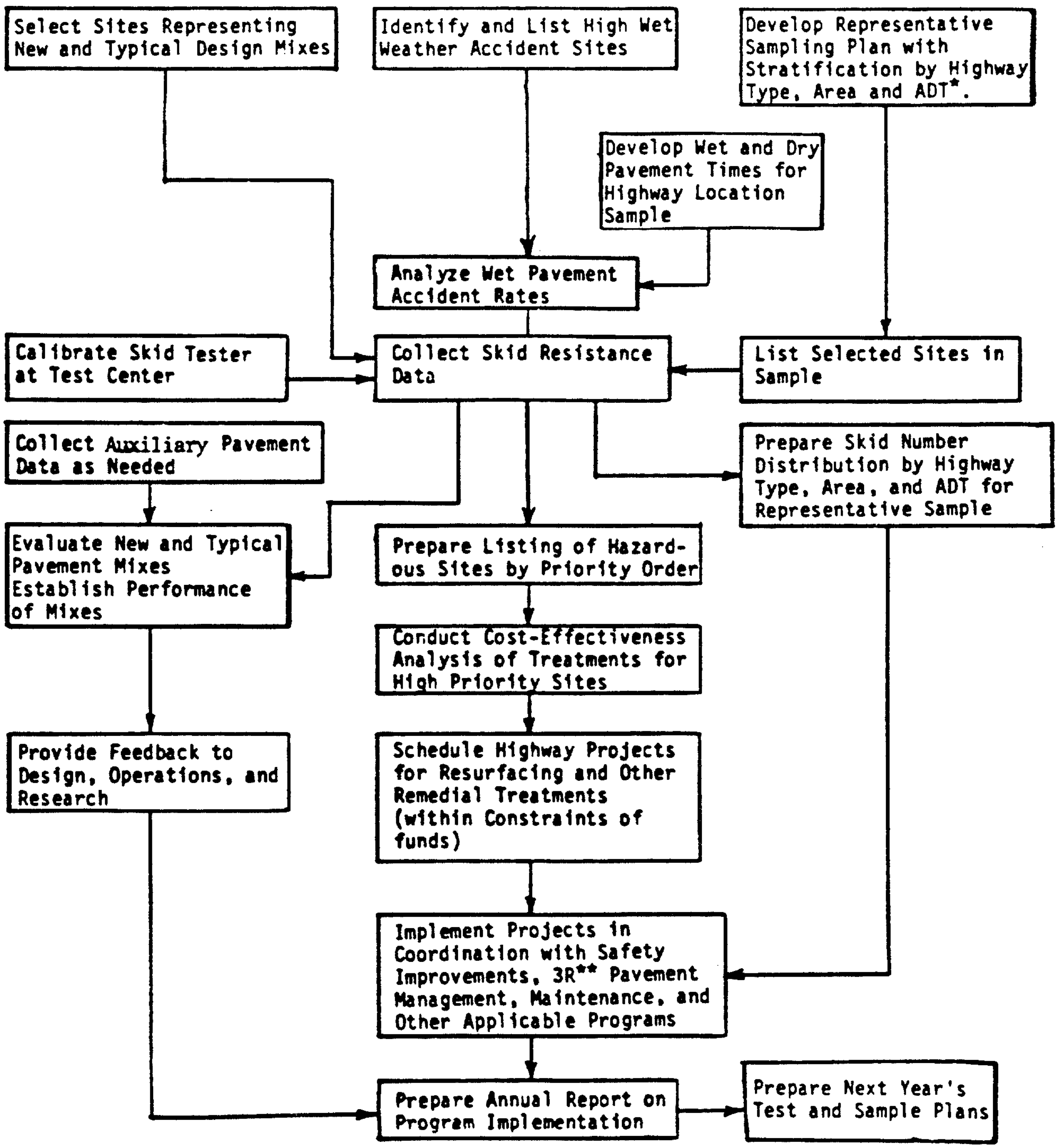
a. General Description of Program. The actual testing of pavement friction provides basic data for use in the three activities introduced in paragraph 3. Figure 1 graphically presents the interrelation between these activities. The upper portion of Figure 1 provides an overview of data to be collected to serve the safety, construction, and maintenance functions of highway organizations concerned with the skidding properties of pavement surfaces. The lower portion of Figure 1 indicates the various uses of the skid testing data, along with weather and accident data. Some of these data are evidence of the durability of particular surfaces, while other data provide a general overview of the skid resistance characteristics of the highway system.

(1) Skid resistance testing should be organized to support the following activities:

(a) Pavement evaluation studies in which measurements of the skid resistance of test sections are made to determine the skid characteristics of typical mix designs. Sufficient numbers of measurements should be

Figure 1

MODEL SKID ACCIDENT REDUCTION PLAN



*ADT: Average Daily Traffic

**3R: Resurfacing, Restoration and Rehabilitation made to determine the level of pavement friction, wear rates, and speed gradient of the pavement under various traffic exposures. These test sections should include the new projects to be tested as described in paragraph 4b(4).

(b) Evaluation of friction characteristics at locations which have a high incidence of wet weather accidents.

(c) System status for which measurements of the skid resistance of a representative sample of roads are made to develop the general levels of pavement friction on all roads in the highway agency's jurisdiction.

(2) Accurate location of sites or road sections requires the use of a standardized reference system. Often each element of the State which collects highway data uses its own reference system. For example, police accident reports may locate accidents by distance to a landmark, pavement records may be kept by project number and geometric features may be identified by station. A unified reference system has many benefits, especially in pulling together technical data for identifying and analyzing locations with a high incidence of wet weather accidents.

(3) Pavement evaluation study sites and wet weather accident sites should be identified by the element within the highway agency responsible for those programs. The skid testing can then become a routine matter for the element charged with operation of the skid test equipment.

(4) A total skid inventory of all roads and streets in a highway system has proven to be impractical and is not necessary to carry out an effective skid accident reduction program. Roads and streets which are used primarily by vehicles traveling at low speeds are not highly susceptible to skid accidents and accordingly can be eliminated from routine sampling of highway sites. For urban areas, this means that most city arterials would be sampled but residential streets and roadways with low speed limits would not. Nearly all rural highway sections could be sampled, since such roads are liable to high-speed use.

(5) Another practical consideration in determining which roads should be sampled is traffic volume. In urban areas, most roads with high speeds have moderate to high traffic volumes whereas this is not the case for rural highways. Relatively few rural roads are used by more than 1,000 vehicles per day. On a cost-effectiveness basis, such roads can seldom justify resurfacing on the basis of safety considerations alone; therefore there is little benefit in routine sampling of low-volume rural roads.

(6) Highway sections within the constraints of higher speeds and volumes need not be tested every year, since few roads vary substantially in skid resistance in any two or three-year period. Beyond this period, however, roads may lose significant skid resistance and may pose a serious danger to users. Using these criteria as part of a sampling plan will permit most if not all highway agencies to make maximum use of skid resistance data without increasing the amount of skid testing undertaken.

(7) Skid resistance measurements should be made with a calibrated locked-wheel skid tester using the ASTM E 274 method and supplemental procedures described in Appendix B or an acceptable alternative method. Locations such as intersections and sharp curves which are not easily measured with the locked-wheel skid tester at the standard speed of 40 miles per hour should be tested at a lower speed. Such tests should be supplemented with texture measurements to permit extrapolation of available skid resistance to operating speeds. Alternative methods of measuring pavement friction properties may be used provided they correlate well with the locked-wheel skid tester.

(8) In analyzing the skid numbers obtained, the time of year the measurements were taken has to be considered. Several States have published the results of their analyses and have developed methods for correcting skid number measurements taken during various periods and for different pavement surface types. See references 5 and 6 in Appendix C.

b. Specific Data From Sample Sites. In conjunction with skid resistance measurements, pavement wet time and accident records are desirable for each roadway section in the sample. The highway location system should be used for correlating data from different sources. An example of specific data which is desirable at each sample site is given in Appendix D.

c. Sites With Low Skid Resistance. When sites with low skid resistance are identified during the testing of system status, these sites should be analyzed for corrective action. This can be done through a pavement management program, a high hazard elimination program, or other efforts. If the high hazard elimination program is used, the analysis should be in accordance with FHPM 8-2-3.

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Appendixes

EVALUATION OF WET PAVEMENT TIME AND ACCIDENT DATA

A.1 The quantity of rainfall (inches) recorded by weather stations may be used to calculate the percentage of pavement wet time. Wet pavement time (WPT) may be estimated from total annual rainfall in inches (AR) as follows:*

$$WPT = 3.45 \ln (AR) - 5.07$$

Dry pavement time may be estimated by subtracting the amount of wet time and ice and snow periods from the total time in the period analyzed. Data from rainfall stations maintained by the National Oceanic and Atmospheric Administration's Weather Service may be used for wet and dry pavement time estimates for various areas within a State.

Isohyetal maps may be used to develop site wet pavement times. If ice and snow cover pavements for a significant portion of the time, a map for dry time should be prepared as well. Figure A-1 provides an example of a wet time map drawn from isohyetal charts.

A.2 Wet Safety Factor (WSF)

There are a number of ways to evaluate the relative safety of the subject location, one of which is the wet safety factor (WSF) approach.** For each wet weather accident location, a WSF may be developed. This factor is expressed as follows:

$$WSF = (DA)(PWT)/(WA)(PDT)$$

where: DA = number of dry weather accidents
WA = number of wet weather accidents
PDT = percent of dry pavement time
PWT = percent of wet pavement time

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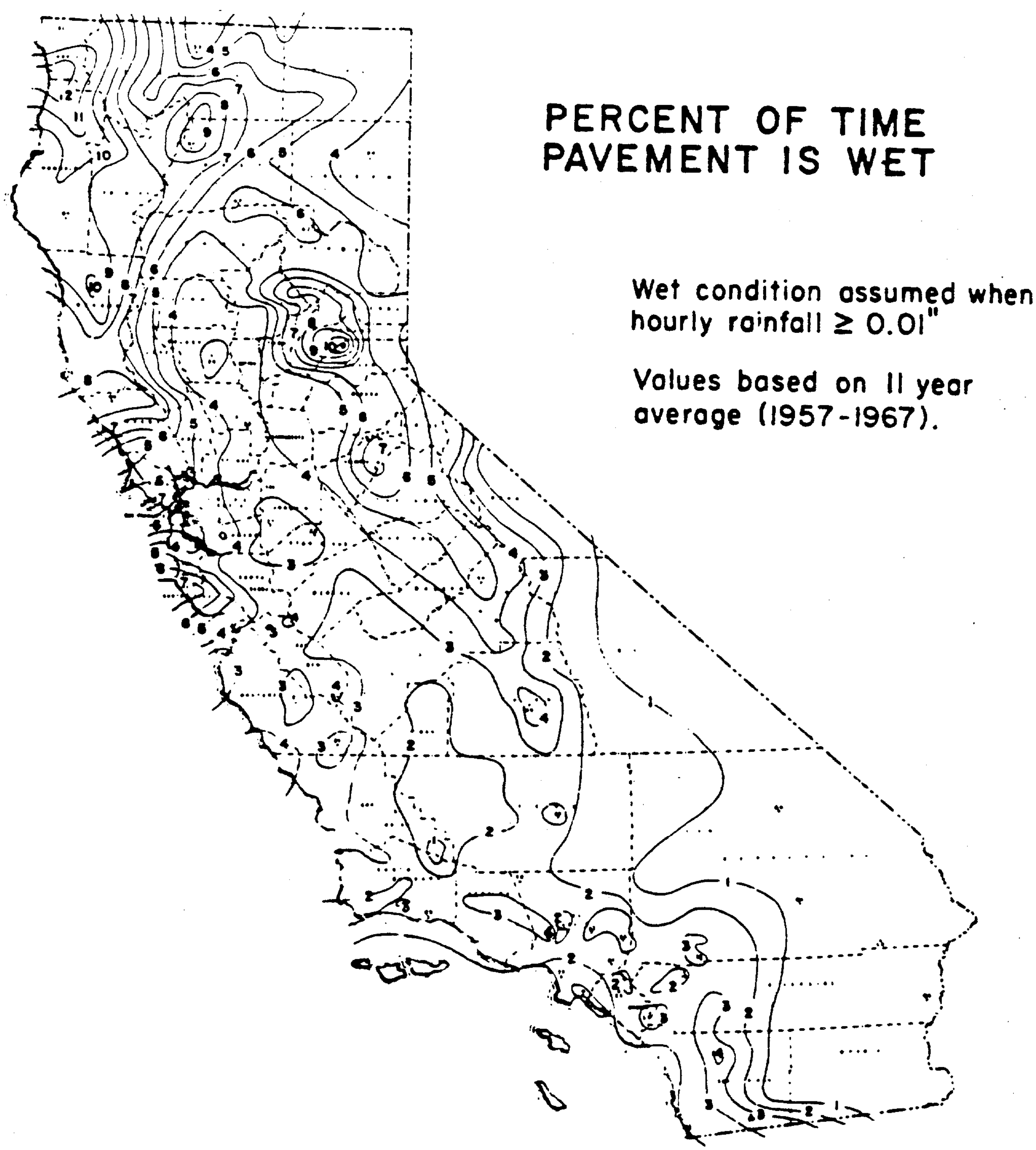
* This equation is based on a relationship developed by K.D. Hankins in "The Use of Rainfall Characteristics in Developing Methods for Reducing Wet Weather Accidents in Texas," Texas State Department of Highways and Public Transportation Study No. 135-4, July 1975.

** The WSF is a generalized form of an index referred to as the "skid trap ratio" and recommended for use in NCHRP Report 37, Tentative Skid-Resistance Requirements for Main Rural Highways," by H. W. Kummer and W. E. Meyer, Highway Research Board, Washington, D.C., 1967.

Figure A-1



[1] "A Method to Determine the Exposure of Vehicles to Wet Pavements," by James I. Karr, California Business and Transportation Agency, Department of Public Works, Division of Highways, January 1972.

This factor is the reciprocal of the risk of having a wet pavement accident relative to having a dry pavement accident. On a specific roadway section, each of these variables must be developed for the same time period; otherwise, traffic exposure must be taken into account. Criteria may be developed for further consideration of pavement sections. A WSF less than 0.67 suggests a wet weather problem. This criteria is based upon the conservative estimate of the overall likelihood of a wet weather accident being 1 1/2 as great as a dry pavement accident. This estimate assumes that wet weather accidents at the site or road section under consideration are attributable entirely to a skidding problem. A low WSF in most cases is due to poor skid resistance. However, traffic engineering evaluations may reveal deficiencies in sight distance, road markings, inadequate drainage, etc. Auxiliary information obtained during the test program should provide indications of the safety problems.

December 23, 1980
Appendix B

SKID MEASUREMENT SYSTEM DESCRIPTION AND OPERATING PROCEDURES

B.1 DESCRIPTIONS OF SKID MEASUREMENT SYSTEM

The requirements of American Society for Testing and Materials (ASTM) E 274 states "The method utilizes a measurement representing the steady state friction force on a locked test wheel as it is dragged over a wetted pavement surface under constant load and at constant speed while its major plane is parallel to its direction of motion and perpendicular to the pavement."

Although this specification may be met by a system involving only one wheel attached to a towing vehicle and although a few such systems are in use, the vast majority of skid measurement systems in use and expected to be in use in the near future consist of a towing vehicle and two-wheel trailer. On many systems either wheel may be locked during testing, but most commonly, the left is used.

The ASTM considers testing the left wheel track to be "normal." However, a differential in friction levels between the left and right wheel track may exist. When testing a site where a differential may exist, especially a high wet weather accident site, all lanes and wheel tracks should be tested. If a two-wheel trailer system is used, it is desirable to have the capability of testing with either wheel.

A skid measurement system must have a transducer associated with each test wheel which senses a force equal or directly related to the force developed between the sliding wheel and the pavement during test, electronic signal conditioning equipment to receive the transducer output signal and modify it as required, and suitable analog and/or digital readout equipment to record either the magnitude of the developed force or the calculated value of the resulting skid number (SN).

The system must include a facility for the transport of a supply of water--usually 200 to 500 gallons--and the necessary apparatus to deliver a specified amount of water--4.0 gallons per minute per wetted inch of pavement at 40 miles per hour within specified limits in front of the test wheel.

Finally, the system must include provision for measuring (and preferably for recording) the speed at which the test is conducted.

B.2 FIELD OPERATING PROCEDURES

B.2.1 Field Force Verification

It is generally impractical to perform force plate calibrations at frequent intervals while the measurement system is in the field. Facilities should, however, be available to permit the operator to ascertain that significant changes have not occurred in the force measurement subsystem since the most recent force plate calibration.

If the measurement system uses a torque transducer and is adaptable to mounting a torque arm, the verification can be accomplished within a reasonable time and effort. This device, consisting of an arm capable of being bolted to the test wheel in a horizontal position and of supporting known weights located at specified distances from the center of the test wheel, may be used to test the torque transducer to predetermined values of torque. Typically, the test wheel of the inventory system is raised off the ground, the torque arm is attached to the test wheel and held in a horizontal position, the brake of test wheel locked, and a series of known weights are suspended on the torque arm. This procedure will induce a series of known strains on the transducer, resulting in a series of output signals through the signal conditioning equipment. The magnitude of these signals should then be compared to the magnitude of signals produced through use of the same technique immediately after the most recent force plate calibration. Adjustment of signal conditioning equipment gain setting may be made to offset small force measurement subsystem variations which could occur.

Verification should be repeated periodically.

B.2.2 Test Tire and Wheel Preparation, Control of Tire Pressure

Tire Specification

Unless otherwise specified, all tests shall be performed with tires meeting the requirements of ASTM E 501, Standard Tire for Pavement Skid Resistance Tests, and all pertinent sections of that specification as well as ASTM E 274 should be observed in their use.

Tire Mounting and Break-in Procedure

The tire should be mounted on a Tire and Rim Association 6JJ rim. The rim should have been examined to determine that it has suffered no damage or misalignment in prior use. After mounting, and before break-in, the tire and wheel should be balanced. The tire should be subjected to a break-in of 200 miles use before being used for testing. This break-in may be accomplished by using the tire on the skid trailer wheel which is not used for testing. If the tire must be remounted before test use, it should be rebalanced after remounting.

Tire Warm-Up Procedure

The test tire should be inflated to 24 + 0.5 pounds per square inch measured at ambient temperature. After tire pressure measurement and adjustment, the tire should be subjected to a 5-mile warm up, travelling at conventional highway speeds, before tests are performed. The 5-mile warm-up should be repeated on any occasion when the measurement system is parked for a period of 15 minutes or more.

Tire Wear and Replacement Procedure

The standard pavement test tire has a series of visual wear guide sipes (small circular holes) cast into each of the outer ribs of the tire. The test tire should be withdrawn from testing use when wear has progressed to a point at which the wear guide sipes are no longer visible. During routine testing, test tires should be examined at least twice daily (and more frequently as tire nears unacceptable wear level) to determine that wear has not progressed beyond acceptable limits.

Additionally, after any series of tests on pavements having very high skid numbers (in excess of SN=70) or in the event of a deliberate or inadvertent dry skid, the test tire should be examined for the development of a flat spot. If a significant flat spot or spots develop on a test tire, it should be withdrawn from test use due to the tendency of the test wheel to seek out and return to such a flat spot in subsequent lockups.

B.2.3 Watering Subsystem Procedures

Daily Procedures

Prior to the beginning of each day's activity, the crew should perform at least the following functions with respect to the water subsystem:

1. Determine that the water nozzle (nozzles) when in the testing position assumes the proper angle with respect to the pavement (ASTM E 274 requires an angle of 25 + 5 degrees).

2. If the measurement system has provision for raising and lowering the nozzle between tests, determine that the mechanism is working properly and that the nozzle assumes a fully lowered position during the test sequence.

3. Determine that the nozzle, when in the test position, will discharge water directly in front of and centered on the test wheel.

4. Examine the nozzle outlet orifice to determine that it is free from damage or distortion.

The above inspections should be repeated during a day's testing in the event of operation on very rough highways (or in the event of any off-highway travel) which may have caused damage to the nozzle or adversely affected its orientation.

Water Trace Width Check

Periodically the crew should make a measurement of the water trace width as a gross measure of overall water subsystem performance. This may be accomplished by driving the measurement system over a pavement at a selected convenient speed (the same speed should be used on all occasions), initiating water flow without locking the test wheel brakes, and measuring the width of the resulting water trace on the pavement. The trace width measurement should be made as quickly as possible after passage of the inventory system (preferably within 30 seconds). This would require that one member of the crew drive and operate the measurement system while the other member is positioned off the side of the pavement at the location at which the measurement is to be made. Best results are achieved if this procedure is performed on a relatively smooth pavement surface (low macrotexture).

B.2.4 Instrumentation Calibration Verification

Provision should be made to allow for verification of the signal conditioning instrumentation calibration (to account for the effects of zero and gain drifts).

General Requirements for Calibration Signal

The minimum acceptable facility for verification of conditioning instrumentation is a calibration signal subsystem. The calibration signal should be provided from such a source and in such a manner that there is little likelihood of variation in the calibration signal itself. This assurance then permits the operator to make adjustments in the measurement subsystem gain to offset the frequent small deviations which occur due to changes in ambient temperature and other operating parameters.

Force Measurement Calibration Signal

The most straightforward technique for providing a force measurement calibration signal is to make provisions for switching a high quality shunting resistor of known value in parallel with one arm of the force transducer strain gauge bridge. This induces an imbalance in the bridge equivalent to the application of a known force to the transducer. The resultant signal is sufficient to verify, or provide means of adjustment for, all elements of the force measurement system forward of the transducer itself.

Frequency of Use

Instrumentation calibration verification through use of calibration signals should be accomplished at the beginning of each day's operation after equipment warm up, at intervals of no more than 2 hours when the system is in continuous use, and upon the renewal of operation throughout the day after any period during which the signal conditioning equipment has been turned off or the unit has been allowed to stand without use for 30 minutes or more.

B.2.5 Check List

A check list should be available to the crew and should be used prior to the beginning of daily operations and on any occasion during the day when testing is suspended for 30 minutes or more or when instrumentation has been turned off. The check list varies from system to system due to differences between the systems, but should provide for at least the following checks:

1. all power subsystems on and providing proper levels of power
2. all signal conditioning subsystems on for adequate time to reach stable operation (typically 10 to 30 minutes)
3. all recording systems on and functioning properly
4. instrument calibration (described above) performed
5. tire pressure checked and adjusted if necessary
6. test tire checked for wear
7. water nozzles checked for position and condition
8. water tank adequately filled
9. fuel supply adequate
10. safety chains and all other connections between trailer and towing vehicle properly connected, positioned, and protected if necessary
11. trailer jacks (if available) in retracted position
12. all auxiliary equipment (air-compressors, lights, etc.) functioning properly

B.3 USE OF STATIC AND DYNAMIC CALIBRATION PROCEDURES

B.3.1 Purpose of Field Test Center

At the present time the highest order of calibration and evaluation available for a State skid measurement system is that provided through the Field Test Center established under contract by the Federal Highway Administration (FHWA). Arrangements to receive the services of the Field Test Center may be initiated by a State through submittal of a request for such services to the local FHWA division office.

B.3.2 Criteria for When to Use the Field Test Center

Each measurement system should be submitted for calibration and evaluation at the Center as soon as possible after its introduction into service. It should be resubmitted for calibration and evaluation whenever:

1. significant repair or modification has been accomplished by the owning agency which might reasonably be expected to affect test results, or

2. whenever it has experienced sufficient use such that normal wear in the various subsystems might be expected to have affected their operation.

The second consideration suggests that each measurement system should be resubmitted at least every 2 years.

B.3.3 Calibration Services Provided by Field Test Center

The static and dynamic calibration services provided by the Field Test Center include the following:

1. Horizontal and Vertical Force Calibration. This provides for evaluation of the accuracy, linearity and hysteresis of the measurement system force transducers and signal conditioning equipment through use of an air bearing force plate maintained by the Center, and periodically calibrated by the National Bureau of Standards.

2. Flow Rate Evaluation and Adjustment if Required. This includes determination that the water delivery subsystem of the measurement system provides a quantity of water (dependent upon trace width) in front of the test tire which meets ASTM E 274 requirements at speeds between 20 and 60 miles per hour.

3. Static Evaluation of Water Distribution. This provides an evaluation of the uniformity with which the total water flow is distributed across the trace width and adjustment, if necessary, to assure that the water is in fact delivered uniformly and in line with the test tire.

4. Force Plate or Load Cells. The visitors force plate used for routine checks of the force measurement subsystem can be calibrated while at the Center.

5. Speed Calibration. The speed measurement (and recording if available) subsystem is evaluated, calibrated and, where necessary and possible, adjusted to produce accurate speed measurement values over the range of 20 to 60 miles per hour.

6. Tire Pressure Gauge Calibration. This provides assurance that tire pressures in the test wheels and in the speedmeasuring fifth wheel (if used) can be accurately measured and set.

7. Dynamic Correlation. Two such correlations are conducted: The first with the measurement system in the "as arrived" condition and the second after all of the foregoing evaluations have been conducted and indicated adjustments accomplished. The first correlation results in the development of mathematical relationships between the measurement system and the Area Reference Skid Measurement System that permit data collected by the measurement system, prior to its visit to the Center, to be adjusted to a common base provided by the use of the Area Reference System. The second correlation permits the development of similar relationships which may be used to relate the results of subsequent testing to the Area Reference System base. The data from the second correlation also provide an estimate of the system measurement variance.

B.4 MAINTAINING SYSTEM INTEGRITY BETWEEN FIELD TEST CENTER CALIBRATIONS

Two basic types of procedures are available for determining that significant changes have not occurred in the measurement system since its most recent evaluation and calibration at the Center. These involve techniques for evaluating important subsystem performance and techniques for evaluating performance of the total system.

B.4.1 Techniques to Evaluate Subsystem Performance

As a minimum, the owner of each measurement system should maintain and periodically make use of facilities for evaluating the force, water, and speed measurement subsystem of the inventory system.

Evaluation of Force Subsystem

The force subsystem should be evaluated through use of a force plate. An air-bearing force plate is recommended since its action is such as to essentially eliminate the effect of friction in the plate itself. If an air-bearing force plate is not available, any of several commercial mechanical force plates may be used. If a mechanical device is used, precautions should be taken to assure that all moving parts (particularly load application screws and spherical or roller bearings) are well lubricated and that the lubricant is periodically removed and replaced.

To conduct an evaluation, the test wheel of the measurement system should be centered on the force plate, the test wheel brake locked, and known frictional forces introduced to the tire-force plate interface through appropriate motion of the force plate. Frictional forces should be both increased and decreased in a stepwise manner to allow for detection of possible hysteresis effects. The indicated force readout values for the system should then be plotted against known force input values. The resulting plotted calibration line should be evaluated for nonlinearity and hysteresis characteristics. Also actual readout values for known force inputs should be compared with those readout values determined from tests conducted with the same equipment after the most recent Center evaluation.

Evaluation of Water Subsystem

The most effective evaluation of the water subsystem to discern variations in performance is that of flow. Flow rate may be evaluated by raising the rear wheels of the towing vehicle, running the vehicle at an indicated speed of 40 miles per hour (or any other desired speed), collecting the water pumped through the system and out the nozzle during a measured time period, and calculating the flow rate in gallons. This procedure should be repeated at two or more speeds to evaluate linearity of the water delivery subsystem with test speed.

The Pennsylvania State University has developed a water rate flow tank which is circular in cross section and of such size that it fits easily into a standard manhole. The tank has a threaded opening in the bottom for drainage and a stop-plug with a long handle which permits the plug to be removed and replaced from the top of the tank after it is hanging in the manhole. It also has a scale calibrated in gallons on the inside of the tank. This tank may be suspended in a standard manhole, the measurement system positioned so that the nozzle will discharge directly into the tank, the rear wheel of the towing vehicle raised, and total flow measured at any desired speed. The only additional equipment required is a stopwatch.

Evaluation of Speed Measurement Subsystem

The speed measurement subsystem should be evaluated by operating the measurement system at various test speeds over a measured mile course. If the basic speed measure is done through the use of the tow vehicle speedometer or through a tachometer-generator driven by the tow vehicle or by a fifth wheel, then the vehicle should be driven over the measured mile course at a selected speed and the time of transit measured with a stopwatch. The actual speed, calculated from the distance and the elapsed time, is then compared to the indicated speed.

If speed measurement is based upon a pulse generator driven by a fifth wheel, the accuracy of the speed measurement is directly dependent upon the accuracy of the fifth wheel for distance measurement. To evaluate this subsystem, the fifth wheel tire pressure is adjusted until the distance indicated agrees with the known distance traversed (the assumption being made here is that the electronic package which converts the pulses to velocity is functioning properly).

If tapeswitch event detectors, placed 200 feet apart, and an interval timer (+0.01 second resolution) are available to measure the time required by the inventory system to travel 200 feet, a very accurate speed measurement is obtained to check against the indicated value.

Time Between Subsystem Evaluations

The force, water and speed measurement subsystems of the measurement system should be checked by the methods described above at intervals no greater than 3 months.

B.4.2 Techniques to Evaluate Total System Performance

Use of Measurement System Sample Variance as Performance Measure

A portion of the information furnished, as a result of an evaluation at the Center, is the pooled sample standard deviation of the measurement system for repeated test at three test speeds on five special test surfaces. If the sample standard deviation at the desired speed is squared, the resulting value, SD_t^2 is an estimate of the skid measurement system variance. Subsequent to the Center evaluation, the crew should periodically select a pavement location having a skid number of approximately 30 to 40 and run 20 repeat tests at the desired speed over the same location. From the results of these latter tests, a new estimate, SD_E^2, can be calculated. If the ratio SD_E^2/SD_t^2 does not exceed 2.0, the chances are 19 in 20 that the system standard deviation has not doubled over that established during its visit to the Center. (If the system has not been to a Center to obtain an estimate of SD_t^2, its crew should select a pavement location having a skid number of approximately 30 to 40, run repeat tests at each desired speed over the same location, and calculate the sample standard deviation at each such speed.)

As an alternative, the above procedure could be performed making only 10 repeat tests on the selected pavement. In this case, the ratio of SD_E^2/SD_t^2 should not exceed 2.2 The chances are then four in five that the system standard deviation has not doubled over that previously established.

The above procedure should be performed at time intervals no greater than 3 months.

Short Term Checks of System Performance

The agency operating the measurement system should select several pavements located close to the site at which the system is normally garaged and perform repeated tests on the surfaces at quite frequent intervals, preferably weekly. Measured values of skid resistance on these surfaces will obviously change as the surfaces change from traffic wear, environmental, and/or seasonal variations. However, these changes should occur in an orderly and predictable fashion and any abrupt change would be an indication of possible erratic performance of the measurement system. A continually updated record of the results of such tests should be maintained and examined after each updating for evidence of such erratic performance.

REFERENCES

The following is a selected list of references which may be helpful in implementing the program described in this Technical Advisory. This list is not intended to be a bibliography of all documents available in this field:

*1. Effectiveness of Alternative Skid Reduction Measures, Benefit Cost Model, Report No. FHWA-RD-79-12, Volume II, November 1978, Federal Highway Administration.

*2. Accident Research Manual, FHWA-RD-80-016, February 1980, Federal Highway Administration.

*3. Evaluation of Minor Improvements (Part 8), Grooved Pavement (Supplemental Report) CA-DOT-TR-2152-11-75-01, September 1975, R. N. Smith and L. E. Elliott, Office of Traffic, California Department of Transportation.

4. Evaluation of Minor Improvements (Part 9), Open Graded Asphalt Concrete Overlays, January 1972, James I. Karr, Office of Traffic, California Department of Transportation.

5. Variations in Skid Resistance Over Time, FHWA-VA-80-33, February 1980, S. N. Runkle, David C. Mahone, Virginia Highways and Transportation Research Council.

6. Seasonal Variations in the Skid Resistance Pavements in Kentucky, Research Report 532, November 1979, James L. Burchett, Roland L. Rizenbergs, Kentucky Department of Transportation.

* These studies are available through the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161

SPECIFIC DATA TO BE REPORTED FOR SAMPLE SITES

The following data should be collected in testing sample locations:

D.1 Skid numbers (SN) should be taken for major classes of roads stratified by traffic volume and geographical location.

D.2 Auxiliary data which should be included in order to establish distribution of skid numbers may include the following:

(a) Location of site or roadway section

(b) Responsible jurisdictional unit and route number or other designator

(c) Functional classification of road (e.g., two-lane, four-lane divided without full control of access, etc.)

(d) Surface type (e.g., bituminous, open-graded, concrete, tine finish, etc.)

(e) Average annual daily traffic (use traffic count data if available)

(f) Length of roadway section

(g) Lane where skid measurements are made

(h) Date of skid measurements

(i) Number of tests made in section

(j) Average SN

(k) Range of SN measurements

(l) Presence of atypical geometric or feature

(m) Evidence of skidding (e.g., skid marks, scarred posts, etc.)

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

SUBJECT	FHWA TECHNICAL ADVISORY
Texturing and Skid Resistance of Concrete Pavements and Bridge Decks	T 5140.10 September 18, 1979

1. PURPOSE. To issue guidance for providing an adequate level of wet pavement skid resistance on portland cement concrete (PCC) pavement surfaces and plain or latex modified concrete bridge deck surfaces.

2. CANCELLATION. This issuance supersedes FHWA Notice N5080.59 dated September 10, 1976, "Texturing of Concrete Pavements and Bridge Decks."

3. BACKGROUND

   a. If a pavement or bridge deck surface is to provide adequate skid resistance for high speed traffic, two important, and to some extent independent, requirements must be satisfied. First, the pavement surface must provide for adequate adhesion between the tire and the pavement under wet weather conditions. Second, the pavement surface must provide sufficient surface texture and drainage potential to prevent the buildup of water pressure at the tire-pavement interface. On PCC riding surfaces, this drainage potential can be provided initially by the texturing of the concrete surface. The adhesion component of skid resistance is dependent on the wear and polish resistance characteristics of the aggregates.

   b. A concrete finishing procedure that will provide an adequate and durable skid resistant surface texture is needed on all pavement and bridge deck surfaces.

   c. While a skid resistant surface texture is needed on all pavement surfaces, the need may be especially critical on bridge decks because of the limited recovery area available to out-of-control vehicles.

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H-DM-4 HNG-32

4. RECOMMENDATIONS

a. A burlap drag finish should not be used as the sole means of providing surface texture on projects with design speeds of 40 miles per hour (65 km per hour) or greater.

b. A broom or artificial grass drag finish, while often producing good initial surface texture, may not be durable unless a deep texture is produced by heavy bristles and high pressures. When a broom, artificial grass drag, or similar finish is specified for use on projects with design speeds of 40 miles per hour (65 km per hour) or greater, specifications that will insure a deep, durable texture must be used. Also, it should be conclusively demonstrated that safe, durable surfaces can be consistently produced by finishes before they are utilized extensively on high speed highways.

c. Metal tines, when preceded by a burlap or other type of drag finish, are recommended as being the most practical and dependable method of providing positive texture in PCC surfaces. The use of other procedures and equipment that will provide a similar grooved pavement surface is also encouraged. An Attachment to this advisory provides a summary of some of the more important research findings and recommendations relative to the texturing of PCC pavement surfaces.

d. The use of a deep surface texture on bridge decks may warrant increased concrete cover over the top layer of reinforcement steel beyond the 2.5 inches (63mm) required for minimum design cover and construction tolerance. The use of heavy transverse textures may also increase the concentration of deicing salts along the curb line unless proper provisions are made to drain these areas. The last 12 inches (300mm) of deck next to the curb should be left untextured to facilitate drainage.

e. Special techniques may be required in order to produce a durable grooved finish on latex modified and other low water-cement ratio bridge deck surfaces. The use of flexible tines on such surfaces frequently fails to produce a groove depth sufficient for reasonable durability. In at least one State, satisfactory grooves have been produced in these dense surfaces through the use of a roller consisting of sharpened flat washers welded at intervals onto the outside of a weighted section of pipe. Other techniques may also prove to be successful. As is the case with pavement surfaces, it is normally desirable that the plastic grooving application be preceded by a burlap or other drag finish or a light broom finish.

f. Regardless of the finishing or texturing method used, adequate durable skid resistance characteristics cannot be attained unless the fine aggregate has suitable wear and polish resistance characteristics. Research conducted by the Portland Cement Association indicates that the siliceous particle content of the fine aggregate should not be less than 25 percent. If past experience indicates that the coarse aggregates will be exposed by surface wear and have a significant effect on the skid resistance of the pavement, they, too, should have suitable polish resistance characteristics. Crushed material will normally provide higher skid resistance than uncrushed gravels.

R. D. Morgan
Associate Administrator for
Engineering and Traffic Operations

Attachment

TEXTURING OF PCC PAVEMENT SURFACES

1. Transverse grooving will assist in providing a pavement surface with good durable pavement skid resistance characteristics at high speeds, will reduce splash and spray and headlight glare from wet roadway surfaces, and will continue to facilitate surface drainage until the depth of the wheelpath ruts exceeds the depth of the grooves. Longitudinal grooving assists vehicle control at curves and sites involving lateral movements. Both types of grooving effectively reduce the hydroplaning potential. The longitudinal grooving of existing pavements, while not necessarily producing an improvement in skid number, has been found to be an effective means of reducing accidents at sites having high, wet weather accident rates.

   Although longitudinal grooving may be preferable under some circumstances, and particularly when dealing with existing pavements, transverse grooving is considered to be superior to longitudinal grooving for general use on new construction because of the improved pavement drainage provided. Also, with the increased use of smaller, lighter cars and radial tires, complaints of vehicle handling problems on longitudinal grooved pavements seem to be on the increase.

   From the standpoint of compatibility between tire tread designs and pavement texture, it is desirable that, to the extent practical, longitudinal grooving patterns be standardized. The use of 0.095-inch (2.4mm) wide grooves spaced on 3/4-inch (19mm) centers is recommended. There is insufficient information available on which to base recommendations regarding the optimum texture for a milled PCC pavement riding surface. Some extremely harsh milled surfaces may be unacceptable for use as a riding surface.

2. The use of tines 0.03 inches (0.8mm) thick, 0.08 inches (2.0mm) wide and 4 to 6 inches (100 to 150mm) in length has resulted in good, durable surfaces when the grooves

were constructed to the maximum depth practical. This maximum practical depth, which will vary depending on the concrete mix design and other factors, is normally in the 1/8-inch (3mm) to 3/16-inch (5mm) range. Average transverse groove spacings of approximately 1/2 to 3/4 inch (13-20mm) are recommended. Groove spacings of less than 1/2 inch (13mm) may not have adequate durability. An increase in average groove spacing beyond 3/4 inch (20mm) cannot be expected to increase durability by a significant amount and may lead to noise problems. Relatively uniform groove spacings of 1 inch and 1 1/2 inches (25 and 38mm) have in some instances been considered unacceptable because of the concentration of tire-pavement noise into an objectionable frequency band. Experience with average groove spacings exceeding 1 1/2 inches (38mm) is too limited to facilitate the drawing of any conclusions. The use of wider, deeper grooves can be expected to increase the absolute noise level by a significant amount. Randomization of the groove spacing, which may result to some extent from the flexibility of the tines, is considered desirable from the noise standpoint.

3. The optimum length and angle of the tines will be dependent on the mix design, the weather, the finishing operations, and the stiffness of the tines and may be expected to vary somewhat from project to project.

   The timing of the plastic texturing operation is critical. If performed too early, the grooves may close back up. If performed too late, the groove depth will be reduced. The optimum time for performing the plastic grooving operation will be dependent on many variables which are subject to rapid fluctuations.

   When a tine finish is utilized, care should be exercised to avoid overlaps in the texturing operations. The overlaps will result in weak areas which will wear faster than the normal texture. For the best initial surface texture, a tine finish should be preceded by a burlap or artificial grass drag finish.

Because of the increased surface area, deeply textured surfaces will require a heavier than normal application of curing compound.

A tire tread depth gauge may be used to measure groove depths both in hardened and plastic concrete. When measuring the depth of fresh grooves formed by the tining of plastic concrete, it must be recognized that an artificially high reading may be obtained that will not be representative of the actual groove depths that will exist after a few months' exposure to traffic. To insure that the tined grooves will be durable and effective, it is essential that they be made as deep as practical during construction.

U.S. Department
of Transportation
Federal Highway Administration

Technical Advisory

Subject

OPEN GRADED FRICTION COURSES

Classification Code	Date
T 5040.31	December 26. 1990

Par. 1. Purpose
2. Cancellation
3. Background
4. Recommendations

1. PURPOSE. To provide technical guidance on the use of open graded friction courses (OGFC), also known as plant mix seal courses, to develop good friction characteristics for pavement surfaces.

2. CANCELLATION. Technical Advisory T 5040.13, Open-Graded Asphalt Friction Courses, dated January 11, 1980, is canceled.

3. BACKGROUND

a. Open graded friction courses constructed with high quality, polish resistant aggregates have an outstanding capacity for providing and maintaining good frictional characteristics over the operating range of speeds on high speed highways. Their macrotexture facilitates drainage of water from the tire/pavement interface, improving tire contact with the pavement and reducing the potential for hydroplaning.

b. Open graded friction courses have generally provided good performance for 7 to 10 years under a range of traffic conditions. When failures have occurred, many were resolved by making minor refinements to the mix design and construction procedures to adjust for local conditions.

c. When compared to other high type surfaces, open graded friction courses have demonstrated the following advantages:

(1) provide and maintain good high speed, frictional qualities (the frictional characteristics are relatively constant over the normal range of operating speeds);

(2) reduce the potential for hydroplaning;

(3) reduce the amount of splash and spray;

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(4) are generally quieter, often providing a 3 to 5 decibel reduction in tire noise;

(5) improve the wet weather, night visibility of painted pavement markings; and

(6) conserve high quality, polish resistant aggregates, which may be scarce in some areas, because they are placed only as a surface layer, up to 3/4 inch thick.

d. Open graded friction courses exhibit the following limitations:

(1) increase the potential for stripping of the surface and underlying pavement (they do not seal the underlying pavement against moisture intrusion);

(2) require special snow and ice control methods and generally remain icy longer;

(3) require special patching and rehabilitation techniques;

(4) do not add structural value to the pavement (their performance is governed by the condition of underlying pavement); and

(5) may ravel and shove when used at intersections, locations with heavy turning movements, ramp terminals, curbed sections and other adverse geometric locations.

4. <u>RECOMMENDATIONS</u>. In selecting an OGFC, a number of factors should be considered, such as the environmental conditions, alignment, accident rates and the frictional properties of the State's standard surface mixes. Some locations or pavements may not be appropriate for an OGFC and therefore proper project selection must be considered. For an OGFC to perform as intended, it must be properly designed, constructed, and maintained.

a. An OGFC should only be placed on structurally sound pavements that have minimal cracks, ruts, bleeding and depressions. Pavement cracks are as likely to reflect through an OGFC as with any other thin asphalt course. The high air voids content in an OGFC will allow water to drain into it and attempt to flow laterally. Ruts in the underlying pavement may inhibit lateral flow and cause water to pond in the ruts, promoting separation of the OGFC from the underlying pavement. An OGFC placed on a bleeding pavement may lose its drainage characteristics (close up) due to the migration of the free asphalt from the underlying pavement.

b. The underlying pavement should be sealed with a 50 percent diluted asphalt emulsion, applied at a rate of 0.05 to 0.10 gallons per square yard. An OGFC will increase the amount of time that the underlying pavement will be wet. If the underlying pavement has a high air voids content, stripping potential is increased.

c. Specifications should require the coarse aggregate to be polish resistant and 100 percent crushed material. Carbonate aggregates should not be used. Certain slags and light weight aggregates have demonstrated satisfactory performance. The frictional qualities of an OGFC are affected by the microtexture of the coarse aggregate. It is poor practice to construct a premium friction course and then have its effectiveness lost due to polishing.

d. An OGFC should be designed in accordance with the mix design procedures included as the Attachment to this Technical Advisory. The basic steps in this procedure determine asphalt content, mixing temperature, air voids, and moisture damage susceptibility.

(1) An OGFC generally has a higher asphalt content than a dense graded mix and uses an equal or harder grade of asphalt. A very heavy asphalt film on the aggregate is essential for longevity. The film helps to resist stripping and oxidation of the asphalt cement. Typical dense graded mixes achieve a 4-6 micron average film thickness, where as an OGFC requires a 8-11 micron average film thickness. The OGFC has a black shiny appearance and appears to have excessive asphalt when compared to a dense graded mix. It is critical that no reduction in asphalt content be made based on the appearance of the OGFC. Excessive drain down of asphalt in the haul trucks can usually be corrected by lowering the mixing temperature or correcting deficiencies in the mixing and handling procedures. The combined handling and hauling of the mix should be limited to 40 miles or 1 hour.

(2) To ensure that a heavy asphalt cement film is actually obtained, the mixing temperature should correspond to the asphalt viscosity in the range of 700 to 900 centistokes from the temperature-viscosity curve for the asphalt cement. Higher mixing temperatures can cause the asphalt cement to flow off the aggregate. This may result in some areas of the mat having excessive asphalt, others not enough. A range of 2 to 5 percent minus 200 material in the mix will help achieve a thick asphalt cement film. A number of State and local agencies have successfully used latex modified asphalt and other additives to improve OGFC performance.

(3) The air voids analysis is not necessarily required for each project. However, it should be conducted when developing master gradation bands for open graded mixes or when considering new aggregate sources.

(4) An OGFC should be tested for moisture susceptibility because its high air voids content increases the potential for stripping. The mix should be tested for retained coating (AASHTO T 182) and retained strength (modified AASHTO T 165 and T 167). If stripping is observed, the mix design must be revised. The aggregates may be changed or an asphalt cement additive selected. Additional tests should be performed using the revised mix design.

e. One ounce of silicone should be added to every 5000 gallons of asphalt cement. This additive will improve mix workability and reduce the potential of tearing the mat under the paver screed. It also improves mix discharge from the truck beds.

f. An OGFC is placed as a thin lift and loses heat quickly. An OGFC should only be placed when the underlying pavement surface and ambient temperature have reached 60° F, otherwise raveling may result. Late season placement of an OGFC may prevent adequate curing of the asphalt cement and should be discouraged.

g. An OGFC should be placed full width, from outside edge to outside edge of the shoulders, to provide a cross-section with uniform frictional properties. As a minimum, it should extend 3 feet onto the shoulder. Do not place dense graded mix or curb and gutter adjacent to an OGFC. This will obstruct the lateral flow of water.

h. Handwork during placement should be minimized to avoid roughening of the surface. Rolling of an OGFC should be limited to one or two passes of an 8 to 10 ton static steel wheel roller to seat the mix. Longitudinal and transverse joints should be kept to a minimum. Joints should be butted rather than lapped.

i. Maintenance on roadways surfaced with an OGFC should avoid any activities which may obstruct the lateral flow of water through the OGFC.

(1) Traffic striping may inhibit lateral water flow if the stripe material is applied at a heavy rate or an excessive amount of reflective beads are used.

(2) Snow and ice control should be limited to plowing and chemical deicers. The use of sand or other abrasive to improve traction must be avoided.

(3) All crack and joint sealing should be performed prior to placing OGFC. When sealing is required on reflective cracks through an OGFC, only transverse joints should be sealed.

(4) Only small dense graded patches which allow for lateral flow of water through the OGFC should be considered. When larger areas of patching are involved, OGFC should be replaced with OGFC.

(5) A fog coat can be applied to an OGFC to extend the life of the asphalt binder. The fog coat is a 50 percent dilution of asphalt emulsion applied in two passes at a rate of 0.05 gallons per square yard for each pass. The use of rejuvenating agents should be avoided.

(6) When any additional overlay is required on the pavement, the existing OGFC surface must be removed.

Anthony R. Kane
Associate Administrator for Program Development

Attachment

OPEN GRADED FRICTION COURSE (OGFC)
FHWA MIX DESIGN PROCEDURE

This document combines and updates the design procedure found in Federal Highway Administration Report No. FHWA-RD-74-2, Appendix A and B and Supplements 1 & 2 to the report which were distributed by FHWA Bulletin, dated July 11, 1975. The procedure has been expanded to consider alternative equipment. A suggested laboratory report form is included at the end of the design procedure.

1.0 Material Requirements

Definitions. The grading terminology used in this design procedure is defined as follows:
Coarse Aggregate Fraction - the aggregate from each source or combined job mix formula (JMF), which ever is specified, that is retained on a No.8 sieve.
Fine Aggregate Fraction - the aggregate from each source or combined JMF, which ever is specified, that passes a No.8 sieve.
Predominant Aggregate Fraction - the aggregate from the combined JMF that passes a 3/8" sieve and is retained on a No. 4 sieve.

1.1 Aggregate. Use high quality, polish resistant aggregate with a capacity to provide and maintain good frictional characteristics. It is recommended that relatively pure carbonate aggregates or any aggregates known to polish be excluded from the coarse aggregate fraction. The coarse aggregate fraction should have at least 75 percent by weight of particles with at least two fractured faces and 90 percent with one or more fractured faces. The abrasion loss (AASHTO T 96) should not exceed 40 percent.

1.2 Mineral Filler. Mineral filler as specified in AASHTO M 17 or as specified in the State's Standard Material Specifications is suitable for OGFC design.

1.3 Gradation. The recommended gradation for OGFC is as follows:

U.S. Sieve Size	Percent Passing (by weight)
1/2"	100
3/8"	95-100
#4	30-50
#8	5-15
#200	2-5

1.4 Asphalt Cement. The recommended grade of asphalt cement is AC-20, AASHTO M 226 Table 2. Other grades of asphalt should be considered when local conditions indicate a necessity or when an improved performance can be achieved.

1.5 Asphalt Additives. Additives may be required to improve the properties of the asphalt binder to resist stripping, retard oxidation (aging) or improve temperature susceptibility. Additives routinely used by the highway agency should be acceptable for OGFC mixes. Additives which have not been previously used should be considered experimental features and examined accordingly. In either situation, all additives required for the mix must be incorporated in the mix design.

2.0 Preliminary Data

2.1 Gradation. Test the aggregate from each source, as received for the project, for gradation. If mineral filler is submitted as a separate item, it should also be tested for specification compliance. Analyze the gradation results to determine the JMF that will meet the specification limits of Section 1.3.

2.2 Specific Gravity. Separate the coarse and fine aggregate for each aggregate source and determine the bulk and apparent specific gravity of the coarse and fine aggregate fractions for each source of material submitted. Utilizing the information verified in Section 2.1, mathematically compute the bulk specific gravity (SG_b) for the coarse and fine aggregate fractions for the proposed JMF gradation. If the bulk specific gravities of the aggregate sources are significantly different, a gradation analysis based on aggregate weight will not reflect the actual particle size distribution. Re-examine the gradation of the aggregate blend on a volume basis for compliance with Section 1.3.

Compute the apparent specific gravity (SG_a) of the predominant aggregate fraction based on the proportion of predominant aggregate from each source and utilizing the specific gravity information obtained above.

2.3 Viscosity. Test the asphalt cement to be used for specification compliance with AASHTO M 226. The asphalt cement binder used for the temperature-viscosity data should include all additives proposed for the mix.

3.0 Asphalt Content

3.1 Surface Capacity. Determine the surface capacity of the predominant aggregate fraction in accordance with the following procedure (AASHTO T 270):

3.1.1 Quarter out a 105 gram sample of the predominant aggregate. Dry the sample on a hot plate or in an oven (230 ± 9°F) to a constant weight and allow the sample to cool to room temperature.

3.1.2 Reduce the sample to approximately 100.0 grams (measured to 0.1 gram) and place the sample in a metal funnel with a piece of screen (No. 10 sieve) fastened above the orifice. The suggested funnel size is top diameter 3-1/2 inches, height 4-1/2 inches, orifice 1/2 inch.

3.1.3 Completely immerse the specimen in S. A. E. No. 10 lubricating oil for 5 minutes at room temperature. [IF HIGHLY ABSORPTIVE AGGREGATE IS BEING USED, IMMERSE THE SPECIMEN FOR 30 MINUTES.]

3.1.4 Drain the sample in the funnel for 2 minutes. Place the funnel containing the sample in an oven (140 ± 5°F) for 15 minutes of additional drainage.

3.1.5 Pour the sample from the funnel into a tared pan, cool to room temperature, and reweigh the sample to the nearest 0.1 gram.

3.1.6 Compute the percent oil retained (POR) using the following equation:

$$POR = \frac{SG_a}{2.65} \times \frac{(B-A)}{A} \times 100$$

where SG_a = apparent specific gravity of the predominant aggregate
A = oven dry weight of the sample (Step 3.1.2)
B = coated weight of the sample (Step 3.1.5)

3.1.7 WHEN USING THE PROCEDURE FOR HIGHLY ABSORPTIVE AGGREGATE, AFTER DETERMINING THE POR, POUR THE SAMPLE ONTO A CLEAN DRY ABSORPTIVE CLOTH AND OBTAIN A SATURATED SURFACE DRY CONDITION.

3.1.8 POUR THE SAMPLE FROM THE CLOTH INTO A TARED PAN AND REWEIGH THE SAMPLE TO THE NEAREST 0.1 GRAM.

3.1.9 COMPUTE THE PERCENT OIL ABSORBED (POA) USING THE FOLLOWING EQUATION:

$$POA = \frac{(C-A)}{A} \times 100$$

WHERE A = DRY WEIGHT OF THE SAMPLE (STEP 3.1.2)
C = SATURATED SURFACE DRY WEIGHT OF THE SAMPLE (STEP 3.1.8)

DETERMINE THE PERCENT (FREE) OIL RETAINED (POR_A) USING THE FOLLOWING EQUATION:

$$POR_A = POR - POA$$

3.1.10 Compute the surface constant value (K_c) for the predominant aggregate using the following equation or use Figure 1 below:

$$K_c = 0.1 + 0.4(POR)$$

WHEN USING THE PROCEDURE FOR HIGHLY ABSORPTIVE AGGREGATE, THE EQUATION FOR THE K_{cA} VALUE IS:

$$K_{cA} = 0.1 + 0.4(POR_A)$$

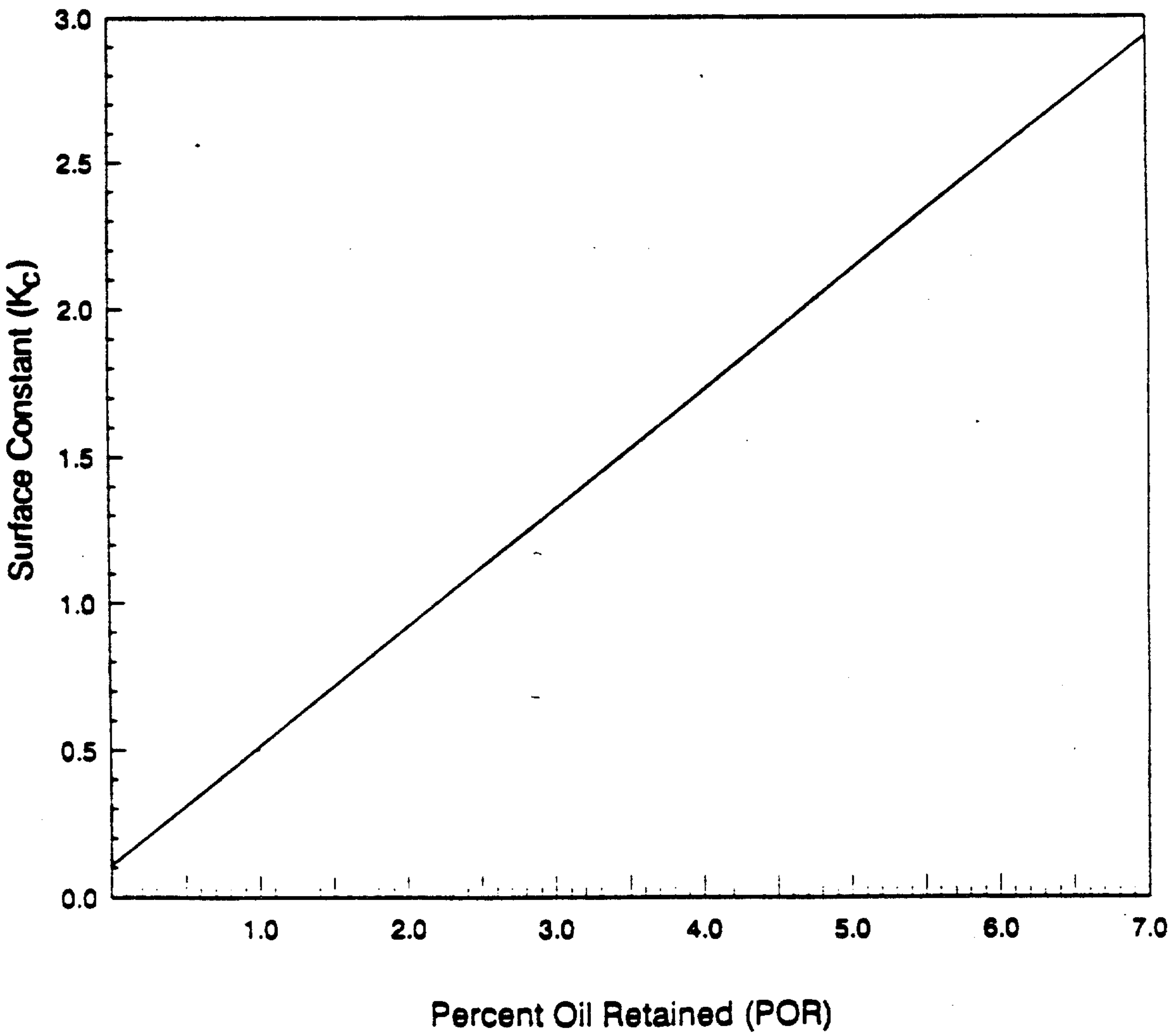

Figure 1 **SURFACE CAPACITY (K_c) GRAPH**

3.2 Asphalt Content. Compute the required JMF asphalt content (AC_{JMF}) which is based on the weight of aggregate from the following equation. The asphalt content computed from this formula is the same regardless of the asphalt grade or viscosity.

$$AC_{JMF} = (2(K_c) + 4.0) \times \frac{2.65}{SG_a}$$

WHEN USING THE PROCEDURE FOR HIGHLY ABSORPTIVE AGGREGATE, DETERMINE THE REQUIRED ASPHALT CONTENT (AC_{JMF}) AS FOLLOWS:

COMPUTE THE EFFECTIVE ASPHALT CONTENT (AC_{EFF}) FROM THE FOLLOWING EQUATION:

$$AC_{EFF} = (2(K_{ca}) + 4.0) \times \frac{2.65}{SG_a}$$

COMPLETE SECTION 4.0 AND 5.0, THEN CONTINUE WITH THE DETERMINATION OF THE ASPHALT CONTENT AS FOLLOWS:

PREPARE A TRIAL MIXTURE USING AN ASPHALT CONTENT EQUAL OR SOMEWHAT GREATER (ESTIMATE AMOUNT THAT WILL BE ABSORBED) THAN THE EFFECTIVE ASPHALT CONTENT (AC_{EFF}) DETERMINED ABOVE AND USING THE AGGREGATE GRADATION AS DETERMINED IN SECTION 5.2.

USING A SUITABLE TECHNIQUE, SUCH AS THE TEST FOR MAXIMUM SPECIFIC GRAVITY OF ASPHALT MIXTURES (AASHTO T 209), DETERMINE THE ACTUAL QUANTITY OF ASPHALT ABSORBED (IN PERCENT, BASED ON TOTAL WEIGHT OF AGGREGATE).

DETERMINE THE JMF ASPHALT CONTENT (AC_{JMF}) OF THE ABSORPTIVE MIXTURE USING THE FOLLOWING EQUATION:

$$AC_{JMF} = AC_{EFF} + \textit{actual asphalt absorbed}$$

4.0 Void Capacity of Coarse Aggregate

4.1 Unit Weight. Determine the unit weight of the coarse aggregate fraction of the proposed JMF by either of the following procedures (FHWA-RD-72-43 or ASTM D 4253 modified).

4.1.1 Apparatus

Compaction Mold. - A 6 inch nominal diameter solid-wall metal cylinder with a detachable metal base plate. A detachable metal guide-reference bar as shown in Figure 2 is required for Method 1.

Vibratory Compactor

Method 1 Rammer. - A portable electromagnetic vibrating rammer as shown in Figure 3, having a frequency of 3,600 cycles a minute, suitable for use with 115-volt alternating current. The rammer shall have a tamper foot and extension as shown in Figure 4.

Wooden Base. - A plywood disc 15 inches in diameter, 2 inches thick, with a cushion (rubber hose) attached to the bottom. The disc shall be constructed so it can be firmly attached to the base plate of the compaction mold.

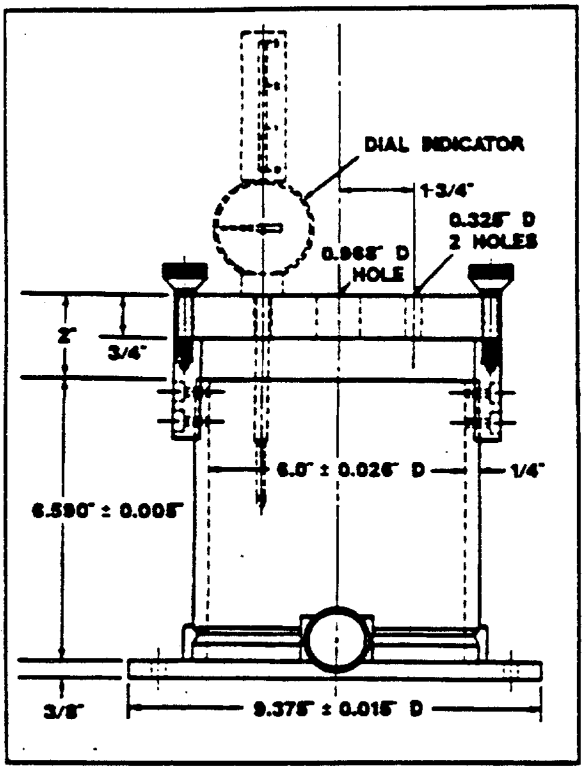

Figure 2 **COMPACTION MOLD**

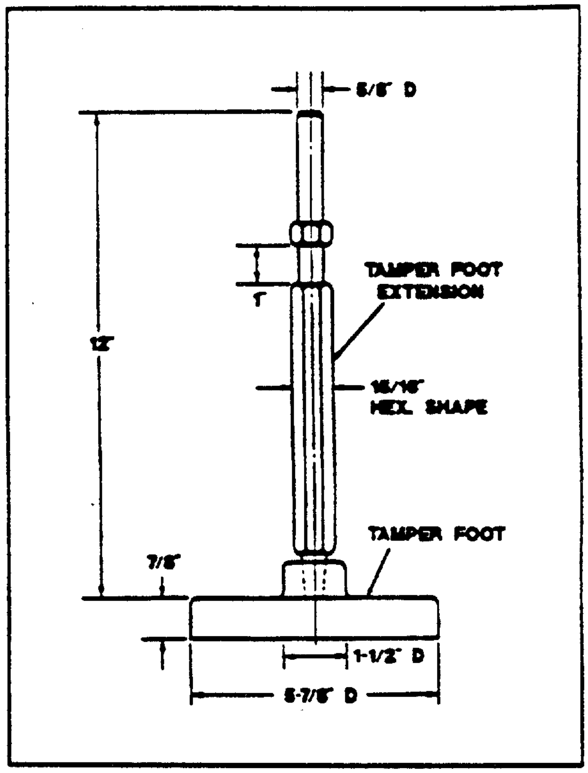

Figure 4 **TAMPER FOOT**

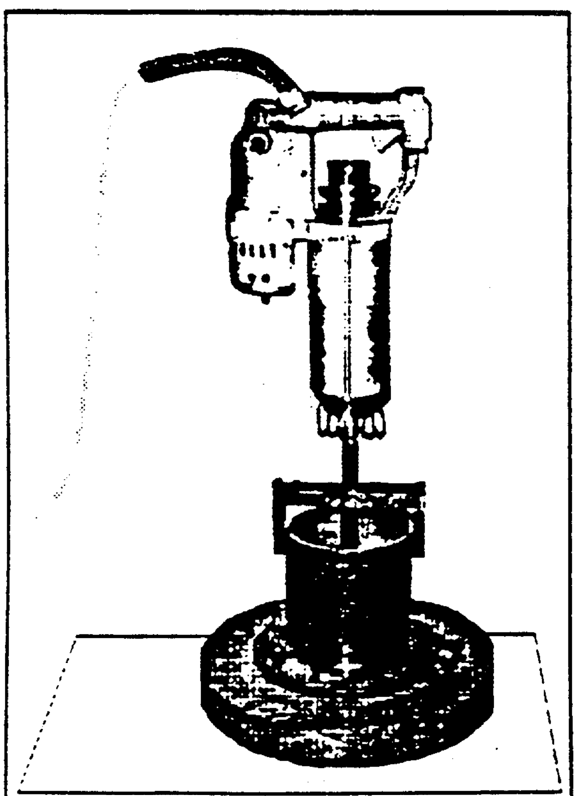

Figure 3 **VIBRATORY COMPACTION ASSEMBLY**

Method 2 (experimental) Vibrating Table. - A vibrating table capable of inducing a vibratory force to the sample at 3,600 cycles a minute and at an amplitude of (0.013 ±0.002 inch). (Soiltest CN-166 or equivalent)

Confining Load. - A circular steel disc weighing 60 pounds with a diameter of 5 7/8 inches. (Soiltest CN-167 or equivalent)

Timer. - A stopwatch or other timing device graduated in divisions of 1.0-second and accurate to 1.0-second, and capable of timing the unit for up to 2 minutes. An electric timing device or electrical circuits to start and stop the vibratory compactor may be used.

Dial Indicator - A dial indicator graduated in 0.001-inch with a travel range of 3.0 inches.

4.1.2 Sample. Select a sample of the coarse aggregate fraction (approx. 5 lb.) from the proposed JMF as verified in Section 2.1. If the bulk specific gravity of the coarse aggregate is less than 2.0, reduce the size of the sample to approximately 3.5-lb. Weigh the sample to the nearest 0.1 pound.

4.1.3 Procedure

Method 1. Pour the selected sample into the compaction mold and place the tamper foot on the sample. Place the guide-reference bar over the shaft of the tamper foot and secure the bar to the mold with the thumb screws.

Place the vibratory rammer on the shaft of the tamper foot and vibrate for 15 seconds. During the vibration period, the operator must exert just enough pressure on the hammer to maintain contact between the sample and the tamper foot.

Remove the vibratory rammer from the shaft of the tamper foot and brush any fines from the top of the tamper foot. Measure the thickness (t) of the compacted material to the nearest 0.01 inch.

Method 2. (experimental) Pour the selected sample into the compaction mold and place the surcharge base plate on the sample.

Lower the surcharge weight onto the surcharge base plate and vibrate the assembly for 2 minutes.

Remove the surcharge weight and brush any fines from the top of the surcharge base plate. Measure the thickness (t) of the compacted material to the nearest 0.01 inch.

4.1.4 Calculation. Calculate the vibrated unit weight (X) (in pounds per cubic feet) as follows:

$$X = \frac{6912\,w}{\pi\,d^2\,t}$$

Where w = weight of coarse aggregate fraction (pounds)
d = diameter of compaction mold (inches)
t = thickness of compacted specimen (inches)

4.2 Void Capacity. Determine the void capacity of the coarse aggregate (VCA) as percent of total volume using the following equation:

$$VCA = \left(1 - \frac{X}{U_c}\right) \times 100$$

Where X = vibrated unit weight from step 4.1.4
U_c = bulk dry solid unit weight of the coarse aggregate fraction (pcf).

5.0 Optimum Content of Fine Aggregate

5.1 Compute the optimum fine aggregate content with the following relationship:

$$Y = \frac{VCA - V - \dfrac{(AC_{JMF})\,(X)}{U_a}}{\dfrac{(VCA - V)}{100} + \dfrac{X}{U_f}}$$

Where Y = percent of fine aggregate by weight of total aggregate
VCA = voids in the coarse aggregate (percent)
V = design percent air voids = 15.0 percent
AC_{JMF} = asphalt content for the JMF (percent by weight of aggregate) [WHEN USING THE PROCEDURE FOR HIGHLY ABSORPTIVE AGGREGATE, USE AC_{eff} FROM SECTION 3.2, NOT AC_{JMF}]
X = vibrated unit weight of coarse aggregate (pcf)
U_a = unit weight of asphalt cement (pcf)
U_f = bulk dry solid unit weight of fine aggregate (pcf)

5.2 Compare the optimum fine aggregate content (Y) determined in Section 5.1 to the amount passing the No. 8 sieve of the proposed JMF. If these values differ by more than 1 percentage point, revise the JMF using the value determined for optimum fine aggregate content. Recompute the proportions of coarse and fine aggregates (as received) to meet the revised JMF. If the proposed and revised JMF gradations are significantly different, it may be necessary to rerun portions of this procedure.

7.0 Resistance to Effects of Water

Conduct the Immersion-Compression Test (AASHTO T 165 and T 167) on the designed mixture. Prepare samples at the optimum mixing temperature determined in Section 6.0. Use a molding pressure of 2000 psi rather than the specified value of 3000 psi. Determination of the Bulk Specific Gravity is not required for this design procedure.

After 4-day immersion at 120°F, the Index of Retained Strength shall not be less than 50 percent unless otherwise permitted. Additives to promote adhesion that will provide adequate retained strength may be used when necessary.

6.0 Optimum Mixing Temperature

Prepare a sample of aggregate (approximately 1000 grams) in the proportions determined under Section 5. Mix this sample with the proposed asphalt cement at the asphalt content (AC_{JMF}) determined under Section 3.2 at a mix temperature corresponding to an asphalt viscosity of 800 centistokes determined under Section 2.3. When the aggregate is completely coated, transfer the mixture to a pyrex glass plate (8-9 in. diameter) and spread the mixture with a minimum of manipulation. Place the plate with the sample in the oven at the mixing temperature. Observe the bottom of the plate after 60 minutes. A slight puddle of asphalt cement at the points of contact between the aggregate and the glass plate, as shown in Figure 5, is suitable and desirable after the 60 minute period. Otherwise, repeat the test at a higher or lower mixing temperature to achieve the desired contact area. If asphalt drainage occurs at a mixing temperature which is too low to provide for adequate drying of the aggregate (typically not lower than 225°F), an asphalt of a higher viscosity should be used.

An intermediate observation of the plate can be made at 15 minutes. If there is excessive drain down at the contact points, the sample can be discarded and the test repeated at a lower temperature.

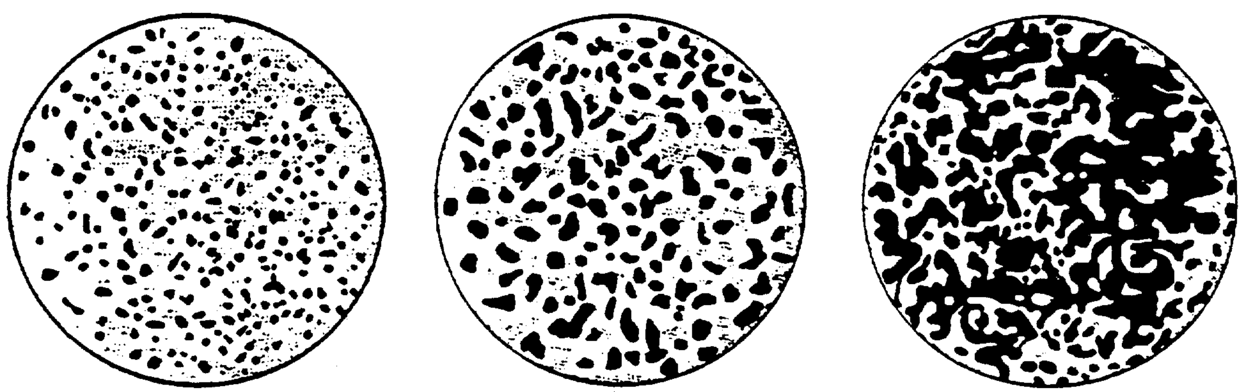

Figure 5 **DRAIN DOWN CHARACTERISTICS**

OGFC MIX DESIGN REPORT

1.0 MATERIAL PROPERTIES

A. Proposed Aggregate Proportions (by weight)

B. Proposed Job-Mix Gradation (percent passing)

Sieve Size	Specification Limit	Aggregate Sources			Job-Mix Formula
1/2"	100				
3/8"	95 - 100				
#4	30 - 50				
#8	5 - 15				
#16					
#200	2 - 5				

C. Specific Gravity - Unit Weight

COARSE AGGREGATE (Retained No. 8 Sieve)	Aggregate Sources			JMF
Bulk Sp. Gr. (SG_b)				

Bulk Solid Unit Weight (U_c) where U_c=62.4(SG_b) ________pcf

FINE AGGREGATE
(Passing No. 8 Sieve)
Bulk Sp. Gr. (SG_b) ________ ________ ________ ________

Bulk Solid Unit Weight (U_f) where U_f=62.4(SG_b) ________pcf

PREDOMINANT AGGREGATE
(Passing 3/8" - Retained No.4)
Apparent Sp. Gr. (SG_a) ________

ASPHALT BINDER
Specific Gravity @ 77.0° F. ________

Unit Weight (U_a) ________

2.0 ASPHALT CONTENT

Percent Oil Retained POR = ________

Surface Capacity K_c = ________

Asphalt Content AC_{aw} = ________% wt aggr

3.0 VOID CAPACITY

A. Void Capacity of Coarse Aggregate

Vibrated Unit Weight X = ________pcf

Voids Coarse Aggregate VCA = ________%

B. Optimum Fine Aggregate Content

Where: X = ________pcf VCA = ________%

U_f = ________pcf V = 15 %

U_a = ________pcf AC_{aw} = ________%

Specs. Limit 5 < Y < 15 Y = ________%

4.0 OPTIMUM MIXING TEMPERATURE

Asphalt Grade ________

Viscosity (cSt)	Temperature (°F)	Observed Drainage
600		
700		
800		
900		
1000		

Target Mixing Temperature ________°F

5.0 RESISTANCE TO EFFECTS OF WATER
(AASHTO T 165 & T 167, 2000 psi)

Air Dry Strength = ________psi

Wet Strength = ________psi (4 Days @ 120°F)

Retained Strength = ________% (50% Minimum)

6.0 DESIGN SUMMARY

Aggregate Proportions (by Weight)

JMF Gradation (percent passing)

Sieve Size	JMF
1/2"	
3/8"	
No. 4	
No. 8	
No. 16	
No. 200	

Asphalt Grade ________

Asphalt Additives ________

Asphalt Content = ________% wt aggr

= ________% wt mix

Mixing Temperature Range ________ to ________°F

REMARKS:

Mix Design Recommendation Accepted ___ Rejected ___

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject Automatic Profile Index Computation

Date FEB 21 1991

From Chief, Pavement Division
Washington, D.C. 20590-0001

Reply to Attn of HNG-42

To Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Recently, it has been brought to our attention that a significant difference exist between the results of an automated profilograph and a manual profilograph.

The attached South Dakota Department of Transportation report explains and analyzes the problems with automated profilographs. Some of the filtering algorithms used in the computerized profile index computations result in significantly underestimating of the profile heights for wavelengths shorter than 10 feet. Therefore, the overall profile index is also underestimated.

In view of the increased use of automated profilographs by the contractors and the States, it is important that the findings and recommendations of the attached report be shared with the divisions and States.

At the present time, we are developing a short training module to explain some of the problems in evaluating pavement profile. If you have any questions concerning this issue, please contact Mr. Aramis Lopez, Jr., at FTS 366-2084.

Louis M. Papet

Attachment

Analysis and Recommendations Concerning Profilograph Measurements on F0081(50)107 Kingsbury County

David L. Huft
South Dakota Department of Transportation
Office of Research
Pierre, South Dakota 57501-2586
(605)773-3292

November 26, 1990

I.Background

During the summer of 1990, Portland Cement Concrete pavement was placed on an 8.9 mile segment of US81 south of Arlington, South Dakota by Castle Rock Construction Company (Figure 1). The project number was F0081(50)107.

In accordance with contract provisions, the contractor conducted profilograph tests to determine the ride quality of the finished pavement. The contractor's measurements indicated that generally high ride quality had been achieved, and that the contractor was entitled to an incentive bonus of nearly $89,000.

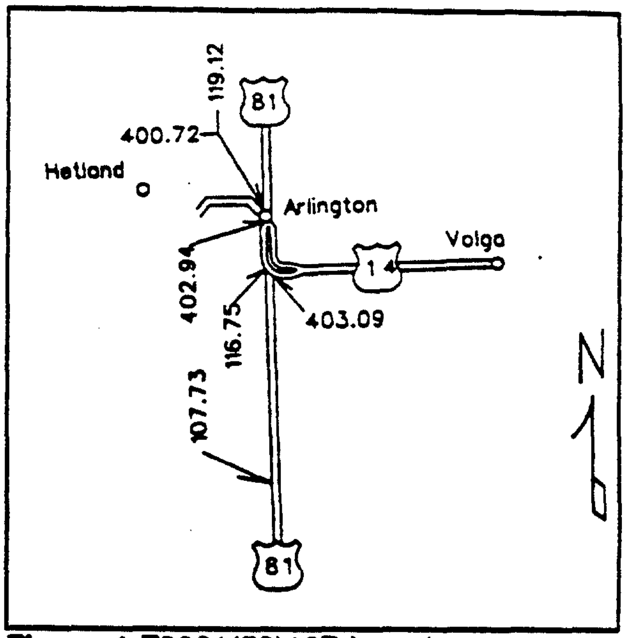

Figure 1 F0081(50)107 Location

Profilograph tests performed by the South Dakota Department of Transportation's Office of Materials and Surfacing also showed good ride quality, but not as good as the contractor's tests had indicated. Profile indexes measured during SDDOT's quality control tests were typically one to two inches per mile higher than those measured by the contractor. Traces generated by the SDDOT unit consistentently showed higher profile amplitude than did traces from the contractor's (Figure 2).

SDDOT attempted a simple check to determine whether the contractor's profilograph measured the pavement profile accurately. When the profilograph was run over a short piece of one-by-two lumber, the unit indicated approximately one-half inch rather than three-quarters of an inch, the nominal wood thickness. This unexpected result seemed to suggest a problem in the contractor's profilograph.

The contractor and SDDOT also tested sections of pavement simultaneously to determine whether their profilographs would produce the same profile indexes on the same day. On August 28, both northbound and southbound lanes were tested at stations 21+71 to 48+11 and 438+83 to 470+51. Again, SDDOT's profile indexes and trace amplitudes were higher than Castle Rock's.

The contractor attempted to verify the operation of his profilograph by comparing its performance with another unit owned by the Iowa Department of Transportation. Castle Rock's profilograph measured profile indexes which agreed closely with Iowa's. When asked why the profilograph underestimated the thickness of the one-by-two, Iowa personnel speculated that the unit's filtering algorithm might be responsible. They advised that SDDOT might evaluate traces more carefully, to avoid misinterpreting spikes as roughness.

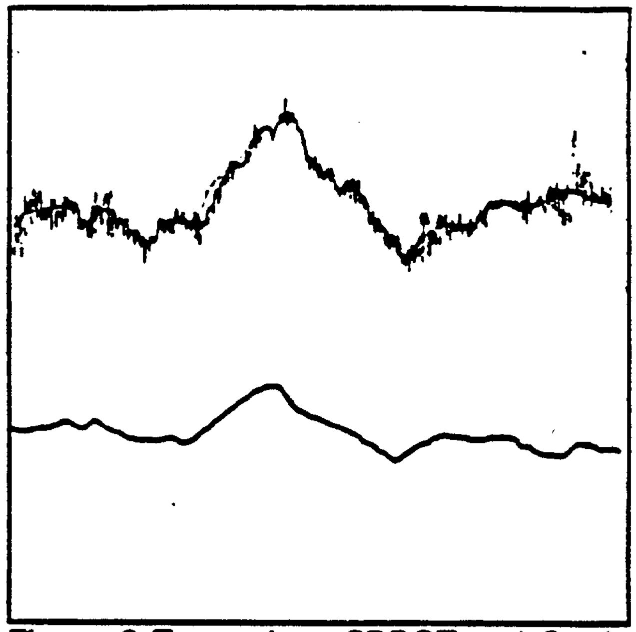

Figure 2 Traces from SDDOT and Castle Rock Profilographs

After the discrepancies were discovered, SDDOT retested the entire project. Profile indexes were consistently higher than those originally measured by the contractor. On the basis of SDDOT's measurements, the contractor was entitled to a bonus of less than $48,000. It seemed clear that the contractor's profilograph performed differently from SDDOT's profilograph, perhaps because of a filtering process performed by its on-board computer. But because SDDOT's tests were performed weeks after the contractor's, direct comparisons were not possible.

Because of the unresolved questions surrounding the profilograph measurements, the Aberdeen Region requested that the Office of Research provide technical assistance. Specifically, the objectives of this study were to:

1. Determine whether profilograph measurements obtained by the contractor's automated profilograph differ significantly from those obtained by SDDOT's manual profilograph;

2. If differences exist, determine their cause(s);

3. If differences are attributable to the filtering employed by the contractor's profilograph, develop a method to determine a fair ride quality bonus.

II. Significance of Profilograph Measurement Differences

It is essential to any analysis to first establish that the profile indexes measured by the contractor's and SDDOT's profilographs are statistically different. If observed differences only represent random variations, it would be pointless to conclude that either instrument was in error. If systematic differences exist, however, their causes might be determined. Two statistical tests were performed.

First, the project-wide profile indexes obtained by SDDOT on September 5-6 and 10-11, 1990 were compared to the profile indexes measured by the contractor within forty-eight hours of construction (Figure 3 and Figure 4). The hypothesis "Project-wide profile indexes measured by SDDOT are higher than the contractor's" was tested using the one-sided t-statistic with unknown standard deviations. That test demonstrated the hypothesis to be true with greater than 99% confidence.

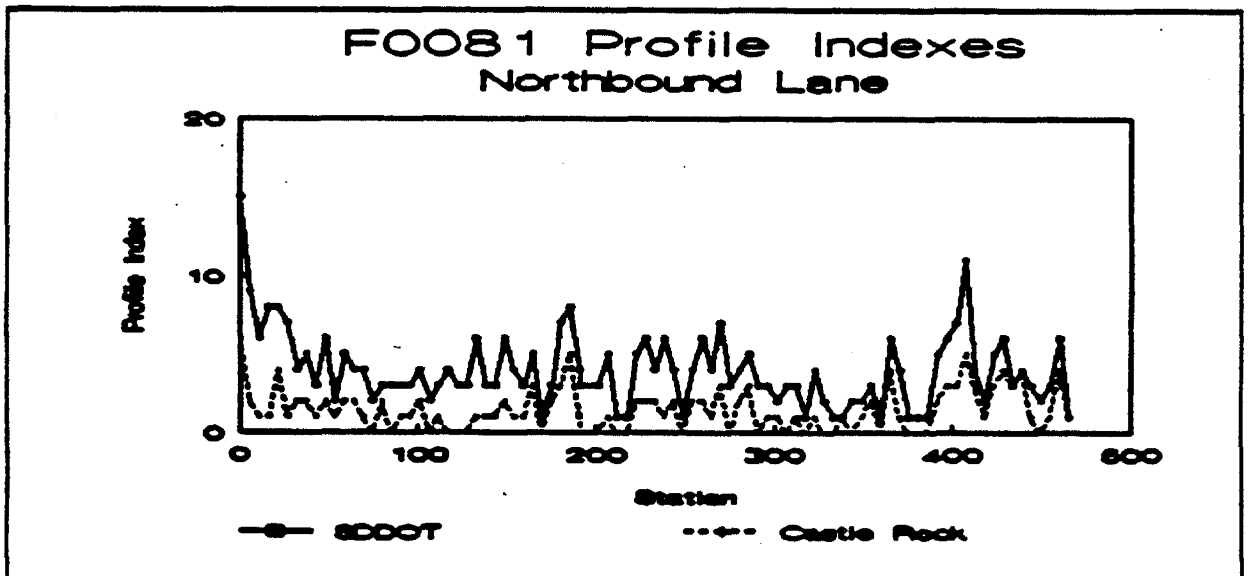

Figure 3 Project-Wide Profile Indexes Measured by SDDOT and Castle Rock Construction (Northbound Lane)

Second, the profile indexes obtained during head-to-head tests on August 28, 1990 were compared (Figure 5 and Figure 6). The hypothesis "SDDOT's profilograph generates higher profile indexes than does the contractor's" was tested, using the same statistical test. With more than 99% confidence, the hypothesis was determined to be true.

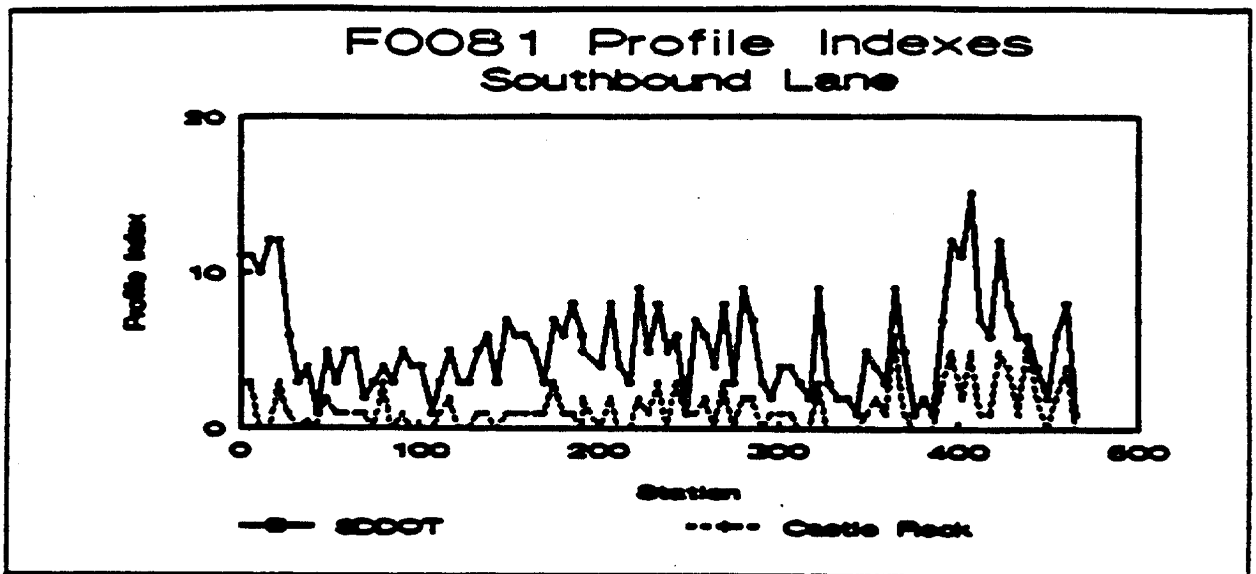

Figure 4 Project-Wide Profile Indexes Measured by SDDOT and Castle Rock Construction (Southbound Lane)

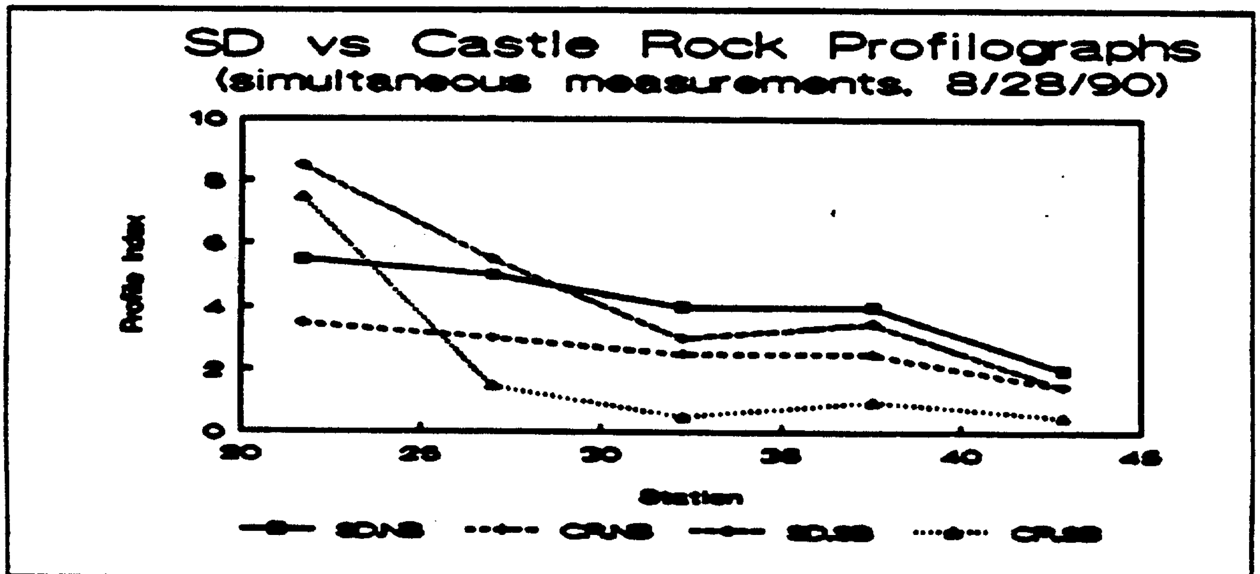

Figure 5 Profile Indexes Measured Simultaneously by SDDOT and Castle Rock Construction on August 28, 1990

On the basis of these statistical tests, it can be confidently stated that SDDOT's project wide measurements are higher than the contractor's and SDDOT's profilograph generates higher profile indexes than does the contractor's.

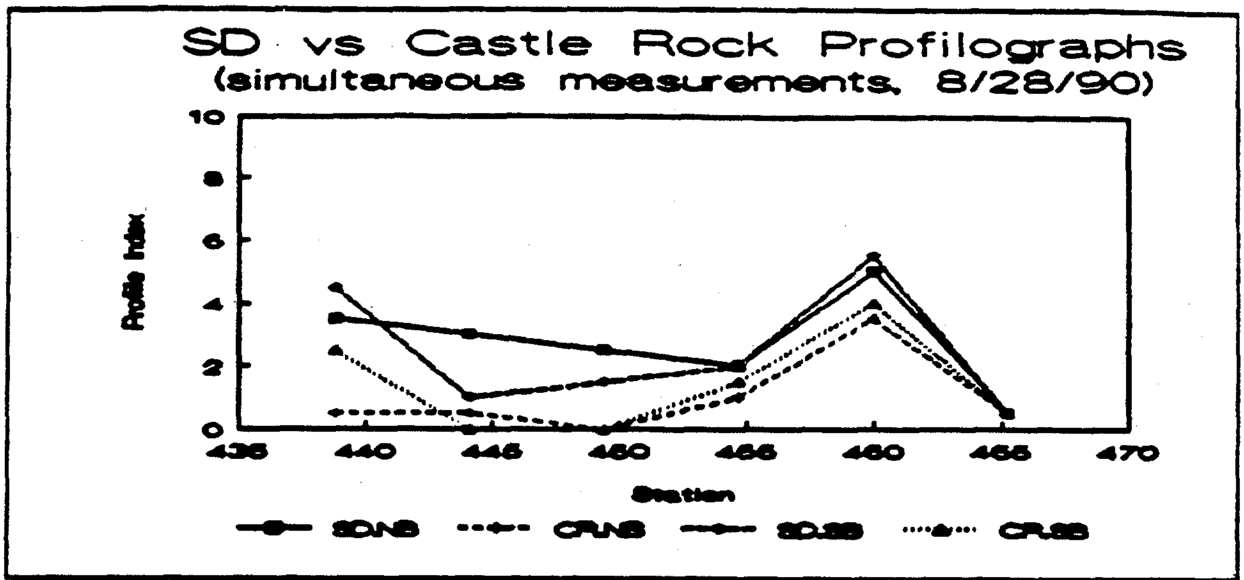

Figure 6 Profile Indexes Simultaneously Measured by SDDOT and Castle Rock Construction on August 28, 1990

IV. Profile Index Difference Causes

Two causes were considered likely to explain the differences between the profile indexes measured by SDDOT and those measured by the contractor. They were time between measurements (in the case of project-wide indexes) and differences between manual and automated profile interpretation.

A. Time Between Measurements

Although it seemed probable that differences between the profilographs were responsible for much of the discrepancy between SDDOT's and the contractor's profile index measurements, it also appeared that pavement roughness had changed since the time of paving. Indexes obtained by the contractor's profilograph on August 28 were higher than those taken immediately after construction, suggesting that the pavements had become slightly rougher following construction. This is reasonable, because the curing process and temperature changes can easily effect slab shape.

This observed change is important, because it means SDDOT measurements taken long after construction should not be used to determine ride quality bonuses. There can be no assurance that SDDOT's measured profile indexes accurately represent the ride quality immediately following construction.

B. Profilograph Differences

From the comparison of profile index measurements taken on the same pavement sections on August 28, 1990, it was clear that the contractor's profilograph measured lower profile indexes than did SDDOT's. Because the two machines are geometrically identical, the profile filtering process incorporated in the contractor's unit was considered the most likely cause of the difference.

(It should be noted that SDDOT's manual procedures were also evaluated, primarily because of Iowa's concern that spikes may have been incorrectly interpreted. However, no incorrect procedures were discovered. SDDOT's engineer had correctly smoothed the profile so spikes were ignored, just as Iowa had advised.)

Castle Rock's unit is a James Cox and Sons, Inc. Model CS8200 profilograph. It includes an on-board computer which digitizes the profile signal at 1.3" intervals and computes profile index automatically. To make profile interpretation less difficult, the computer uses a simple recursive digital filter to remove spikes (caused by extraneous mechanical vibrations) from the profile signal.

Mathematically, the filter is of the form

$$Y_n = AY_{n-1} + BX_n$$

where X_n is the raw (unfiltered) digitized elevation at point n, Y_n is the filtered elevation at point n, and Y_{n-1} is the filtered elevation at point n-1. A and B are constants which determine the filter's effect, and are defined:

$$B = N/65536$$

$$A = 1 - B$$

Cox recommends using a filter factor of N=8000 for most purposes.

The performance of this filter can be analysed with standard signal processing techniques. One useful analysis determines the response of the filter as a function of profile wavelength. Specifically, the analysis defines the filter's amplitude response $H(\lambda)$, which is the ratio of the filter's output to its input. It can be shown analytically that the amplitude response of this filter is given by the formula

$$H(\lambda)=\frac{B}{\sqrt{C^2+D^2}}$$

where

$$C=1-A\cos\left(\frac{2\pi\lambda_s}{\lambda}\right)$$

and

$$D=A\sin\left(\frac{2\pi\lambda_s}{\lambda}\right)$$

λ_s is the sampling interval of 1.3" used by the Cox profilograph.

As shown in Figure 7, the filter attenuates shortest wavelengths most. Wavelengths shorter than one foot are attenuated by more than 80%, so the effect of spikes will be reduced greatly. However, the filter also attenuates significantly longer wavelengths as well. Wavelengths of two feet are attenuated by over 60%; five foot wavelengths are attenuated by 30%. Even ten foot wavelengths are attenuated by 10%.

The filter can also be described in terms of its response to a step function. As Figure 8 shows, the filter is quite slow to recognize a step in the pavement profile. After the profilograph travels one foot past the step, it measures only 70% of the step's height. It is not until the profilograph has traveled three feet past the step that it measures 95% of the true height. This explains why the profilograph failed to measure the correct thickness of the one-by-two, which was less than one foot long.

The significance of Figure 7 and Figure 8 is that while the filter successfully removes spikes from the raw profile, it also removes longer features which are known to affect the pavement's ride quality. In other words, the filtering underestimates the amplitude of the pavement profile, causing estimates of profile index to be low.

Cox acknowledges that the profile index is affected by the filter. Their manual states, "It is important to understand that the test results are heavily affected by the selected filter factor." However, the filter's performance is fundamentally a consequence of its simple, first order formulation. Regardless of the filter factor used in the computation, the filter's selectivity will not be good. Invariably, long wavelengths will be removed along with the short. The performance of the filter could be improved by using a higher order filtering

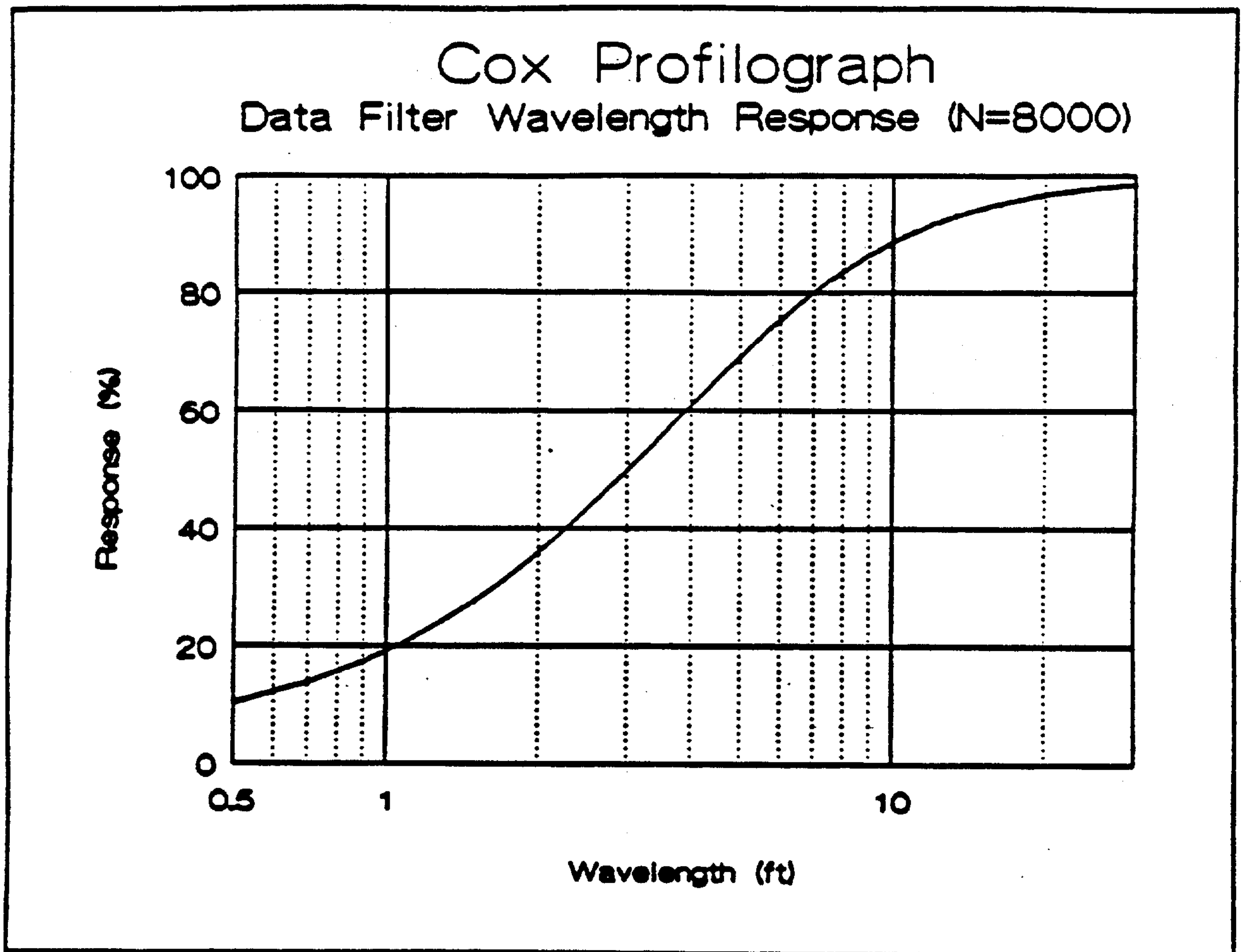

Figure 7 Response of Cox Data Filter as a Function of Wavelength

algorithm, if the profilograph's computer had sufficient power.

The filter's ultimate effect on computed profile index cannot be simply determined. Because the amount of attenuation depends upon the wavelengths present in the pavement profile, the effect on profile index will also vary. If a pavement contains predominantly short wavelengths, the profile index will be reduced greatly. If only longer wavelengths are present, the reduction will be slight.

This behavior explains why Castle Rock's automated profilograph correlated well with Iowa's manual profilograph. Iowa's test section consists of large amplitude bumps at predominantly long wavelengths (longer than twenty feet). At these wavelengths, the effect of filtering is too slight to be detected by visual inspection of profilograph traces.

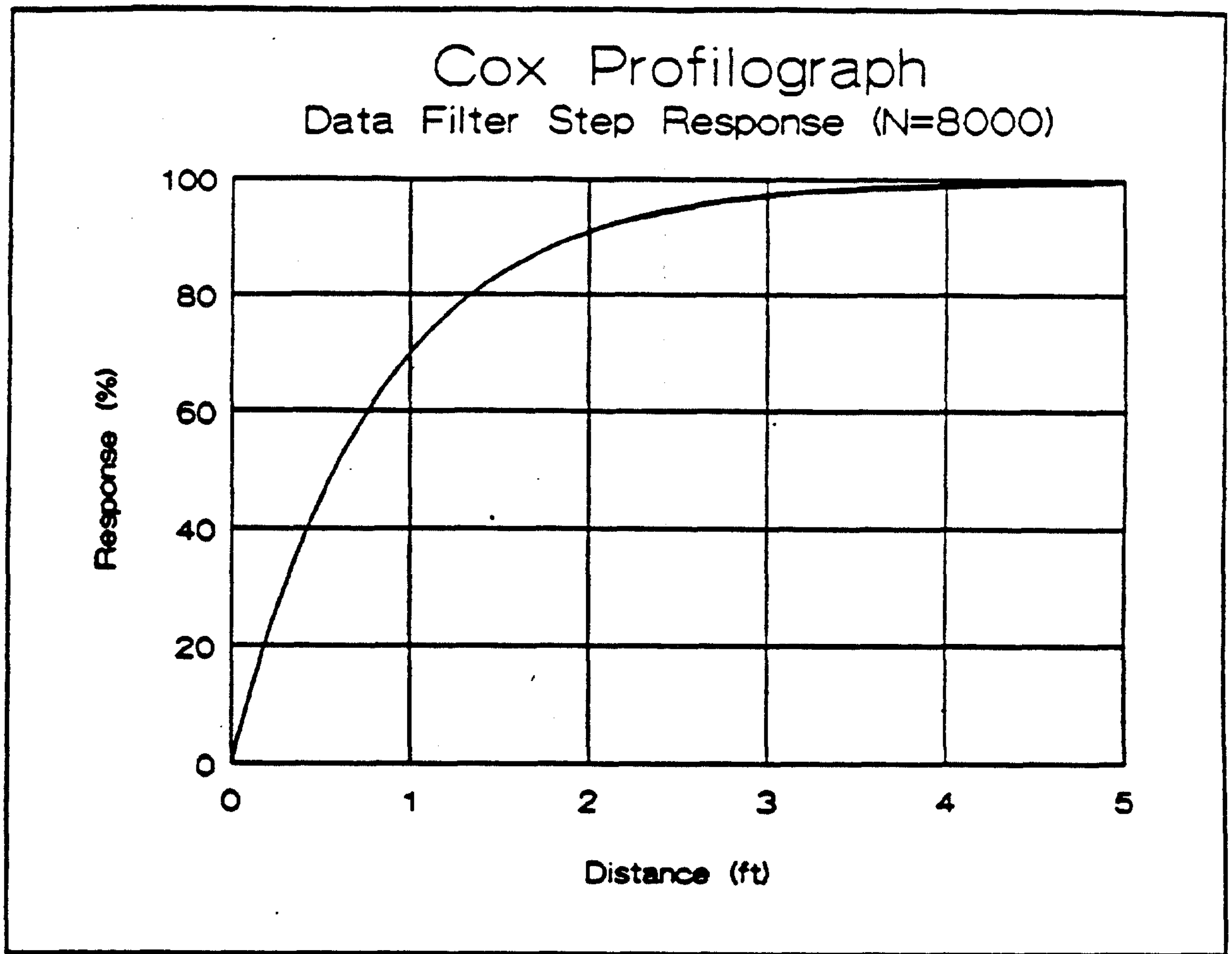

Figure 8 Cox Filter Step Response

III. Bonus Computation

In view of the discrepancies between profilograph measurements taken by SDDOT and those taken by Castle Rock, the question of fair bonus payments arises. Because the contractor's profilograph underestimated the height of profile features on the pavement, profile indexes were artificially low, and inconsistent with the measurement method assumed in the specifications. Consequently, the bonuses computed from the profile indexes were excessive. SDDOT's profilograph did not underestimate the profile, but because measurements were not taken within forty-eight hours of paving, they cannot be used directly as a basis for bonus payment.

In the interest of fairness to the contractor and the state, it would be best to correct the contractor's measurements somehow to remove the adverse effects of filtering. This could be done directly, but would require complete redigitizing of all profile traces taken from the contractor's profilograph. The procedure would use complex mathematics (Fourier transforms and inverse Fourier transforms) to reconstruct a profile of proper amplitude.

An alternative to this difficult and lengthy process is to correlate the two profilographs, using the measurements which were taken by both instruments on August 28. The resulting regression equation can then be used to adjust the contractor's profile indexes so they better represent unfiltered values.

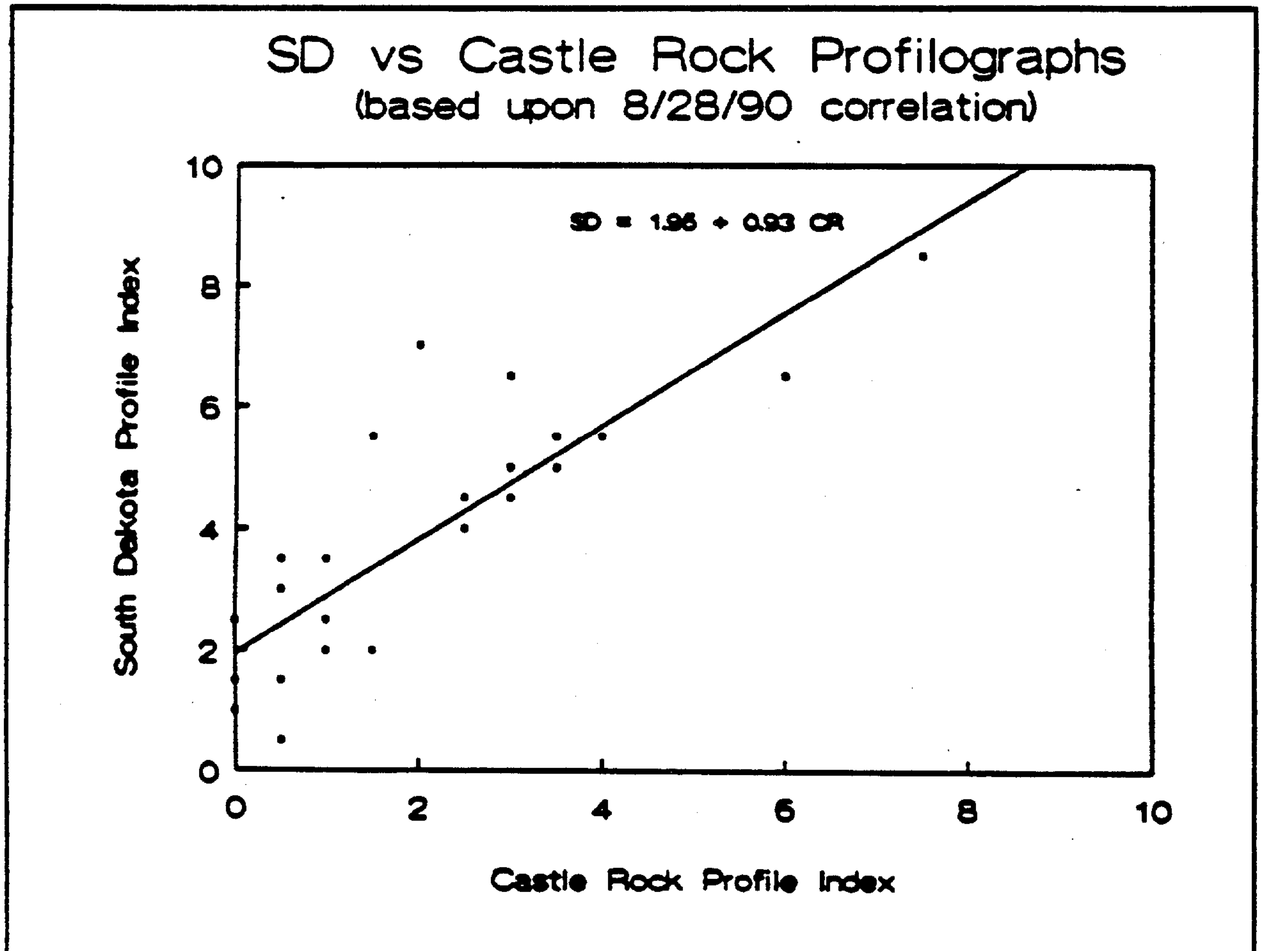

Figure 9 Correlation of SDDOT Profile Index vs Castle Rock Profile Index

Figure 9 shows the profile indexes measured by the two profilographs. From the data obtained in the simultaneous testing, the best equation relating filtered (Castle Rock) and unfiltered (SDDOT) profile indexes is:

$$PI_{unfiltered}=1.95+0.93\,PI_{filtered}$$

Application of this equation to the contractor's profile indexes produces adjusted values which more realistically represent the ride quality of the pavements. The bonus computed from these adjusted values total $68,975.36, which coincidentally falls approximately halfway between the contractor's original estimate and SDDOT's estimate derived from late measurements. A complete listing of SDDOT, contractor, and adjusted profile indexes, along with bonuses computed from them, is presented in Appendix A.

IV.Conclusions and Recommendations

Comparisons between profile indexes measured by the South Dakota Department of Transportation and Castle Rock Construction Company show that the profilographs used for the measurements are inconsistent with each other. Analysis of the filtering algrorithm used on the contractor's profilograph demonstrates that the Cox unit significantly underestimates profile heights at wavelengths shorter than ten feet. Therefore, profile indexes are also underestimated.

It was possible to derive a regression equation from profile indexes measured by the SDDOT unit and the contractor's unit on the same pavement sections on the same day. The contractor's profile indexes were adjusted using this equation, yielding new indexes which more realistically describe the ride quality achieved in the paving operation. The bonus computed from the adjusted profile indexes is approximately midway between the bonus computed from the contractors unadjusted indexes and the bonus computed by SDDOT from its late measurements.

It is clear that the filtering algorithm incorporated in the Cox unit should not be used in conjunction with SDDOT's special provision for paving incentives. Because the filtering falsely indicates a smoother pavement, its use (with the present special provision) is unfair to the state and to other paving contractors. To be fair, one or more of the following is recommended:

> Prohibit filtering. This might make automatic profile index computation impossible, if the computer requires smoothed profiles. The benefits of objectivity and quick test results would be lost.
>
> Lower the acceptable inches per mile if filtering is used. Additional correlation between filtered and unfiltered profile indexes, especially at higher roughness levels, may be desirable if this method is adopted.

Improve the filtering algorithm by using a higher order filter. This option would place additional computational demands upon the profilograph's on-board computer, but would provide much better selectivity between short and intermediate wavelengths.

A final recommendation concerns the criterion by which SDDOT and other transportation agencies determine whether a contractor's profilograph operates acceptably. It is common practice to accept a contractor's profile indexes if they fall within two inches per mile of the state's indexes. This criterion fails if a profilograph generates indexes which are within two inches per mile, but consistently low (or high). A statistical test to determine whether mean profile indexes are different would provide a much better indication of measurement validity, and would ensure fairness to both the contractor and the state.

In view of the increased use of automated profilographs, and ongoing development of new units by other vendors, it is important that the issues raised in this report be addressed prior to the 1991 construction season.

NORTHBOUND						
Station	State		Contractor		Adjusted	
	PI	Bonus	PI	Bonus	PI	Bonus
0+59 - 5+87	15	($533.87)	6	$106.77	7	$0.00
5+87 - 11+15	9	$0.00	2	$533.87	4	$320.32
11+15 - 16+43	6	$106.77	1	$533.87	3	$427.09
16+43 - 21+71	8	$0.00	1	$533.87	3	$427.09
21+71 - 26+99	8	$0.00	4	$320.32	6	$106.77
26+99 - 32+27	7	$0.00	1	$533.87	3	$427.09
32+27 - 37+55	4	$320.32	2	$533.87	4	$320.32
37+55 - 42+83	5	$213.55	2	$533.87	4	$320.32
42+83 - 48+11	3	$427.09	1	$533.87	3	$427.09
48+11 - 53+39	6	$106.77	2	$533.87	4	$320.32
53+39 - 58+67	2	$533.87	1	$533.87	3	$427.09
58+67 - 63+95	5	$213.55	2	$533.87	4	$320.32
63+95 - 69+23	4	$320.32	2	$533.87	4	$320.32
69+23 - 74+51	4	$320.32	1	$533.87	3	$427.09
74+51 - 79+79	2	$533.87	0	$533.87	2	$533.87
79+79 - 85+07	3	$427.09	2	$533.87	4	$320.32
85+07 - 90+35	3	$427.09	0	$533.87	2	$533.87
90+35 - 95+63	3	$427.09	1	$533.87	3	$427.09
95+63 - 100+91	3	$427.09	1	$533.87	3	$427.09
100+91 - 106+19	4	$320.32	2	$533.87	4	$320.32
106+91 - 111+47	2	$533.87	0	$533.87	2	$533.87
111+47 - 116+75	3	$427.09	1	$533.87	3	$427.09
116+75 - 122+03	4	$320.32	0	$533.87	2	$533.87
122+03 - 127+31	3	$427.09	0	$533.87	2	$533.87
127+31 - 132+59	3	$427.09	0	$533.87	2	$533.87
132+59 - 137+87	6	$106.77	1	$533.87	3	$427.09
137+87 - 143+15	3	$427.09	1	$533.87	3	$427.09
143+15 - 148+43	3	$427.09	1	$533.87	3	$427.09
148+43 - 153+71	6	$106.77	2	$533.87	4	$320.32
153+71 - 158+99	4	$320.32	1	$533.87	3	$427.09
158+99 - 164+27	3	$427.09	1	$533.87	3	$427.09
164+27 - 169+55	5	$213.55	3	$427.09	5	$213.55
169+55 - 174+83	1	$533.87	0	$533.87	2	$533.87
174+83 - 180+11	3	$427.09	2	$533.87	4	$320.32
180+11 - 185+39	7	$0.00	3	$427.09	5	$213.55
185+39 - 190+67	8	$0.00	5	$213.55	7	$0.00

190+67 - 195+95	4	$320.32	1	$533.87	3	$427.09
195+95 - 201+23	3	$427.09	0	$533.87	2	$533.87
201+23 - 206+51	3	$427.09	0	$533.87	2	$533.87
206+51 - 211+79	5	$213.55	1	$533.87	3	$427.09
211+79 - 217+07	1	$533.87	0	$533.87	2	$533.87
217+07 - 222+35	1	$533.87	0	$533.87	2	$533.87
222+35 - 227+63	5	$213.55	2	$533.87	4	$320.32
227+63 - 232+91	6	$106.77	2	$533.87	4	$320.32
232+91 - 238+19	4	$320.32	2	$533.87	4	$320.32
238+19 - 243+47	6	$106.77	1	$533.87	3	$427.09
243+47 - 248+75	4	$320.32	2	$533.87	4	$320.32
248+75 - 254+03	1	$533.87	0	$533.87	2	$533.87
254+03 - 259+31	4	$320.32	2	$533.87	4	$320.32
259+31 - 264+59	6	$106.77	2	$533.87	4	$320.32
264+59 - 269+87	4	$320.32	1	$533.87	3	$427.09
269+87 - 275+15	7	$0.00	3	$427.09	5	$213.55
275+15 - 280+43	3	$427.09	0	$533.87	2	$533.87
280+43 - 285+71	4	$320.32	2	$533.87	4	$320.32
285+71 - 290+99	5	$213.55	3	$427.09	5	$213.55
290+99 - 296+27	3	$427.09	0	$533.87	2	$533.87
296+27 - 301+55	3	$427.09	1	$533.87	3	$427.09
301+55 - 306+83	2	$533.87	1	$533.87	3	$427.09
306+83 - 312+11	3	$427.09	0	$533.87	2	$533.87
312+11 - 317+39	3	$427.09	1	$533.87	3	$427.09
317+39 - 322+67	1	$533.87	0	$533.87	2	$533.87
322+67 - 327+95	4	$320.32	1	$533.87	3	$427.09
327+95 - 333+23	2	$533.87	0	$533.87	2	$533.87
333+23 - 338+51	1	$533.87	0	$533.87	2	$533.87
338+51 - 343+79	1	$533.87	1	$533.87	3	$427.09
343+79 - 349+07	2	$533.87	0	$533.87	2	$533.87
349+07 - 354+35	2	$533.87	1	$533.87	3	$427.09
354+35 - 359+63	3	$427.09	2	$533.87	4	$320.32
359+63 - 364+91	1	$533.87	0	$533.87	2	$533.87
364+91 - 370+19	6	$106.77	4	$320.32	6	$106.77
370+19 - 375+47	4	$320.32	1	$533.87	3	$427.09
375+47 - 380+75	1	$533.87	0	$533.87	2	$533.87
380+75 - 386+03	1	$533.87	0	$533.87	2	$533.87
386+03 - 391+31	1	$533.87	0	$533.87	2	$533.87
391+31 - 396+59	5	$213.55	2	$533.87	4	$320.32
396+59 - 401+87	6	$106.77	3	$427.09	5	$213.55

401+87 - 407+15	7	$0.00	3	$427.09	5	$213.55
407+15 - 412+43	11	($106.77)	5	$213.55	7	$0.00
412+43 - 417+71	4	$320.32	3	$427.09	5	$213.55
417+71 - 422+99	2	$533.87	1	$533.87	3	$427.09
422+99 - 428+27	5	$213.55	3	$427.09	5	$213.55
428+27 - 433+55	6	$106.77	4	$320.32	6	$106.77
433+55 - 438+83	3	$427.09	3	$427.09	5	$213.55
438+83 - 444+11	4	$320.32	4	$320.32	6	$106.77
444+11 - 449+39	3	$427.09	1	$533.87	3	$427.09
449+39 - 454+67	2	$533.87	0	$533.87	2	$533.87
454+67 - 459+95	3	$427.09	1	$533.87	3	$427.09
459+95 - 465+23	6	$106.77	4	$320.32	6	$106.77
465+23 - 470+51	1	$533.87	1	$533.87	3	$427.09
NB Subtotal		$28,401.82		$44,417.57		$33,740.27

SOUTHBOUND						
Station	State		Contractor		Adjusted	
	PI	Bonus	PI	Bonus	PI	Bonus
0+59 - 5+87	11	($106.77)	3	$427.09	5	$213.55
5+87 - 11+15	11	($106.77)	3	$427.09	5	$213.55
11+15 - 16+43	10	$0.00	0	$533.87	2	$533.87
16+43 - 21+71	12	($213.55)	0	$533.87	2	$533.87
21+71 - 26+99	12	($213.55)	3	$427.09	5	$213.55
26+99 - 32+27	6	$106.77	1	$533.87	3	$427.09
32+27 - 37+55	3	$427.09	0	$533.87	2	$533.87
37+55 - 42+83	4	$320.32	1	$533.87	2	$533.87
42+83 - 48+11	1	$533.87	0	$533.87	2	$533.87
48+11 - 53+39	5	$213.55	2	$533.87	4	$320.32
53+39 - 58+67	3	$427.09	1	$533.87	3	$427.09
58+67 - 63+95	5	$213.55	1	$533.87	3	$427.09
63+95 - 69+23	5	$213.55	1	$533.87	3	$427.09
69+23 - 74+51	2	$533.87	1	$533.87	3	$427.09
74+51 - 79+79	3	$427.09	0	$533.87	2	$533.87
79+79 - 85+07	4	$320.32	3	$427.09	5	$213.55
85+07 - 90+35	3	$427.09	0	$533.87	2	$533.87
90+35 - 95+63	5	$213.55	1	$533.87	3	$427.09
95+63 - 100+91	4	$320.32	0	$533.87	2	$533.87
100+91 - 106+19	4	$320.32	0	$533.87	2	$533.87
106+91 - 111+47	1	$533.87	0	$533.87	2	$533.87
111+47 - 116+75	3	$427.09	1	$533.87	3	$427.09

116+75 - 122+03	5	$213.55	2	$533.87	4	$320.32
122+03 - 127+31	3	$427.09	0	$533.87	2	$533.87
127+31 - 132+59	3	$427.09	0	$533.87	2	$533.87
132+59 - 137+87	5	$213.55	1	$533.87	3	$427.09
137+87 - 143+15	6	$106.77	1	$533.87	3	$427.09
143+15 - 148+43	3	$427.09	0	$533.87	2	$533.87
148+43 - 153+71	7	$0.00	1	$533.87	3	$427.09
153+71 - 158+99	6	$106.77	1	$533.87	3	$427.09
158+99 - 164+27	6	$106.77	1	$533.87	3	$427.09
164+27 - 169+55	5	$213.55	1	$533.87	3	$427.09
169+55 - 174+83	3	$427.09	1	$533.87	3	$427.09
174+83 - 180+11	7	$0.00	3	$427.09	5	$213.55
180+11 - 185+39	6	$106.77	1	$533.87	3	$427.09
185+39 - 190+67	8	$0.00	1	$533.87	3	$427.09
190+67 - 195+95	6	$106.77	0	$533.87	2	$533.87
195+95 - 201+23	5	$213.55	2	$533.87	4	$320.32
201+23 - 206+51	4	$320.32	0	$533.87	2	$533.87
206+51 - 211+79	8	$0.00	2	$533.87	4	$320.32
211+79 - 217+07	4	$320.32	0	$533.87	2	$533.87
217+07 - 222+35	3	$427.09	0	$533.87	2	$533.87
222+35 - 227+63	9	$0.00	2	$533.87	4	$320.32
227+63 - 232+91	5	$213.55	1	$533.87	3	$427.09
232+91 - 238+19	8	$0.00	3	$427.09	5	$213.55
238+19 - 243+47	5	$213.55	0	$533.87	2	$533.87
243+47 - 248+75	6	$106.77	3	$427.09	5	$213.55
248+75 - 254+03	1	$533.87	1	$533.87	3	$427.09
254+03 - 259+31	7	$0.00	1	$533.87	3	$427.09
259+31 - 264+59	6	$106.77	2	$533.87	4	$320.32
264+59 - 269+87	4	$320.32	0	$533.87	2	$533.87
269+87 - 275+15	8	$0.00	3	$427.09	5	$213.55
275+15 - 280+43	3	$427.09	0	$533.87	2	$533.87
280+43 - 285+71	9	$0.00	2	$533.87	4	$320.32
285+71 - 290+99	7	$0.00	2	$533.87	4	$320.32
290+99 - 296+27	3	$427.09	0	$533.87	2	$533.87
296+27 - 301+55	2	$533.87	1	$533.87	3	$427.09
301+55 - 306+83	4	$320.32	1	$533.87	3	$427.09
306+83 - 312+11	4	$320.32	1	$533.87	3	$427.09
312+11 - 317+39	3	$427.09	0	$533.87	2	$533.87
317+39 - 322+67	2	$533.87	0	$533.87	2	$533.87
322+67 - 327+95	9	$0.00	3	$427.09	5	$213.55

327+95 - 333+23	3	$427.09	0	$533.87	2	$533.87
333+23 - 338+51	2	$533.87	0	$533.87	2	$533.87
338+51 - 343+79	2	$533.87	0	$533.87	2	$533.87
343+79 - 349+07	1	$533.87	0	$533.87	2	$533.87
349+07 - 354+35	5	$213.55	1	$533.87	3	$427.09
354+35 - 359+63	4	$320.32	2	$533.87	4	$320.32
359+63 - 364+91	3	$427.09	1	$533.87	3	$427.09
364+91 - 370+19	9	$0.00	6	$106.77	7	$0.00
370+19 - 375+47	5	$213.55	1	$533.87	3	$427.09
375+47 - 380+75	1	$533.87	0	$533.87	2	$533.87
380+75 - 386+03	2	$533.87	0	$533.87	2	$533.87
386+03 - 391+31	1	$533.87	0	$533.87	2	$533.87
391+31 - 396+59	7	$0.00	3	$427.09	5	$213.55
396+59 - 401+87	12	($213.55)	5	$213.55	7	$0.00
401+87 - 407+15	11	($106.77)	2	$533.87	4	$320.32
407+15 - 412+43	15	($533.87)	5	$213.55	7	$0.00
412+43 - 417+71	7	$0.00	1	$533.87	3	$427.09
417+71 - 422+99	6	$106.77	1	$533.87	3	$427.09
422+99 - 428+27	12	($213.55)	5	($213.55)	7	$0.00
428+27 - 433+55	8	$0.00	4	$320.32	6	$106.77
433+55 - 438+83	6	$106.77	1	$533.87	3	$427.09
438+83 - 444+11	6	$106.77	6	$106.77	7	$0.00
444+11 - 449+39	4	$320.32	2	$533.87	4	$320.32
449+39 - 454+67	2	$533.87	0	$533.87	2	$533.87
454+67 - 459+95	6	$106.77	2	$533.87	4	$320.32
459+95 - 465+23	8	$0.00	4	$320.32	6	$106.77
465+23 - 470+51	1	$533.87	0	$533.87	2	$533.87
SB Subtotal		$19,539.46		$44,204.02		$35,236.09
TOTAL		$47,941.08		$88,621.59		$88,975.38

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE **167**

MEASUREMENTS, SPECIFICATIONS, AND ACHIEVEMENT OF SMOOTHNESS FOR PAVEMENT CONSTRUCTION

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Carmichael, California

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TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. NOVEMBER 1990

8.7.01

MEASUREMENTS, SPECIFICATIONS, AND ACHIEVEMENT OF SMOOTHNESS FOR PAVEMENT CONSTRUCTION

SUMMARY

The concern about the smoothness of highway surfaces precedes the development of motorized vehicles. In the early days, the simple straightedge was used as the sole indicator of smoothness. But even before the turn of the century, efforts were directed at developing improved devices for smoothness evaluation. From 1900 to near midcentury, numerous devices of increasing complexity were invented. These were primarily mechanical devices with elaborate multi-wheeled support systems. Advances in several technological fields have now been applied to smoothness-measuring equipment, resulting in the incorporation of electrical circuitry, electronics, ultrasonics, lasers, and computerization.

Although the early devices were primarily of concern to the practicing engineer, the advent of test road construction brought the research engineer onto the scene. Many devices were developed in connection with specific research efforts. The automotive industry became interested because of the effect that certain types of pavement had on motor vehicles. In recent years highway managers have recognized that the public rates a highway primarily on its riding characteristic. Thus it is necessary to program an increasing amount of highway funds to address the issue of pavement smoothness on a system-wide basis.

As a consequence, several smoothness-measuring devices have been developed and are in current use. The fundamentals of operation, cost, and appropriateness to address a specific need vary considerably. Certain devices are far better suited than others to the purpose of controlling the smoothness of newly constructed pavements. Therefore, it is important for those concerned with obtaining smoothness in construction to be aware of the equipment best suited for that purpose and the relation of that equipment to the entire spectrum of smoothness-measuring devices.

Smoothness-measuring equipment currently used in new pavement construction includes straightedges (static and rolling), profilographs, response-type road-roughness-measuring systems, and inertial profilometers. All agencies use a straightedge—a few as the sole approach to smoothness control, but most as an adjunct to other equipment. The type of instrument receiving increased application is the profilograph, either the California or Rainhart type. These devices are similar in that they portray graphically certain characteristics of pavement smoothness, are relatively simple mechanical devices, can be used on new concrete pavement surfaces soon after construction, are low-cost/low-maintenance devices, and provide information that is readily acceptable by specifying agencies and the construction industry. Profilographs provide an analog trace to which specification tolerances are applied. The traces can be used to locate specific pavement features in the field. The primary disadvantages with this type of instrument are the slow speed of operation (3 mph) and the time required for evaluating the profiles, although the latter item has been addressed by computerized models that are now available. Other disadvantages include the exaggeration and suppression of parts of the surface wavelength spectrum, the occasional

exclusion by the blanking band of surface irregularities that may be of importance, and a mediocre correlation to other reference roughness standards.

Other devices being used in evaluating smoothness of new construction, including response-type road-roughness-measuring systems and inertial profilometers, are used considerably less often than profilographs for a variety of reasons. They are not able to be used on concrete pavements for a considerable time after paving (i.e., until the concrete gains sufficient strength), they don't allow ready identification or location of pavement surface aberrations, and, in some cases, they are very costly items. However, they can operate at high speeds; thus a considerable amount of data can be obtained at a lower cost. Also, the smoothness statistic is achieved with little or no manual processing. High-speed equipment has its greatest application in entire highway system assessment, research applications, and for calibration purposes.

Numerous research efforts as well as symposia and workshops have been directed toward providing information on the use of smoothness-measuring equipment. Although there are vast differences in equipment types and their ultimate application, the relationships of several smoothness indexes have been compared and are reasonably well defined.

A survey of practices in use in the United States and Canada revealed that there is a wide diversity in the use of smoothness specifications and equipment. However, emphasis on smoothness by specifying agencies, together with strong support from the construction industry, has led to the attainment of increasingly smoother pavements.

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4. Title and Subtitle A HALF CENTURY WITH THE CALIFORNIA PROFILOGRAPH Phase I Experiment	5. Report Date February,1992 6. Performing Organization Code
7. Author(s) Larry A. Scofield, Sylvester Kalevela, Mary Anderson, Asm Hossain	8. Performing Organization Report No.
9. Performing Organization Name and Address Arizona Transportation Research Center College of Engineering, ERC, Rm 405 Arizona State University	10. Work Unit No. 11. Contract or Grant No. HPR-PL-1(41) ITEM 114
12. Sponsoring Agency Name and Address ARIZONA DEPARTMENT OF TRANSPORTATION 206 S. 17TH AVENUE PHOENIX, ARIZONA 85007	13. Type of Report & Period Covered Final-Sept. 1990 - June 1991 14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. Abstract

This study was performed to establish equipment and operator variability for mechanical and computerized California profilographs. Future work, based on testing conducted during this study, should develop precision and bias statements for profilographs.

The research consists of two phases. Phase I, reported herein, provided a literature review, performed the field testing and conducted the statistical anlysis. The historical development of the profilograph and California test procedures and specifications were evaluated in relationship to todays incentive/disincentive specifications. Additionally, equipment parameters which influence test variability were reviewed.

Two field experiments were conducted. The first experiment, designed to evaluate variability, consisted of a 4x4x2 randomized block design with replication. Two levels of pavement roughness, four operators, and four profilogrphs were utilized. The second experiment, designed to evaluate the effects of data filter settings on profile index obtained with computerized profilographs, consisted of a 3x2x2x2 randomized block design with replication. Two levels of pavement roughness, two computerized profilographs, two operataors, and three data filter settings were used.

The results of the study indicated that the average repeatability was 0.75 inches/mile and 0.56 inches/mile for the rough and smooth track conditions, respectfully.

The average repeatability for an operator performing trace reduction was 0.94 inches/mile for one device and 1.72 inches/mile for a second device.

The data filter setting used on computerized profilographs has a significant affect on the resulting profile index. For each 1000 unit change in the data filter setting, a 7% reduction in the profile index was obtained when compared to the manufacturers recommended value of 8000.

23. Registrant's Seal

13499
LARRY A. SCOFIELD

17. Key Words Profilograph, Pavement Roughness, Pavement Smoothness, California Profilograph, Profile, Specifications, Incentive, Disincentive	18. Distribution Statement Document is available to the U.S. public through the National Technical Information Service, Springfield, Virginia 22161

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CHAPTER 9

PAVEMENT MANAGEMENT

National Perspective on Pavement Management

FRANK BOTELHO

The nation's highway network represents a multibillion dollar investment that allows for the essential movement of people and goods.

Sound decisions on preventive maintenance, rehabilitation, and reconstruction of highway pavements are crucial to protecting that investment. For this reason, Pavement Management Systems (PMS) have become increasingly important and are now federally mandated on all Federal-aid highways. PMS provide valuable assistance to decision makers in determining cost-effective strategies for providing and maintaining pavements in serviceable condition.

History of PMS

Unlike other management systems that have begun in recent years, PMS were started two decades ago. Although they have made steady progress since that time, they are still new compared with other institutional functions such as planning, design, construction, maintenance, and research.

By the mid-1980s PMS were proving themselves and the benefits were being documented. By the end of the 1980s more than half the states were developing or implementing PMS. In 1989 the Federal Highway Administration (FHWA) issued a policy requiring all states to have a PMS that would cover principal arterials under the states' jurisdiction. It was therefore apparent to FHWA that a PMS was needed by all to ensure the cost-effective expenditure of Federal-aid funds.

The scope of federal and state involvement in PMS expanded when Congress passed the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) and required all states to have a PMS that covers all Federal-aid highways. The most significant aspect of this law was the expanded network coverage. FHWA's 1989 policy covered 313,700 centerline miles and ISTEA approximately tripled that coverage, increasing it to 916,200 centerline miles. This expanded coverage translates into a need for significant coordination among state and local governments. For example, of the total of 916,200 miles covered, 365,200 are under local jurisdiction.

In December 1993, FHWA issued a regulation covering all management systems. Section 500, Subpart B, of the regulation describes the ISTEA requirements for PMS. The following items are noteworthy:

1. The regulation is nonprescriptive;
2. Federal-aid funds are eligible for the development, implementation, and annual operation of a PMS;
3. States must develop their work plan by October 1994, designed to meet the implementation requirements;
4. Standards are included for the National Highway System (NHS);
5. The PMS for the NHS must be fully operational by October 1995;
6. The states have full flexibility to develop the standards for the PMS that cover the non-NHS routes;
7. The PMS for non-NHS routes must be fully operational by October 1997; and
8. PMS information must be used as input into the development of the metropolitan and statewide transportation plans and improvement programs.

Section 500.207, PMS Components, contains the components of a PMS for highways on NHS. There are three primary components: data collection, analyses, and update. The components under data collection include

1. *Inventory:* physical pavement features including the number of lanes, length, width, surface type, functional classification, and shoulder information;
2. *History:* project dates and types of construction, reconstruction, rehabilitation, and preventive maintenance;
3. *Condition survey:* roughness or ride, pavement distress, rutting, and surface friction;
4. *Traffic:* volume, vehicle type, and load data; and
5. *Data base:* compilation of all data files used in the PMS.

The components under analyses include

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1. *Condition analysis:* ride, distress, rutting, and surface friction;

2. *Performance analysis:* pavement performance analysis and an estimate of remaining service life;

3. *Investment analysis:* an estimate of network and project level investment strategies. These include single- and multi-year period analyses and should consider life-cycle cost evaluation;

4. *Engineering analysis:* evaluation of design, construction, rehabilitation, materials, mix designs, and maintenance; and

5. *Feedback analysis:* evaluation and updating of procedures and calibration of relationships using PMS performance data and current engineering criteria.

Advantages of PMS

A PMS involves a systematic approach that supplies quantifiable engineering information to help highway engineers and administrators manage highway pavements. The total decision-making process is based on information from PMS coupled with engineering experience, budget constraints, scheduling parameters, management prerogatives, public input, political considerations, and planning and programming factors.

The purpose of a PMS is to enhance the way an agency manages and engineers the preservation of its pavement network. A PMS brings to the table "condition data," the past, present, and predicted future condition of the pavement network. Coupled with inventory, project history, and cost data, a PMS can perform a myriad of engineering, management, and investment analyses.

A PMS helps provide the engineering justification for a multiyear network-level pavement preservation program. It can be used to measure the cost-effectiveness of the preservation program and in doing so it can determine the value added to the assets. When all the information in a PMS is analyzed (including key items such as the remaining service life), an agency can determine if it is meeting its own goals. Some basic questions a PMS should answer are

- Is the network in acceptable condition according to the agency's policy?
- Is the trend in condition staying the same, improving, or declining?
- Is there a backlog, and if so, how large is it?

A PMS should explore and seize opportunities to extend the service life of pavements—a major investment in the future of the nation's infrastructure. This goal can be accomplished by using the information in a PMS data base (i.e., performance data) to evaluate how well pavements are designed, constructed, and maintained. The quality of engineering and the materials used are of the utmost importance because these factors determine the rate at which pavements deteriorate. In general terms, a PMS should help accomplish work more efficiently and provide a way to measure how well it is carried out.

ISTEA requires that states have pavement management systems covering all Federal-aid highways, many of which are under local jurisdiction.

PMS Perspective

The following is an item-by-item perspective on current practices, future trends, and common hurdles in PMS.

Inventory

Most, if not all, states have an inventory of the physical features that are on the surface of the pavement (i.e., number of lanes, length, width, surface type, functional classification, and shoulder information). A number of states are lacking information on features that lie below the surface because of the time and expense involved in coring the pavement. The newest proven technology being used by the states to measure pavement layer thicknesses is ground-penetrating radar. When calibrated and using computer analysis, ground-penetrating radar can measure pavement layer thickness within plus or minus 5 percent for materials that have different dielectric constants. State-of-the-art equipment operates at highway speeds that makes it fast, safe, and cost-effective.

Project History

Most states do not have a complete project history (i.e., preventive maintenance, rehabilitation, and reconstruction data) for the NHS. Maintenance information is the weakest link. Most states have recently developed, or are in the process of developing, a PMS file for preventive maintenance activities. In cases for which it is impractical to resurrect the pavement history because of time, labor, and cost, agencies are now beginning to track the project history.

Roughness

The technology for measuring pavement roughness at the network level generally began with response-type devices, followed by ultrasonic and visible optical devices. The future trend is toward infrared optical and laser profile devices.

Rutting

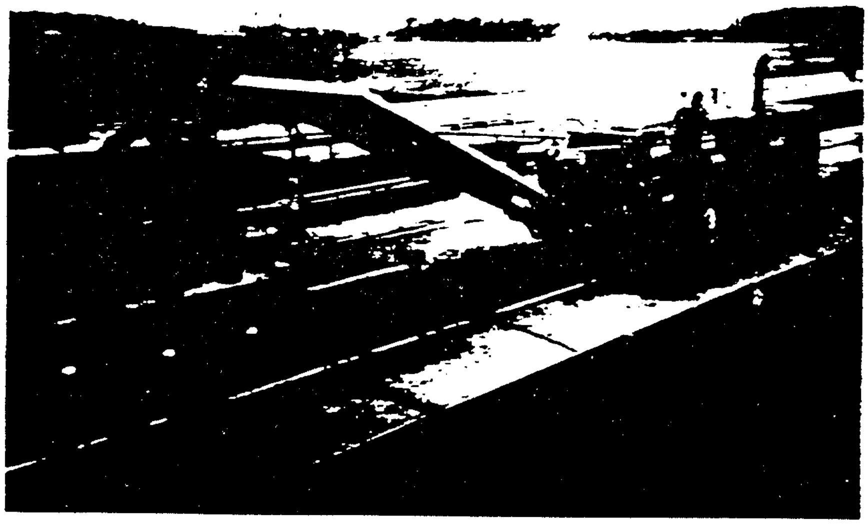
When PMS was first introduced 15 to 20 years ago, rutting was measured using straight edges and string lines. During the past 10 years, most state highway agencies (SHA) have acquired automated devices that measure rutting at highway speeds. These are typically ultrasonic devices with either three or five sensors. There are two other devices: one has 19 ultrasonic sensors and another has 11 lasers.

Cracking

In general, cracking is the distress that "drives" most PMS. For many years, cracks were measured using trained survey crews who walked or drove on the pavement. There are two types of driven surveys: slow and highway speeds (typically 40 to 50 mph). Currently, various SHAs use 35-mm film and super VHS video to photograph the surface of the pavement. The film and videos are then viewed on a monitor at an office workstation by a trained observer who performs the distress survey.

Viewing a film or video at an office workstation is safer and more convenient than conducting a walking field survey. However, pavement management engineers using walking surveys are able to detect more low-severity distresses than they can by watching a film or video survey because of its limited resolution.

A number of PMS engineers believe the optimum system is a fully automated approach that uses the science of pattern recognition. This type of system videotapes the pavement surface, enhances the images using gray scales and pattern recognition, and counts the cracks using computer software and algorithms. The obvious advantages of this type of system are high-speed data processing, safety, labor savings, and consistent data. Fully automated systems have now been developed, including one by the Texas Department of Transportation.



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Pavement management systems provide valuable help in determining cost-effective strategies for providing and maintaining pavements in serviceable condition.

Structural Carrying Capacity

Only a handful of states are currently measuring the structural carrying capacity of their pavements at the network level using deflection measurements. Network-level measurements are not intended to have the same degree of accuracy as project design measurements. States that collect network-level data have shown them to be good general indicators of the overall carrying capacity of the network. These types of data and analysis can flag attention to special situations; for example when certain roads appear to have less carrying capacity than needed. Stationary deflection-measuring devices do not lend themselves to network-level PMS because the process is slow and costly. In the future, PMS will need a deflection-measuring device that operates at or near highway speeds. The deflection measurements obtained from a "rolling deflectometer," as it is known, and the pavement layer thicknesses obtained from the ground-penetrating radar, are used to compute the structural carrying capacity of the pavement.

Performance

Most states have the raw data needed to monitor and predict pavement performance, which is typically measured as condition or serviceability over a period of time. Currently half the states have performance curves, one-quarter are in the process of developing performance, and the remainder are not yet active. Excellent off-the-shelf software packages that PMS engineers can use for regression analysis are available. In the future, these software packages, coupled with today's high-speed and ever-more-powerful PCs, will enable PMS engineers to track and predict performance on a "route-specific" basis. This capability has already been proven and put into operation in at least some SHAs.

Traffic and Load Data

PMS need average daily traffic flow maps and equivalent single-axle load (ESAL) flow maps on a route-specific basis. Currently all SHAs have traffic flow maps. However, few SHAs have or can produce ESAL flow maps. Most traffic-collection procedures are geared toward collecting traffic volumes, which are primarily used by highway engineers and planners for capacity analysis. Until PMS came along, there was no need to collect traffic data for load analysis on a route-specific basis. Unfortunately for PMS engineers, collecting load data on a route-specific basis is more expensive than the existing traffic-collection process and it is not known if the additional expense (which has not been calculated for each state) is justifiable. More study is needed on this topic. Many PMS engineers and planners believe that better traffic- and load-prediction models are needed.

Ranking Projects

The backbone and heart of a PMS is its ability to rank in priority order pavement preservation projects that are justifiable and cost-effective. The most important phrase in the new (December 1993) FHWA regulation on management systems is the requirement that PMS for NHS produce "a prioritized list of recommended candidate projects with recommended preservation treatments that span single-year and multi-year periods using life-cycle cost analysis." Currently most state PMS do not produce a multiyear ranked list of projects with recommended treatments using life-cycle cost analysis, but are expected to have this capability in the future.

Remaining Service Life

Determining "remaining service life" is a requirement in the new regulation for NHS. Currently only 10 SHAs perform this analysis, but in the future it is anticipated that most will find this an unencumbered task. It is important to monitor the long-range health of a network and this analysis enables managers and programmers to maintain a "steady state" in their multiyear workload and budget.

Relational Data Base

A PMS cannot automatically, systematically, consistently, and efficiently function without a "relational data base" because the amount and complexity of data cannot be computed manually for a typical state PMS. Currently half the SHAs have relational data bases, one-quarter are developing them, and the remainder are not active at the present time. Given the state-of-the-art capabilities in relational data-base management systems, it is anticipated that most SHAs will have relational data bases in the near future.

Uniformity

Currently there is little-to-no uniformity among the states in the way they measure, collect, and report PMS condition data. The reason is that all states developed their PMS independently. This independence, of course, has many advantages for designing a PMS to meet the needs and objectives of any agency. But states are at a disadvantage when communicating with each other about basic condition information such as roughness, rutting, and cracking. They will find a lack of uniformity, which means that they cannot communicate or help each other to enhance this area of PMS. Efforts are under way and accomplishments have been made by ASTM and the Road Profiler Users Group (RPUG) that deserve commendation. The other management systems such as bridge and safety already have national standards for data collection and reporting.

PMS will benefit if the 50 states, Puerto Rico, and the District of Columbia agree to adopt more uniform methods to collect and report condition data. Future efforts by ASTM; RPUG; Strategic Highway Research Program, Long-Term Pavement Performance; FHWA; and the American Association of State Highway and Transportation Officials' Task Force on Pavements are aimed in that direction.

In-House and Outside Resources

Pavement management is a procedure that includes a wide variety of technical components. Some of these require a high degree of technical skill to develop and implement, whereas others require a high concentration of effort to establish. Each agency should carefully and objectively weigh its in-house capabilities, and if it does not have the resources, it should seriously consider seeking assistance from a consultant or a university. In the long run, it will save a lot of time and money and result in a better final product.

Staffing

The biggest problem the states face in developing, implementing, updating, and operating a PMS is staffing. There is a significant shortage of people who understand PMS. Once employees are trained and gain some experience, they are often promoted or transferred to other jobs. For the past five years, the annual turnover rate of state PMS engineers has been approximately 25 percent. The [illegible] incentives for early retirements have fueled that rate in the past two years. Generally, most SHAs have only one person who oversees the management and daily operation of the complete PMS program, and when that person leaves, most often the PMS shuts down. This situation occurs quite frequently and because of the current budget constraints and staffing ceilings in most highway agencies, it is not likely to improve. Unfortunately there is no quick fix to this problem.

Future Implementation of PMS

In gauging the future success of implementing PMS as called for in ISTEA, organizations must first decide whether they are serious about PMS. If so, and the commitment is made to do the work, supply the resources, and use the system, then PMS use is likely to be successful.

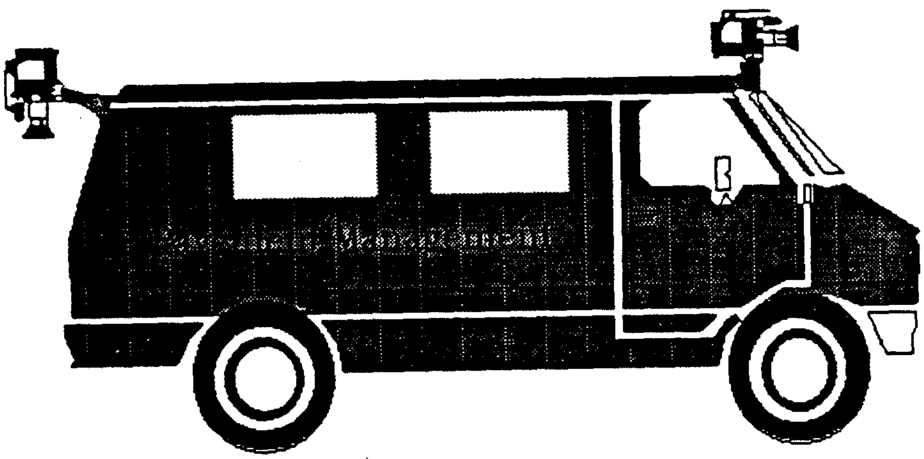
Students in the nation's colleges and universities will provide the life blood for PMS in the future. Currently 24 such institutions offer courses on PMS, but more are needed. FHWA and SHAs should support academia in providing more education about PMS and other management systems.

The largest institutional obstacle facing PMS today is acceptance by all managers and engineers in all agencies (including federal, state, and local). The reasons for this are many. The future holds more hard work for those who are serious about pavement management.

AUTOMATED PAVEMENT

CONDITON DATA

COLLECTION EQUIPMENT



Resource Paper

FHWA Pavement Division

July, 1989

NOTICE

This paper was prepared in the interest of technology sharing. It is not intended to be an all-inclusive discussion of the pavement data collection equipment available today.

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PAVEMENT MANAGEMENT

Our nation's pavement network, much like the national population, is growing older. Resources to meet the needs of pavement preservation continue to fall short of existing needs, and the gap can be expected to widen as we approach the twenty-first century.

In order to address pavement needs, many agencies have turned toward a systematic process for Pavement Management. The AASHTO Guidelines on Pavement Management define Pavement Management as "the effective and efficient directing of the various activities involved in providing and sustaining pavements in a condition acceptable to the travelling public at the least life cycle cost." Simply stated, the purpose of Pavement Management is to get the most bang for the buck.

An effective Pavement Management System encompasses many of the disciplines within an agency's organization. These may include planning, programming, budgeting, data collection, design, type selection, construction, materials, research, maintenance, monitoring, and performance evaluation. The Pavement Management System draws from these disciplines, providing feedback to assess the adequacy of important decisions such as selected rehabilitation alternatives. Through the systematic Pavement Management System process, the individual disciplines may be constantly evaluated, refined, and improved.

In order to preserve pavements in a cost effective manner, the pavement condition for the agency's entire system, or network, must be known and periodically monitored. The condition of all pavements deteriorates with time and traffic loading (Figure 1).

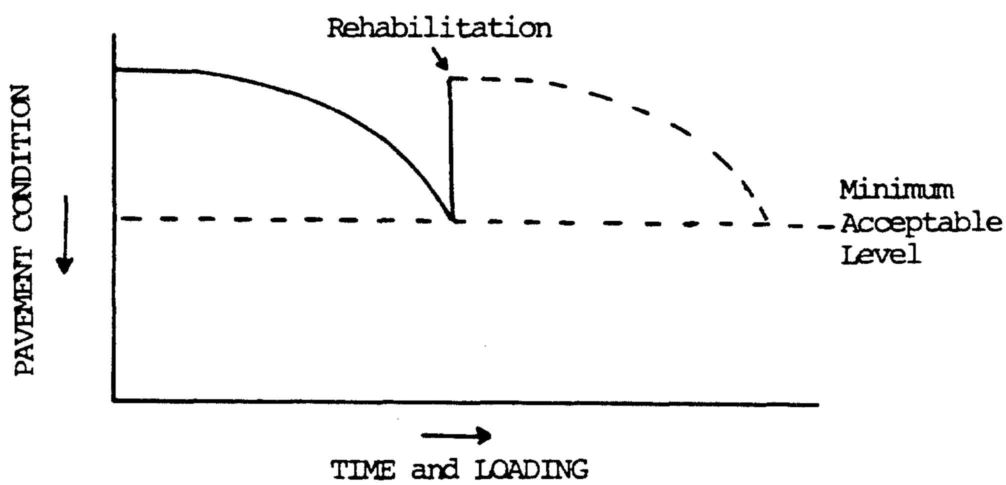


Figure 1 - Pavement Condition vs. Time and Loading

Following initial construction, pavement condition deteriorates slowly at first, then more rapidly as load applications are added. At some time after initial construction, the condition deteriorates to a minimum acceptable level at which time the pavement is rehabilitated. The condition is improved to a point above its minimum acceptable level, depending on the type and extent of the rehabilitation performed. This process continues indefinitely for every pavement section on the network at varying rates depending on variables such as design, construction, soils, materials, drainage, environment, loading, etc.

The minimum acceptable level varies depending on the classification of the pavement. For example, the minimum acceptable level for a heavily travelled Interstate facility can be expected to be considerably above the minimum acceptable level of a local service road.

If an agency is to make the most effective use of its scarce resources, then maintenance, rehabilitation, and reconstruction should be performed at the proper time (Figure 2). As the slope

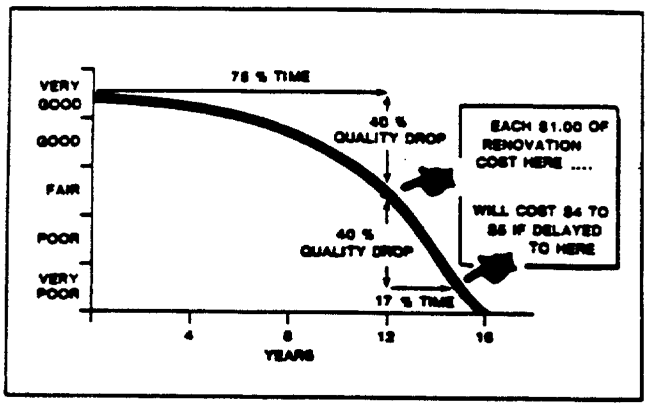


Figure 2 - Pavement Rehabilitation Cost vs. Time Performed

of the pavement deterioration curve changes from a horizontal line (zero) to a line approaching vertical (infinity), the incremental cost to repair an equal number of additional loads increases considerably. In a very short period of time the cost to rehabilitate the pavement to an equivalent level of serviceability may double, triple, or more.

Effective Pavement Management begins with the collection of the most reliable, consistent, and objective pavement condition data obtainable. The technological explosion of the past 10 years has permitted a tremendous improvement in the types, accuracy, repeatability, reliability, and degree of automation of available equipment to collect this data. Significant improvements are now being made annually, and can be expected to continue in the years ahead, especially in the area of automated crack detection technology.

CONDITION SURVEYS

Pavement condition data is collected by means of a condition survey. In past years, condition survey teams, made up of trained raters, walked or drove along the pavement and recorded observations of the pavement condition (Figure 3). On some networks, this technique is still in use.

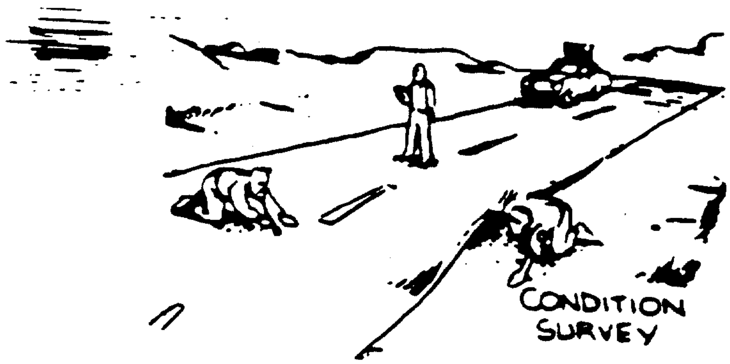


Figure 3 - Condition Survey Team

Depending on the size of the network, one or more teams obtain the condition data. This data may be collected either for an entire network or statistically representative samples of the network. The productivity, accuracy, repeatability, reliability, and sampling intervals are related to the team's speed and the amount of data required. The limitation of this type of survey is its slow speed, the data collected is subjective and often varies from one observer to another, and the team members are exposed to traffic. Variations in the data collected inevitably result.

The use of high speed automated equipment is becoming more prevalent. While human observers are more versatile and creative than automated equipment, machines are fast, objective, tireless, consistent, and generally less disruptive to traffic because they travel at high speeds. Some State highway agencies (SHAs) have been moving toward automated pavement data acquisition. This trend is expected to increase in the years ahead.

AUTOMATED DATA COLLECTION

Most SHAs rely on four important pavement condition measurements to determine priorities for maintenance, rehabilitation, or reconstruction. These measurements generally include skid resistance, deflection, roughness, and distress. This paper provides an overview the state-of-the-art practice in automated data collection equipment.

The concept of automated collection of these and other desired data is not a new one. A roughness device was developed as early as 1923. The Bureau of Public Roads (now the Federal Highway Administration) standardized the BPR Roughometer, a devise to measure ride, in 1940. Automated procedures have been used to measure skid resistance since the 1940's, structural capacity since the 1950's, and distress since the early 1970's. New technology has been developed for various uses and is now being adapted to pavement evaluation. Examples are the use of non-contact transducers (sonic and ultrasonic probes, incandescent light, and lasers), ground penetrating radar, stress waves, still photography, thermal infrared photography, and video technology. Each of these areas is undergoing continuous change and constant improvement as more public and private funding is provided to develop the technology.

A SHA may realize cost and time savings through the proper selection and application of automated equipment for Pavement Management. Larger networks may realize greater savings by using automated devices. The most commonly used devices to collect skid, deflection, roughness, and distress data, and the equipment which may be seen in the future is described in the following sections. Appendix A contains a partial list of the commercially available equipment to collect pavement condition data.

Equipment for Skid Data Collection

Pavement skid resistance or surface friction is measured to evaluate pavement safety. Skid resistance varies with many factors such as pavement material, texture, aggregate type and amount of polish, temperature, type and amount of foreign material such as rubber, oil, grease, and dust on the pavement surface, water film thickness, and tire type, condition, inflation, tread pattern, and material composition.

Pavement skid resistance is usually measured directly through the use of locked wheel skid trailers. The trailer is towed over the pavement surface at a speed of 40 mph or higher and water is applied in front of the test wheel (Figure 4). The test wheel is

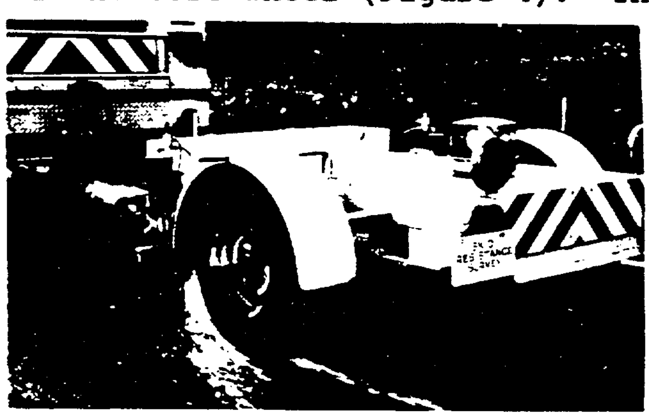


Figure 4 - Locked Wheel Skid Tester

locked by a brake, and after it has been sliding along the pavement for a certain distance the force that the friction in the tire contact patch produces or the resulting torque on the test wheel is measured and recorded for a specified length of time. Either a ribbed tire or a smooth tire can be used to perform the test. The ribbed tire is insensitive to the pavement macrotexture, allowing water dissipation through the tire grooves. The smooth tire is sensitive to the macrotexture.

Standard procedures (ASTM E 274-85) have been developed for the performance of skid testing with the locked wheel skid trailer. The result of the test is reported as a skid number. On-board computers are now being used to record and calculate the skid number, as well as to plot skid number versus speed, and peak incipient friction, if desired.

Another device available to measure skid resistance is the mu-meter (Figure 5).

Figure 5 - Mu-Meter

The mu-meter, like the locked wheel tester is trailer mounted. It uses smooth tires, yawed at equal but opposite angles to measure side friction force. Operation procedures are similar to the locked wheel trailer. On highway pavements the mu-meter may not provide an accurate indication of the pavement skid resistance due to the location of the narrowly spaced trailer wheels. The wheel paths of the mu-meter wheels generally fall between the normal wheelpaths of highway traffic. The use of the mu-meter has declined on highway pavements during the 1970's and 1980's.

New methods to improve testing efficiency and reduce skid testing costs are being studied. Recently completed research indicates that the spin-up tester may produce accurate results at lower costs. Like the locked wheel tester and mu-meter, this device is also trailer mounted. Testing begins following the locking of the wheels, and continues after the release of the brake until the wheels reach full angular velocity. The time interval between the moment the brake is released and the achievement of full angular velocity is indicative of the skid resistance of the pavement.

Automated equipment is now available and being refined to measure or correlate skid data indirectly with lasers and video technology. Two indirect methods to collect data for skid correlation are under development. Devises using laser sensors are capable of measuring the macrotexture of the pavement surface, which has some influence on skid resistance.

Video technology may also hold promise for the future, but has not yet been fully investigated in the United States for the purpose of correlation to skid resistance. A device known as the Yandell Mee Texture Friction Device is now in operation in Australia. The device uses a video camera, tracking device, and image enhancement to capture an enlarged video picture of the pavement surface. An on board computer collects the data. Software performs a statistical analysis of the texture, and produces output data on the locked wheel braking force friction and the sideways coefficient of friction.

Equipment for Deflection Data Collection

Measurement of pavement structural carrying capacity provides valuable data for the selection and design of a pavement rehabilitation strategy. Until recently, deflection was only collected and used at the project design level, but now several SHAs collect it at the network level, and the trend in this direction will probably continue.

Stronger pavements deflect less than weaker ones, and support far more traffic loadings. Deflection measurements are taken through the measurement of a deflection basin which is created by application of a load to the pavement. This load may be applied in several forms, such as by parking a loaded heavy vehicle of known axle loading, or by dropping masses onto the pavement surface. The load applied to the pavement surface creates a pavement deflection basin (Figure 6). The size, shape and depth of the deflection basin represents an overall system response of the paving layers and the subgrade to the known load. When the load is applied, all layers deflect creating strains and stresses in the supporting layers.

Differences in the size, shape, and depth of the deflection basin can be measured both at the surface and the underlying layers. Deflection will most often be measured at the pavement surface for network or project level analysis. On some research studies measurements may be taken at various depths in the pavement section, such as at the asphalt-aggregate interface, and the bottom of the subgrade.

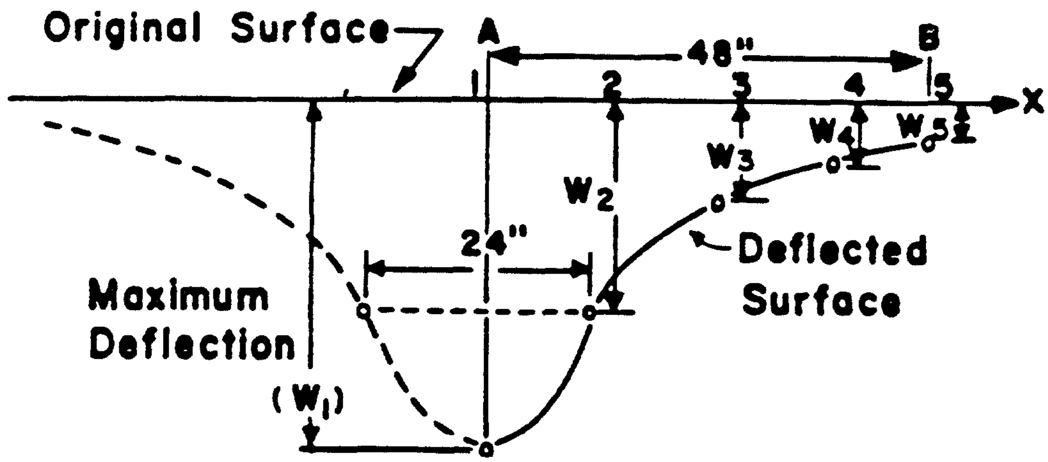


Figure 6 - Typical pavement deflection basin

Many factors influence the measured pavement deflection and interpretation of the results requires a thorough analysis. As the load is increased the pavement deflection will also increase, but not linearly, since most pavement material properties are stress dependent. Factors influencing the measured deflection include the pavement type, stiffness of the pavement/subgrade system, location of the test, proximity to joints or cracks, location of drainage structures, variations in soil composition and moisture, and voids beneath the pavement structure. Climatic factors such as temperature, thermal gradients, moisture, and depth of frost greatly affect the results, as does the season of the year in which the tests are taken. All deflection data should be adjusted to a constant temperature and season prior to plotting or use. Proper procedures must be followed for temperature and seasonal corrections to obtain reliable results.

Four classes of equipment exist to measure deflection: static deflection equipment, automated beam deflection equipment, steady-state dynamic deflection equipment, and impulse deflection equipment.

Static deflection equipment

Static deflection equipment is used to measure the deflection of the pavement to slowly applied loads. The most commonly used static deflection device is the Benkelman Beam, a 12-foot beam pivoted at the third point. The pivot provides an 8-foot probe with the extreme tip resting on the pavement and supported by the pivot point. The rear end is a 4-foot cantilever beam which moves upward when the pavement deflects downward. The basic components are depicted in Figure 7.

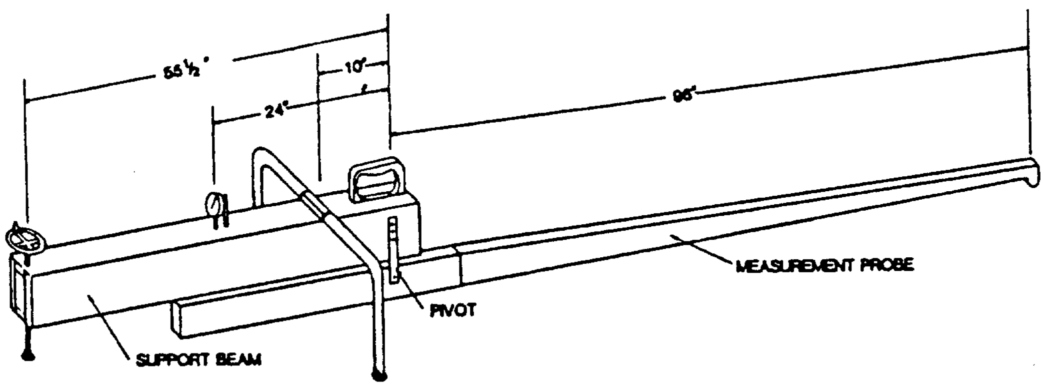


Figure 7 - Basic Components of the Benkelman Beam

The device uses a loaded truck to create the deflection. A dial indicator rests on the rear end and measures this movement. The only measurement recorded with the Benkelman Beam is the maximum rebound deflection. The limitations of the device are insuring that the front supports are not located within the deflection basin, the inability to determine the shape and size of the basin, and poor repeatability of the results. Due to these limitations the SHAs are moving away from the static devices and toward other types such as the impulse deflection equipment.

Other devices in this category are the Curvature Meter and the Plate Bearing Test equipment.

Automated Beam Deflection Equipment

Equipment which automates the Benkelman Beam process is placed in this class. Included is the La Croix Deflectograph which has been used widely in Europe and other parts of the world, and the Travelling Deflectometer which is used by the California Department of Transportation.

Steady-State Dynamic Deflection Equipment

Steady-state dynamic deflection devices place a static preload on the pavement surface. A steady-state sinusoidal vibration is then induced in the pavement with a dynamic force generator. The advantage of this type of equipment over static equipment is that a reference point is not needed. An inertial reference is used and the change in deflection can be compared directly to the magnitude of the dynamic force. One of the limitations of this type of equipment is the use of the static preload. This load is relatively large in comparison with the maximum peak to peak loading. The most commonly used steady-state dynamic deflection devices are the Dynaflect and the Road Rater.

Dynaflect - The Dynaflect (Figure 8) was one of the first commercially available steady-state dynamic deflection devices. It is trailer mounted, and can be towed by a standard automobile.

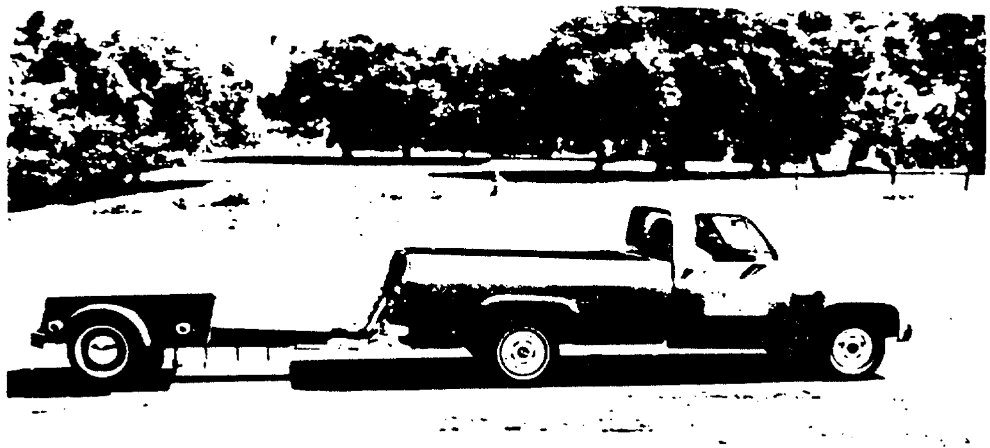


Figure 8 - Dynaflect

The Dynaflect applies a static weight of 2000 to 2100 pounds to the pavement while the dynamic force generator produces a 1000-pound peak-to-peak force. Deflection is measured using five velocity transducers (geophones). The transducers are suspended from a placing bar which is normally placed in the center of the loaded area and at one-foot intervals away from the load (Figure 9).

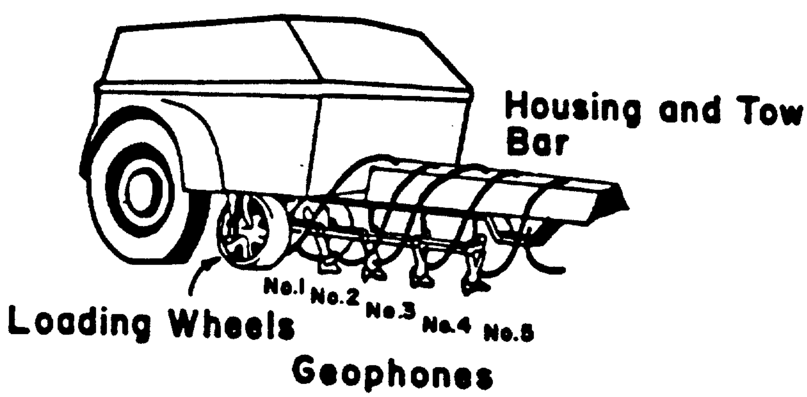


Figure 9 - Schematic of the Dynaflect in testing position

The device is moved to the test point and the loading wheels and geophones are hydraulically lowered to the pavement surface. The device is then moved to the next site on the loading wheels. The limitations of the Dynaflect are the maximum peak-to-peak loading, which is limited to 1000 pounds, the inability to vary the load, and the fixed frequency of the loading, which cannot be changed.

Due to this limitation this device may be inadequate to evaluate thick pavement sections and other devices with heavier loading systems should be considered.

Road Rater - The Road Rater (Figure 10) is the second type of

Figure 10 - Road Rater

commercially available steady-state dynamic deflection device. Three models, which vary in the magnitude of the applied load are available. The static load is applied to the pavement surface through a steel plate. The dynamic force generator produces a peak-to-peak force which is one-half the magnitude of the static preload. The amplitude and the frequency can be altered. This allows different dynamic peak-to-peak loadings ranging from 1000 to 8000 pounds. The loading frequency can be varied between 5 and 70 cycles per second. Three sensors are attached to an arm trailing the loading plate, with an additional sensor in the center of the loaded area.

Testing starts by moving the device to the test point, and lowering the test plate and the sensors to the pavement surface. A test run is performed at selected loads and frequencies, the loading plate and sensors are lifted from the surface, and the device is moved to the next site. The limitations of this device is the limited load level for the lighter models, and the requirement for a heavy static preload for the heavier models.

Other Steady State Deflection Equipment - Other devices in this category include the FHWA Thumper, a custom-built device that can perform static, dynamic, or intermittent pulse loading, and other custom-designed dynamic deflection devices.

Impulse Deflection Equipment

These devices deliver a transient force impulse to the pavement surface. They use a weight which is lifted to a specified height on a guide system, and dropped. The falling weight strikes a plate, which transmits the force to the pavement. By varying the amount of weight and the drop height, the impulse force can be varied.

The advantage of the impulse type equipment is the ability to model a moving wheel load in both magnitude and duration. The resulting deflection closely simulates deflections caused by a moving wheel load. The impulse equipment has a relatively small preload compared to the actual loadings. The preload prior to releasing the weights varies with the equipment. It is usually in the range of 8 to 18 percent of the maximum impulse load which is 9,000 to 24,000 pounds. The preload during the period the weights are dropping is normally in the range of 5 to 14 percent of those same maximum impulse loads.

Dynatest Falling Weight Deflectometer - The most widely used falling weight deflectometer (FWD) in the U.S. is the Dynatest model 8000 FWD (Figure 11). The system is trailer mounted and can

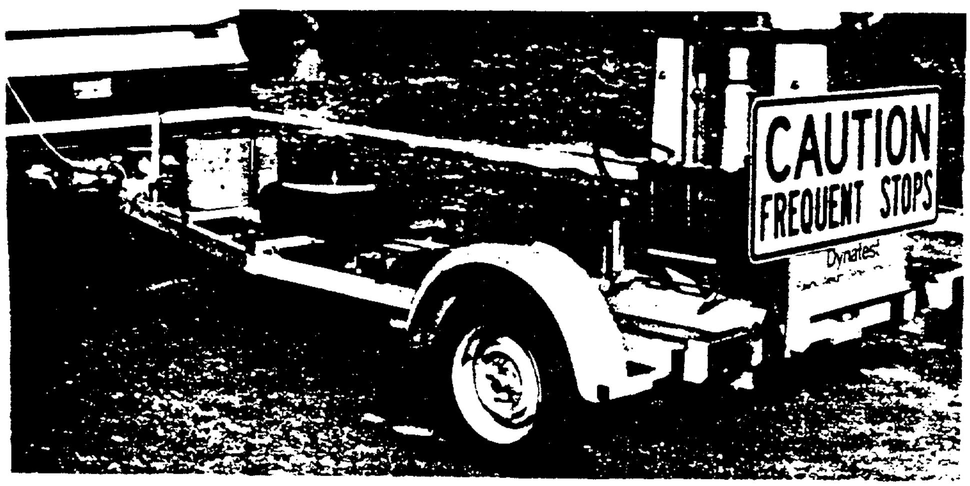


Figure 11 - Dynatest Falling Weight Deflectometer

be towed by a van or pickup truck. Its impulse force is created by dropping weights from different heights. By varying the drop height and weights, a force range of 1,500 to 24,000 pounds can be developed. The load is transmitted through a rubber buffer system and an 11.8 inch diameter loading plate to the pavement. The deflection basin is measured using up to seven velocity transducers which are mounted on a bar and lowered with the loading plate to the pavement surface. The device is moved to the test site, and the loading plate and transducers are lowered to the pavement. A test sequence is completed using a number of drops at each selected drop height. The loading plate and sensors are then hydraulically lifted, and the FWD is moved to the next site.

Other Impulse Deflection Equipment - Other devices in this category include the KUAB and the Phoenix FWDs. Both are trailer mounted and are operated in a similar manner to the Dynatest FWD, using various combinations of number and size of weights and drop heights.

A summary of the most commonly used deflection devices and their various measurement properties and features is contained in Table 1. The most important limitation of current deflection equipment is the inability to collect the data at high speeds. Stops must be made at each test location, requiring the maintenance of traffic. Developmental efforts are underway to automate deflection equipment to reduce traffic conflicts. Again, it should be emphasized that pavement deflection data should be used carefully. Appropriate correction factors for temperature, moisture, time of season, test location, etc. must be applied in order to produce meaningful data. Proper use of this very important data can yield effective information for the design of pavement rehabilitation strategies.

TABLE 1 - DEFLECTION EQUIPMENT

DEFLECTION DEVICE	PRINCIPAL OF OPERATION	LOAD ACTUATOR SYSTEM	STATIC WEIGHT ON PLATE	TYPE OF LOAD TRANSMISSION	RELATIVE COST	DEFLECTION MEASURING SYSTEM	NUMBER OF SENSORS
Benkelman Beam	Deflection Beam	Loaded Truck Axle	N/A	Truck Wheels	Ext. Low	Dial Indicator	1
Dynaflect	Steady State Vibratory	Counter Rotating Masses	2100	Two UrethaneCoated Steel Wheels	Low	Velocity Transducers	5
Road Rater	Steady State Vibratory	Hydraulic Actuated Masses	*2,400 to 5,800 lbs.	*Two rectangular or 1 round plate	Low-Medium	Velocity Transducers	4
KUAB FWD	Impulse	Two Dropping Masses	N/A	Round Plate	Medium	Seismic Deflection Transducers	5
Dynatest 8000 FWD	Impulse	Dropping Masses	N/A	Round Plate	Medium-High	Velocity Transducers	7
La Croix Deflectograph	Mechanized Deflection Beam	Moving Weighted Truck	N/A	Truck Wheels	Ext. High	Inductive Displacement Transducers	2

Equipment for Roughness Data Collection

Over a period of time all pavement surfaces become increasingly rough. Ride quality, a subjective evaluation of pavement roughness, may be evaluated through the use of rating panels (which may be part of the condition survey team), or through the use of manually operated or automated equipment. Other equipment may be used to objectively measure roughness. In terms of pavement profile, roughness can be defined as the summation of variations in the surface profile of the pavement. This pavement roughness consists of surface irregularities with wavelengths and amplitudes ranging from fractions of an inch to several feet. The measurement of pavement roughness corresponds to the measurement of the actual pavement profile or the measurement of the response of a mechanical system to the profile.

Knowledge of the extent of pavement roughness is essential since roughness often provides some indication of a pavement's need for maintenance, rehabilitation, or reconstruction. Roughness is also one of the primary criteria by which the public measures the credibility of an agency that manages pavements. FHWA considers roughness data collection to be of vital importance in the assessment of pavement condition. The States will be required to have automated calibrated equipment operational for the collection of Highway Performance Monitoring System data by the end of 1989.

Equipment for roughness survey data collection may be broadly categorized into 4 categories, the relative degree of automation and complexity of which increases in the order listed:

- Rod and level survey-including the Dipstick Profiler,
- Profilographs,
- Response type road roughness meters (RTRRMs),
- Profiling Devices

Each category has its advantages and limitations, and selection of appropriate roughness equipment should be made following a careful assessment of the primary purpose for which the equipment is to be used, and an analysis of the advantages and limitations of each device. One of the most important considerations in selection of a roughness measuring device is the tradeoff between the relatively low initial and data collection costs of devices such as an RTRRM, versus the frequent need to calibrate the device. Other tradeoffs must also be carefully considered.

Laser devices may also be used to measure pavement roughness. Since the laser can also be used to measure other pavement parameters, its use will be discussed in the section "Equipment for Distress Data Collection".

Rod and Level Survey and the Dipstick Profiler

Rod and level surveys provide an accurate measurement of the pavement profile. The use of the rod and level survey for network or even large project survey data however, is impractical and cost prohibitive. A first-step automation of the rod and level survey which may be used to collect a relatively small quantity of pavement profile measurements is through the use of the Dipstick Profiler (Figure 12).

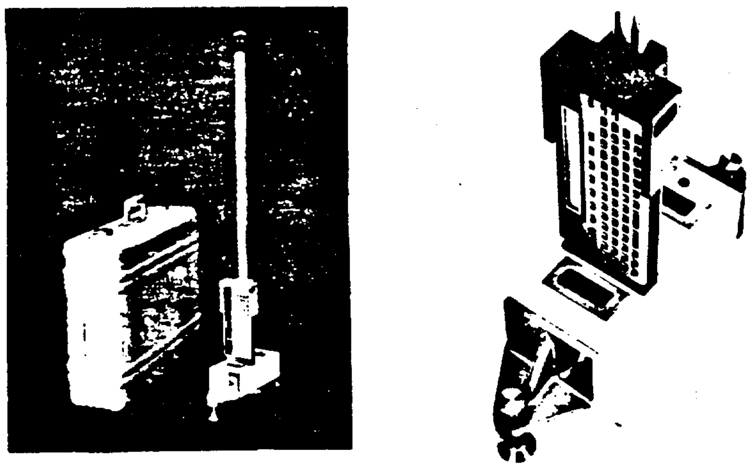


Figure 12 - The Dipstick Profiler

The Dipstick Profiler consists of an inclinometer enclosed in a case supported by two legs separated by 12 inches. Two digital displays are provided, one at each end of the instrument. Each display reads the elevation of the leg at its end relative to the elevation of the other leg. The operator then "walks" the dipstick down a premarked pavement section by alternately pivoting the instrument about each leg. Readings are recorded sequentially as the operator traverses the section. The device records 10 to 15 readings per minute. Software analysis provides a profile accurate to plus or minus 0.005 inch.

A common application for the dipstick is to measure the profile for the calibration of RTRRMs. A strip can be surveyed by a single operator in about one half the time of a traditional survey crew.

Profilographs

Profilographs have been available for many years and exist in a variety of different forms, configurations, and brands. Due to the design they are not suitable for condition surveys. Their most common use today is for portland cement concrete pavement construction inspection, control, and acceptance. The major differences among the various profilographs involve the configuration of the wheels and the operation and measurement procedures of the various devices.

Profilographs have a sensing wheel, mounted to provide for free vertical movement at the center of the frame (Figure 13). The

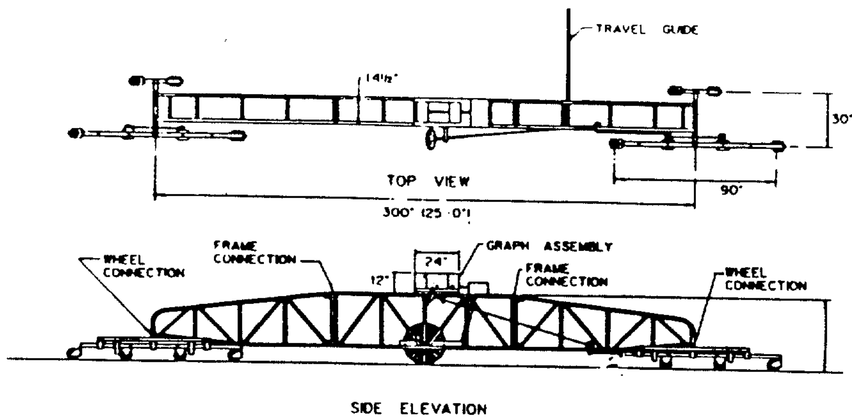


Figure 13 - Profilograph Layout (California type Profilograph)

deviations against a reference plane, established from the profilograph frame, is recorded (automatically on some models) on graph paper from the motion of the sensing wheel. Profilographs can detect very slight surface deviations or undulations up to about 20 feet in length.

Response Type Road Roughness Meters

The third category of roughness data collection equipment is the response type road roughness meters. This category includes such devices as the BPR Roughometer (Figure 14), the Mays Ride Meter (Figure 15), and the PCA Road Meter. RTRRMs have been used for a

Figure 14 - BPR Roughometer

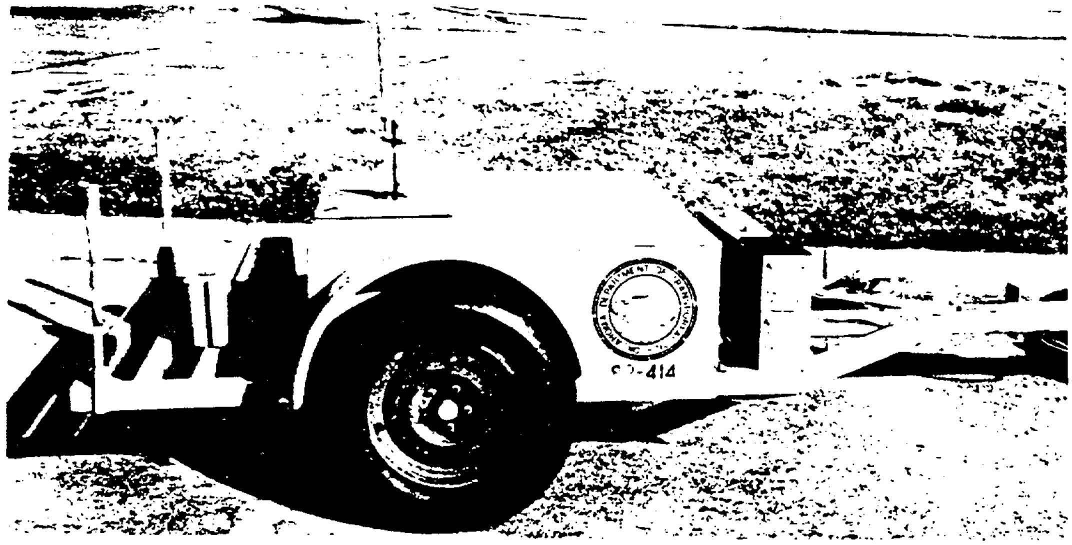


Figure 15 - Mays Ride Meter

number of years and are currently the most widely used roughness data collection device in the United States. Their primary use is for network level roughness data collection.

Road meters or RTRRMs measure the vertical movements of the rear axle of an automobile or the axle of a trailer relative to the vehicle frame. The meters are installed in vehicles with a displacement transducer on the body located between the middle of the axle and the body of a passenger car or trailer. The transducer detects small increments of axle movement relative to the vehicle body. The output data consists of a strip chart plot of the actual axle body movement versus the time of travel.

The <u>advantages</u> associated with the use of RTRRMs are:

- The initial and operating costs are low.
- Reasonably accurate roughness data is provided <u>if the device is properly calibrated.</u>
- Reproducible results may be obtained when the device is properly maintained.
- Data can be collected at high speeds, i.e. 50 mph.
- Efficiency--numerous pavement sections can be evaluated in a relatively short period of time.

In spite of the advantages of RTRRMs, there also several limitations:

- Response type equipment records the dynamic response of a mechanical system travelling over a pavement at a constant speed. Therefore, the characteristics of the mechanical system and speed of travel affect the measurement.

- RTRRMs measure a dynamic effect of the roughness but do not define the profile of the pavement.
- In order to provide accurate, consistent and repeatable data, the device must be frequently calibrated, through a range of operating speeds, against sections of known profile, ranging from very smooth to very rough. The annual costs of the calibration checks can be quite high.
- The vehicles in which the RTRRMs are installed contribute many potential sources of variation including rear suspension damping, tire nonuniformities and inflation pressure differences, and vehicle weight changes.
- Due to the variations of the various mechanical systems of RTRRMs, comparability of data among users with the same user, or with the same device is difficult, unless a common standard roughness index is used.

Several standard calibration procedures have been developed for the RTRRMs which are in use today. Careful operating and maintenance procedures should be followed, including frequent and precise calibrations, in order to improve device accuracy and consistency. The degree of accuracy desired in the calibration of RTRRMs ultimately depends upon the proposed use of the data being collected.

RTRRM systems are adequate for routine monitoring of a pavement network and providing an overall picture of the condition of the network. The output can provide managers with a general indication of the overall network condition and maintenance needs.

Profiling Devices

Profiling Devices are used to provide accurate, scaled, and complete reproductions of the pavement profile within a certain range. They are available in several forms, and can be used for calibration of the RTRRMs. The equipment is expensive, with complexity increasing depending on the types and number of transducer sensors contained on board. Three generic types of profiling systems are in use today:

- Straight edge
- Low speed systems
- Inertial Reference Systems

The simplest profiling system is the straight edge. Modifications to the straight edge, such as mounting it on a wheel are very popular (profilographs). Low speed systems such as the CHLOE profilometer (Figure 16) are moving reference planes that have little or no dynamic effect due to their low speed. The CHLOE is a long trailer that is towed at low speeds of 2 to 5 mph. The slow speed is necessary to prevent any dynamic response measurement during the readings. The device measures the

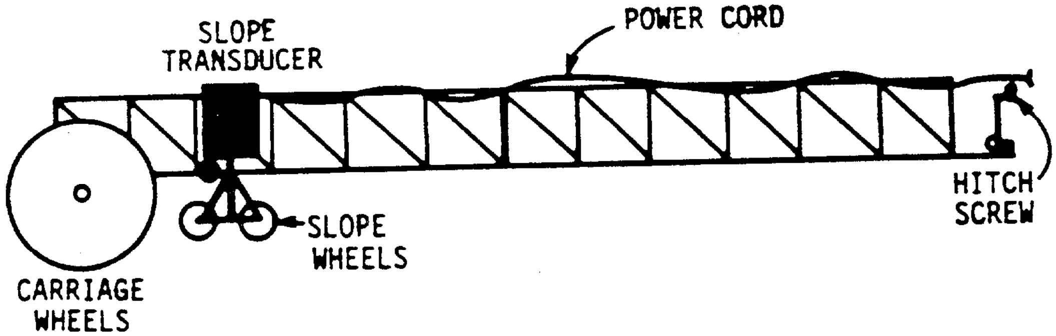


Figure 16 - CHLOE Profilometer

difference in slope between a small arm with two wheels and a trailer frame with 2 larger wheels. A few agencies still use the CHLOE to calibrate their RTRRMs.

Most sophisticated road profiling equipment uses the inertial reference system. The profiling device measures and computes longitudinal profile through the creation of an inertial reference by using accelerometers placed on the body of the measuring vehicle to measure the vehicle body motion. The relative displacement between the accelerometer and the pavement profile is measured with either a "contact" or a "non-contact" sensor system.

The earliest profiling devices used the contact system to measure the profile. The contact system uses a mechanism in direct contact with the pavement. Several contact systems have been used, and are still in use today. The French Road Research Laboratory developed the Longitudinal Profile Analyzer (APL) in 1968 (Figure 17).

Figure 17 - Longitudinal Profile Analyzer

The APL consists of a specially designed single-wheel trailer and electronic data and measurement monitoring equipment. The trailer is pulled at a constant speed by a towing vehicle which contains the electronic equipment. The APL measures pavement profile based on the amplitude of the vertical movements of the follower wheel. These movements are measured in relation to a horizontal reference pendulum (Figure 18). As the follower wheel travels over the

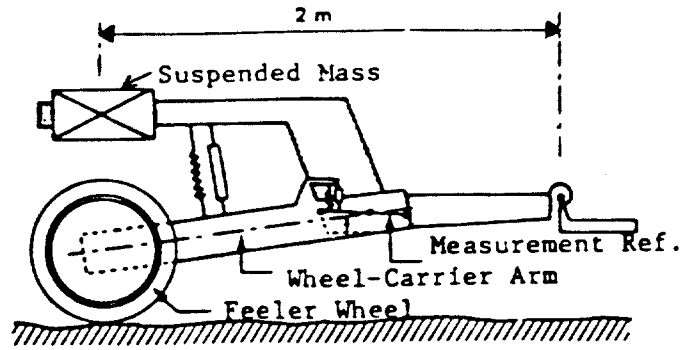


Figure 18 - Longitudinal Profile Analyzer Measurement System

pavement surface the change in the angle between the trailing arm or rocking shaft and the horizontal pendulum is processed by the system software into a profile value.

Systems used today in the United States are frequently installed in vans which contain on board microcomputers and other data handling and processing instrumentation. Older profiling devices are usually contact systems, while the more recently manufactured devices use non-contact sensors. A contact system is depicted in Figure 19. This system uses a small tracking wheel which

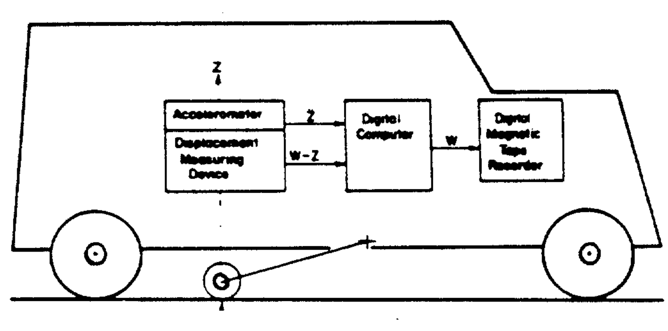


Figure 19 - Contact Profiling Device

measures the surface of the pavement. The mechanical systems experience maintenance problems related to wearing and operation of the wheel. Recently constructed systems use non-contact probes, either acoustic or light, to measure differences in the pavement surface.

Both measure and compute the longitudinal profile through the creation of an inertial reference plane. An accelerometer, placed on the body of the measuring vehicle, measures the vehicle body motion. The relative displacement between the accelerometers and the pavement profile is measured with the non-contact light beam mounted with the accelerometer on the vehicle body. The sensor beam is projected vertically down on the pavement to create a light "foot print".

Displacement between the vehicle and the pavement surface is determined by measuring the angle at which the light beam footprint is viewed by part of the system mounted under the vehicle and just ahead of the light beam footprint (Figure 20).

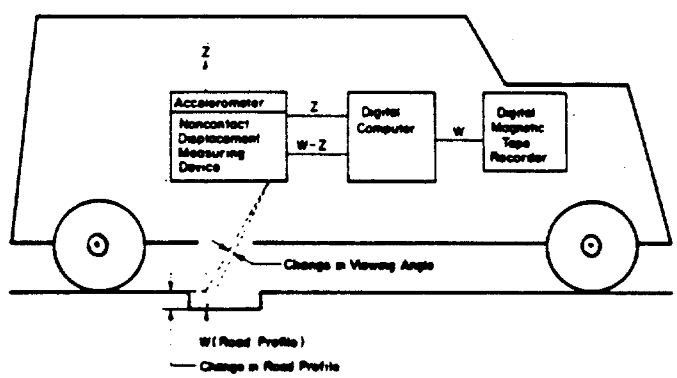


Figure 20 - Principle of Operation of a Non-contact Profiling Device

As the pavement surface profile varies, the distance between the vehicle and the pavement surface changes, and the angle at which the footprint is viewed also changes. Through the geometry of the system, this measured angle can be converted to a measured displacement and used in computation of the profile. Distance measurements of the position of the vehicle are simultaneously recorded in fractions of a foot as the vehicle travels along the roadway. The profile is computed as a function of distance travelled by an on board digital computer and is saved and stored for later use.

Profiling devices are capable of providing highly accurate roughness data and scaled reproductions of the pavement profile. A summary of the most commonly used roughness data collection devices, their measurement principles, relative costs, relative degrees of accuracy, and current and projected future use is contained in Table 2.

TABLE 2 - ROUGHNESS DATA COLLECTION EQUIPMENT

ROUGHNESS DATA COLLECTION DEVICE	PRINCIPAL OF MEASUREMENT	RELATIVE INITIAL COST	RELATIVE DATA COLLECTION COST (NETWORK)	RELATIVE DEGREE OF ACCURACY	APPROXIMATE NUMBER OF YEARS IN USE	EXTENT OF CURRENT USE	PROJECTED EXTENT OF USE YR. 1990-2000
Dipstick	Direct differential Measurement	Low	Impractical	Very High	8	Limited, used for Calibration	Same as Current Use
Profilographs	Direct Profile Recordation	Low	Impractical	Medium	30	Extensive for Const. Accptance	Same as Current Use
BPR Roughometer	Device Response	Low	Low	Medium	50	Limited	None
Mays Meter	Vehicle Response	Low	Low	Medium	30	Extensive	Decreasing Continuously
South Dakota Road Profiler	Direct Profile Recordation	Medium	Low	High	7	Growing	Rapidly Increasing
Contact Profiling Device	Direct Profile Recordation	High	Medium	Very High	20	Limited	Decreasing
Non-Contact Profiling Device	Direct Profile Recordation	Very High	Medium	Very High	8	Medium	Increasing Continuously

PAVEMENT DISTRESS

By far the greatest number of innovations and inventions in the past 10 years have come in the area of distress data collection equipment. Devices may soon be available which automatically collect and reproduce pavement distress information in a form acceptable for use during pavement rehabilitation -- but we're not to this point yet.

Before examining equipment available to collect pavement distress data, we must first stop and try to answer the question "What is pavement distress?" As elementary as this question may seem, it has been surprisingly difficult to answer in the past. First, the pavement type must be known -- is it jointed plain concrete, jointed reinforced concrete, continuously reinforced concrete, prestressed concrete, asphalt, an asphalt overlay of the various types of portland cement concrete, one of the various types of portland cement concrete over asphalt, portland cement concrete over portland cement concrete, bonded, unbonded, asphalt over asphalt, etc. What type of base exists, granular, stabilized, free-draining, etc? What measurement criteria was used? What type of distress was noted, what was its extent, and what was its severity? Each question may generate several others.

Many types of distress have been identified for stabilized pavement surfaces. Methods have been devised by various agencies to standardize distress classifications and to automate the recording, reduction, processing, and storage of the data. Condition survey manuals which define classifications using photos and descriptions have been used to minimize discrepancies between raters. One document which best answers many of the questions about highway pavement distress is the "Highway Pavement Distress Identification Manual for Highway Condition and Quality of Construction Survey". This manual describes 17 or more pavement distress types for each pavement type. Some procedures use detailed measurements of the distress to minimize errors.

To simplify this discussion, distress types will be limited to two very general categories for asphalt pavements -- rutting and cracking, and one category for PCC pavements -- cracking. Extent and severity may be dealt with in relative terms. For the extent, the percentage of the pavement distressed versus the total pavement length or area surveyed can be used. Severity can also be handled in relative terms -- light, medium, and heavy. Numerical parameters can be assigned at a SHAs discretion in order to classify the distress by extent and severity.

Equipment for Distress Data Collection

The thankless job of collecting pavement distress data by visual survey is being simplified through the use of current technology.

Condition survey teams now have at their disposal numerous means and devices to collect distress data. This list of available devices is growing each year, and more fully automated processes are being designed and implemented. Automation of pavement distress data collection may be categorized into five levels of automation:

- Improved subjective rating through data entry with condition rating keyboards or voice activated systems,
- Acoustic systems,
- Laser technology
- Partial objective automation through evaluation of photographic records, and
- Fully automated processing through digitized reduction (or other fully automated methods) of video or other collected images.

<u>Condition Rating Keyboards</u>

The first degree of automation is by data entry with a hand held computer or data logger. The operator uses a keyboard similar to an office personal computer keyboard, or another design, to input observed distress types, extents and severities (Figure 21). The

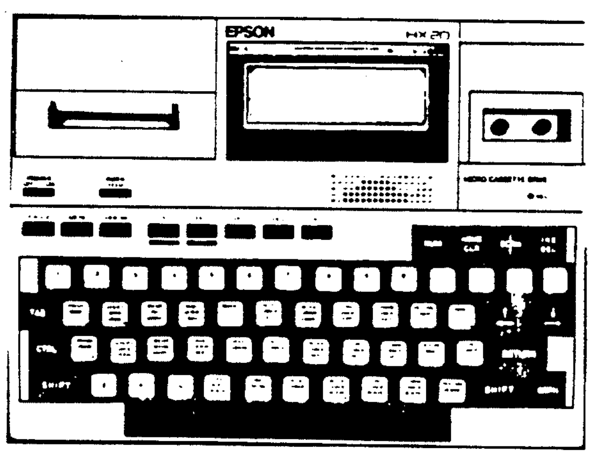


Figure 21 - Condition Rating Keyboard

keyboard may be pre-programmed prior to beginning the condition survey, based on a preliminary assessment of the existing distress, or through knowledge of the most commonly encountered distress types. A distance measuring instrument continuously tracks the location of the vehicle, and interfaces with the data input to define the location of the recorded distress. Information collected is stored on a hand held or personal computer installed in the vehicle, then down loaded to an office computer.

Voice Activated Systems

One new technology now being employed on a more limited basis is the use of a voice activated system. This system provides a significant advantage over the keyboard since it frees the hands of the operator. While at least two raters are required for the condition keyboard system, the voice activated system requires just one rater. The rater speaks into a microphone attached to a headset, and using a voice controlled microprocessor, the information is coded with the location and stored on the computer in the vehicle. A sound track recording is produced which may be used with recorded video information, if used. Depending on the software, the system may recognize several hundred standard condition observations stored in the system.

Acoustic Devices

Sonic and ultrasonic probes are now being used to a great extent, to measure profile and rutting. The probes are used as displacement transducers, measuring the distance to the pavement surface from an established inertial reference plane of the vehicle. Any number of transducers may be mounted on the vehicle or on a bar attached to the vehicle to measure rutting (Figure 22). The probes generate a short burst of sound waves

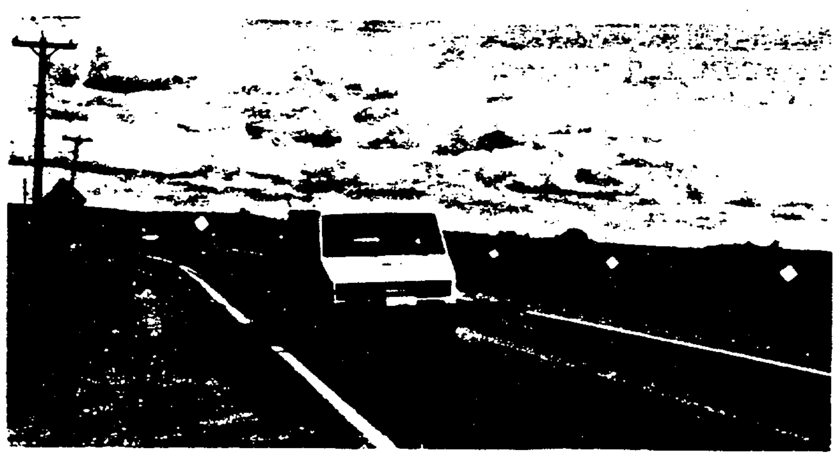


Figure 22 - Survey Vehicle with a Rut Bar

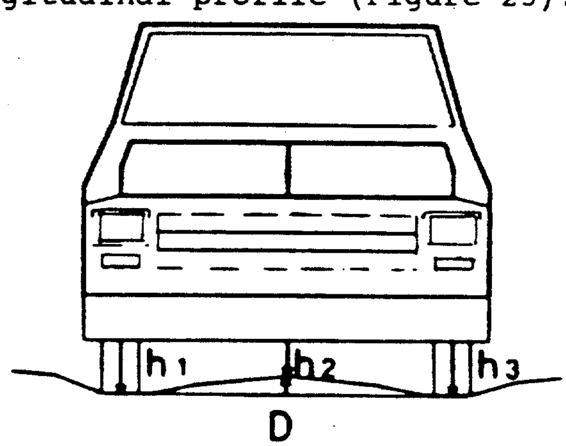
which travel to the pavement surface and are reflected back to the transducer. The elapsed time between the sound generation and the echo detection is proportional to the distance travelled. The number of transducers selected should correspond to the level of detail required. For network level surveys, 3 to 5 points usually provide an adequate transverse profile. Vehicles have been manufactured with up to 37 transducers to provide an accurate cross section of a 12 foot width of pavement.

The sensors should be used only when the pavement surface is dry, as the sensors themselves, and the readings may be adversely affected by moisture. Sound travels comparatively slowly, (about 1125 feet per second through air) therefore the speed of the survey vehicle has some minor effect on the measurement. Since the speed of sound is also dependent upon the density and temperature of the medium through which it travels, air temperature also affects readings. These effects, however, are minimal for network level surveys.

The South Dakota DOT (SD DOT) has developed a system, known as the South Dakota Road Profiler, which uses an ultrasonic transducer, accelerometer, and an on board computer system to measure, process and store pavement profile data. A number of other SHAs are also presently considering manufacture of a system similar to the one developed by the SD DOT. This system uses an ultrasonic probe as a displacement transducer. The probe is mounted on the front of the vehicle and measures the distance from the vehicle to the pavement surface from an established inertial reference plane. An accelerometer, mounted near the acoustic probe, establishes the inertial reference plane by measuring the vertical acceleration of the vehicle body. An on board computer processes collected data, storing it on floppy disks. The Road Profiler can also plot the measured profile.

The advantage of this system is its low initial and operating cost. The limitations are that the data reported are slightly less accurate than with the light-based devices, and the data processing system presently used is not compatible with systems used by many other SHAs.

An added advantage to the system is its ability to measure rut depth. By mounting two additional transducers to the front bumper, rut depth (3 points) can be recorded and used to supplement the longitudinal profile (Figure 23). Additional



Rut Depth, D = $(h_1-2h_2+h_3)$

Figure 23 - Rut depth Measurement

transducers could be added if desired to define a full transverse profile.

Several of the most recent devices to appear on the market have used additional transducers to more clearly define the transverse profile. Some use lasers in lieu of the ultrasonic transducers to increase measurement accuracy and frequency, or to measure other pavement parameters such as macrotexture or cracking. A few have supplemented the existing capabilities with additional devices such as gyroscopes in order to provide measurements of most of the pavement features and geometry including roughness, rut depth, grade, cross slope, and radius of curvature.

Lasers

Light Amplification through Stimulated Emission of Radiation (LASER) has fascinated man since Albert Einstein's atomic theories in 1917. Lasers have many applications today; they are used extensively in science, medicine, industry, commerce, communications, the military, and in the laboratory. Their useful purposes are ever growing -- they are used to drill, weld, cut, scribe, unblock arteries, repair eyes, detect drug concentrations in body fluids, transmit and print information, improve the precision of gyroscopes and radar, and track satellites. They also measure very accurately. Their tremendous potential for condition survey measurement is just beginning to be tapped.

Lasers are now manufactured in many types, including chemical lasers, gas lasers, and semiconductor diode lasers. Non-contact laser transducers measure on the principle of triangulation, most often using gallium arsenide diode lasers (Figure 24). The laser

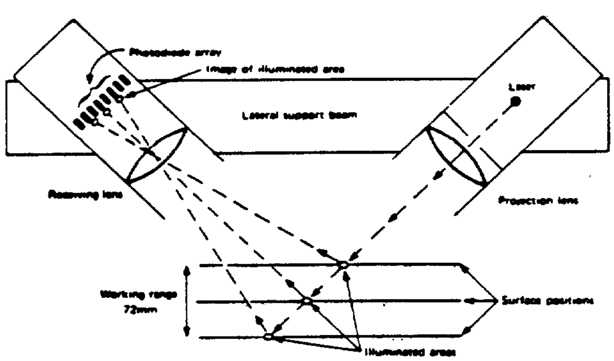


Figure 24 - Laser Measurement Principle

transducer is usually mounted directly to the survey vehicle (Figure 25).

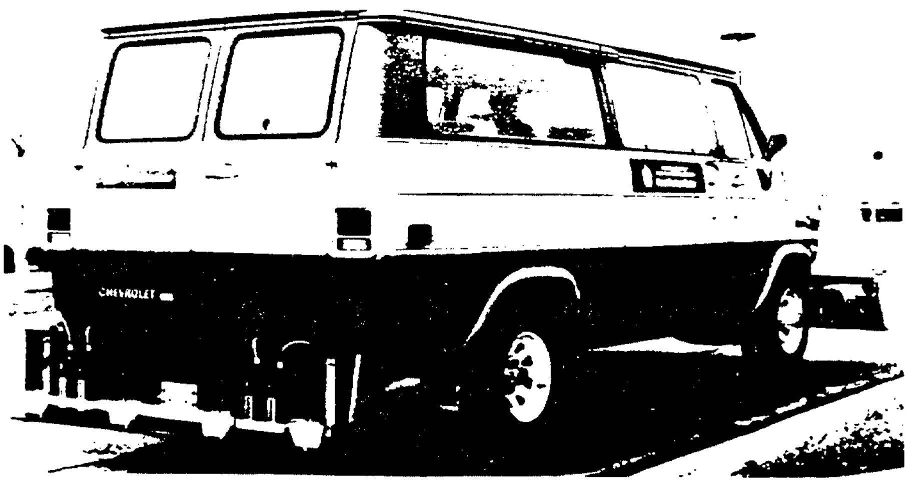


Figure 25 - Lasers installed on a Survey Vehicle

Lasers can operate either during the day or at night at a temperature range from near freezing to about 100 degrees F. During use, the pavement surface should be dry. Lasers possess special properties which allows highly accurate measurement:

- Intensity - the energy is extremely concentrated, as opposed to acoustic transducers, where sound waves are readily diffused
- Monochromatic and Coherent - single-colored light is emitted in a very narrow wave band which is so organized that it stays in phase over long distances
- Collimated - the light is highly directional and travels in a narrow cone
- The energy is generated and reflected back in extremely short pulses which allows a large number of readings to be taken over very short distances even at high speeds

These special properties permit detailed, frequent measurement of the pavement surface characteristics. Lasers are currently used for longitudinal profile, rut depth and macrotexture measurement. A method to measure cracking is under development. Roughness can be measured using the principles described previously. The lasers pulse at either 16,000 hertz (cycles per second) or 32,000 hertz. Even at 60 mph a reading could theoretically be taken each 0.00275 inch, permitting a detailed measurement of the pavement surface texture. In years to come the laser may replace the skid trailers in the search for pavement surfaces with low coefficients of friction. Experimental efforts using lasers for this purpose are underway.

Systems in use today employ the lasers in a variety of numbers, configurations, and locations on the survey vehicle. Similar to the acoustic transducers, 3 to 5 lasers mounted across the vehicle can provide a good transverse profile. Up to 11 lasers have been used for this purpose. Accurate measurements of slab faulting and transverse cracking have been recorded; however, longitudinal cracking is not detectable unless it lies within the small laser footprint. Rapid raster scanning, or closely bunching lasers, could provide a solution to this problem. At least one agency is studying the possibility of detecting all types of cracking by grouping several lasers very closely together, and correlating the measured cracking noted in a small accurately measured area to the entire pavement surface area. Some manufacturers and agencies are also experimenting with lasers for use in continuous deflection devices that will operate at high speeds.

The PRORUT system developed by the University of Michigan for the FHWA provides pavement profile and an average rut depth. The PRORUT is an inertial profilometer which was designed to minimize the overall costs of the system. The vehicle contains three road sensors, one in each wheel track and one centered between the wheel tracks. Two accelerometers are installed in the profilometer, one above the road sensor in each wheel track. The vehicle speed and distance of travel is measured by a pulser installed in the right front wheel. An IBM-PC serves to control the system calibration, operation, data acquisition, and data viewing. Four SHAs have recently evaluated the performance of the PRORUT, and an evaluation report of the findings is being prepared. The device is shown in Figure 26.

Figure 26 - PRORUT

Photographic Evaluation

In the early 1970's two systems were developed that made use of continuous pavement surface photography. These systems are currently in use in Europe, Japan, and recently, in the United States. Cracking and other surface distresses are recorded with a high resolution, continuous pavement surface photographic recorder. The recorder uses a 35-mm slit camera mounted on a boom aimed vertically downward at the pavement surface (Figure 27).

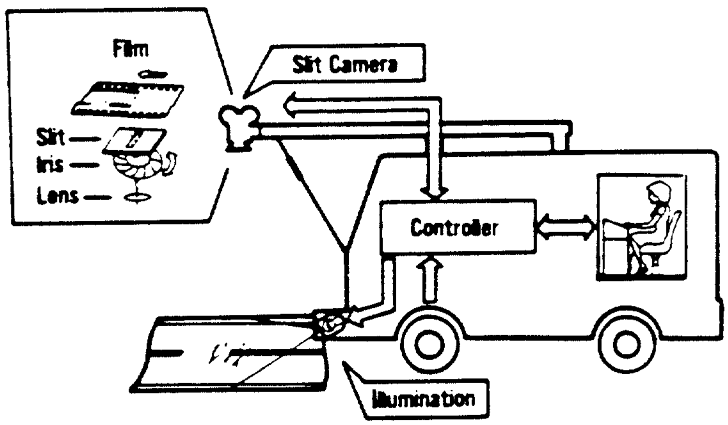


Figure 27 - Schematic of a Continuous Surface Photography Vehicle

The system synchronizes film speed and camera aperture with the speed of the vehicle in order to equalize the image density and photographic reduction. The camera position relative to the pavement is fixed (9.5 feet) resulting in a 1:200 scaled photo reproduction. Pavement widths up to 16 feet can be photographed in each pass. The vehicle operates at night using controlled external illumination. The lights are installed at an angle to the road surface to produce shadows highlighting cracks and other surface defects. The continuous strip photos are saved as a permanent record of the pavement surface condition. In the office, the film is analyzed frame by frame on a film digitizer. Pavement distresses are measured and recorded into a permanent file.

One manufacturer also uses a 35-mm pulse camera and a hairline projector strobe light to photograph rutting wave patterns. The projected hairline parallels the pavement surface, providing an accurate scaled measurement of the rut depth. If no rutting exists and the cross section is uniform, the photographed hairline is straight. In rutted sections the hairline follows the photographed rut pattern permitting very accurate calculation of the rut depth. Both the strobe light and the camera trigger at fixed, adjustable, pre-programmed intervals. The film is then sent for developing, printing, and plotting of the transverse profile.

This system is presently the most fully operational, proven system to measure a wide range of types and severity of pavement distress. The frame by frame analysis, however, is tremendously time consuming, adding significantly to the cost of the system. Efforts to automate the data reduction process are underway.

Video Technology

Nowhere has the effect of the technological explosion of the past 10 years been more apparent than in the area of video technology. Some agencies have turned to videos to provide permanent records of the pavement surface and for other purposes. Features such as identification of signs, utilities, and safety hardware, have been recorded, as have ongoing construction projects, to provide a record of the work and traffic controls during periodic intervals or at key phases of construction.

Videos are replacing the photologging systems in many agencies. In photo-logging, a 35-mm camera is aimed through the windshield of a van, photographing the features in the field of vision 100 times per mile. Video cameras may be installed either to inventory through the windshield, aimed directly down at the pavement surface, or used in both positions.

Video technology offers several advantages over 35-mm photography. Relatively inexpensive, reusable, off-the-shelf technology is readily available. The tape can be replayed in the field where the quality of the record can be immediately verified. Replacement parts and equipment can be easily obtained. The limitation of video is the inferior resolution of the recorded images compared to photographic records. The resolution gap between the two technologies, however, is rapidly narrowing.

The addition of condition rating keyboards and on board computer systems and other instrumentation, allows the condition survey team to supplement the recorded information with distress observations.. Review and supplemental input to the permanent record may be performed in the office, and added to the field-obtained data. Recent systems have added one or more cameras aimed vertically at the pavement surface.

Automated Video Image Processing

Video technology has been in use for about 10 years, and procedures to fully automate the processing and reproduction of the captured images both in real time (immediately after capture on board the vehicle) and in the office environment are now being developed. The automated processing of the recorded analogue images may take several forms.

Conversion to a digital format suitable for image processing is presently the most common. The more limited resolution of video cameras requires use of a very expensive high resolution camera for the detection of small cracks. The high resolution camera, however, is often incompatible with available recorders and digitizing hardware. In order to overcome this shortcoming, either 2 or 3 cameras mounted closer to the pavement surface, each covering an area of about 4'x 4' to 6'x 6' are now in use. This permits detection of small cracks and other surface defects of about 1/8" in size, or smaller. Constant, controlled lighting eliminates the background shadows of passing vehicles and adjacent structures, and minimizes problems related to the changing of the angle of the sun throughout the day.

Digitized processing involves one frame at a time analysis of the recorded analog images. The first step is known as filtering, a process that removes extraneous information that could be misinterpretated as distress. Next, the signal is segmented. Segmentation is based on an intensity threshold and allows the system to identify distinct objects in the scene. Intensities based on the gray scale range from 0 (the whitest white) to 255 (the blackest black). Each analogue image is divided into a matrix of pixels ranging in size from 256 x 256, 512 x 512, or 1024 x 1024, depending on the resolution of the camera. Each pixel in the matrix is assigned a numerical value based on its intensity. A small portion (10 x 10) of a typical matrix is depicted in Figure 28.

In this elementary example, most of the pixels are representative of the normal pavement surface. Processors apply a statistical analysis algorithm to the pixel distribution and establish the range of pixels representing the normal pavement. In this case, values below 107 represented the normal pavement. Values of 107 and above (bold) represent distress, in this case a crack.

99	111	103	98	97	100	103	100	101	102
104	102	110	100	100	104	104	100	102	101
112	**114**	**109**	**114**	99	101	100	96	100	104
107	**112**	**119**	**108**	**111**	**110**	**109**	**111**	**100**	**99**
101	101	98	98	103	104	104	**112**	**115**	**114**
97	96	99	100	101	102	100	100	97	95
101	104	100	97	99	101	102	98	100	100
99	103	103	98	97	100	103	100	101	102
95	97	99	104	103	101	102	100	99	98
104	103	100	97	98	101	102	98	100	100

Figure 28 - A portion of a Pixel Matrix

The resultant reproduced data output may be represented for the covered area in a graphic resembling Figure 29.

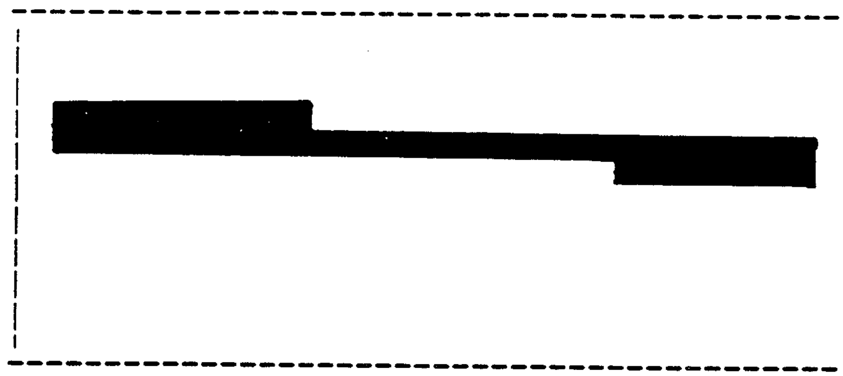


Figure 29 - Reproduction of Pavement Crack through Digitized Processing

The extent and severity can be determined from the matrix and the power of the software. For a 512 x 512 matrix representative of a 4' x 4' pavement section, each pixel would represent an area of pavement measuring 0.09375 square inch, or a little less than 1/10 square inch. Three dark pixels side by side would represent a crack slightly over 1/4 inch wide.

Processors apply software algorithms to identify the edge of the distress, the perimeter and area, and the distress classification. Summary information on distress characteristics is stored and used to create pavement condition summary statistics at intervals specified by the operator. Ultimately, scaled reproductions of the data may be produced on maps; the type, extent, and severity of the distress summarized, and the output used during pavement rehabilitation.

In a 512 x 512 matrix each image contains 262,144 pixels. In standard video formats 30 frames per second are recorded, resulting in the formation of almost 8 million pixels per second. Assuming 3 cameras are used to record a 12 foot wide pavement section, with each camera covering a 4 foot width, about 85 billion pixels would require analysis and processing for each mile of pavement (at 60 mph). Processing obviously requires an extremely high speed microprocessor. Even with today's high speed computers, most data is post processed in the office environment due to the amount of data collected. It is expected that the technology which will be available in a few years to process even this vast amount of information in real time. A summary of these and other distress data collection technologies is contained in Table 3.

TABLE 3 - PAVEMENT DISTRESS DATA COLLECTION TECHNOLOGIES

DATA COLLECTION TECHNOLOGY	DATA REDUCTION TECHNOLOGY	CURRENT USE	DEVELOPMENT TIME IN USE	RELATIVE COST	ADVANTAGES	LIMITATIONS	COMMENTS
Condition Rating Keyboards	Real Time or Office (video)	Growing, Supplements Video, Network	Operational 10 Years	Low	Low cost Eliminates forms	Subjective Data Entry	Inexpensive, great potential use for small agencies
Acoustic Systems	Real Time or Post Process	Growing, also used for Profile, Network	Operational 10 Years	Low to Medium	Low cost for extensive data	Low resolution Affected by weather	Low resolution Affected by weather
Laser Technology	Real Time Analogue & Digital	Growing, also used for Profile & Skid, Network	Operational 8 Years	Medium to High	Highly accurate & fast, great potential	Expensive	Further development needed for Cracking, skid, & Deflection
Photographic Systems	Post Process Analogue	One Evaluation Completed Research	Operational, 18 yrs 1 Year in USA	High	Permanent Record Good Resolution	Expensive, Manual Processing	Operational system Also for Profile
Video Technology	Post Process Analogue	Growing, Multipurpose	Operational 10 Yrs Being Automated	High	Permanent Record, Many Uses, Great Potential	Expensive, Manual Processing	Uses-Traffic, Safety, potential for rutting & skid
Automated Image Processing	Real Time or Post Process	Evaluations Scheduled Use to date Limited	In Development	Very High	Permanent Record Fully Objective	Technically complex Expensive	May provide extensive coverage and reduce cost, if successful
Seismic & Dynamic Testing	Real Time	Experimental	Experimental	Low	Low Cost	Not yet fully developed	Potential to be Determined
Ground Penetrating Radar	Real Time	Project Level	Experimental Refinements Ongoing	Medium	Detects Problems Not Visible on Surface	Output Reliability depends on experienced technician	Used primarily at the project level
Infrared Thermography	Real Time	Project Level	Experimental Refinements Ongoing	Low to Medium	Detects Problems Not Visible on Surface	Output Reliability depends on experienced technician	Used primarily at the project level
Slit Integration	Real Time or Post Process	None	Formerly Experimental	Medium to High?	Used to develop alternate technologies	Never fully developed Low % crack detection	Never fully developed
Flying Spot Laser Scanners	To be determined	None	In development	To be determined	To be determined	To be determined	In development

OTHER TECHNOLOGIES

Other technologies are also now available and are being used and developed for the purpose of evaluating pavements. These include seismic and dynamic test methods, ground penetrating radar, infrared thermography, slit integration, and flying spot laser scanners. A few of these technologies are now in use, others are being evaluated, developed, or refined. A summary of their respective features is contained in Table 3.

CONCLUSION

Automated processes to collect skid, roughness, and rutting are now available. Automated equipment to measure cracks, and moving deflectometers are still in the research and evaluation stages. Improvements in the ability to collect important pavement data through equipment automation can be expected to continue.
The technological explosion of the 1980's has been a great benefit to Pavement Management Systems. Technologies which were only a theory in the 1960's are now being used on an every day basis. The rate of development of many of the new and innovative technologies is expected to continue into the 1990's and the 21st century. Automated data collection and processing of collected pavement condition data can provide considerable cost savings to user agencies, and improve the mechanisms through which agencies effectively manage their pavements.

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APPENDIX A

LIST OF AUTOMATED PAVEMENT DATA COLLECTION EQUIPMENT MANUFACTURERS & SUPPLIERS*

Cox & Sons, Inc.
P.O. Box 674
Colfax, CA 95713
(916) 346-8322
Cox RTRRM
Friction Tester

Dynatest Consulting, Inc.
209 Bald Street
P.O. Box 71
Ojai, California 93203
(805) 646-2230
Dynatest 8000 FWD
Dynatest 5000 RDM

Foundation Mechanics Inc.
421 East El Segundo Boulevard
El Segundo, CA 91245
(213) 322-1920
Road Rater

Highway Products International
R.R. #1 Paris, Ontario
Canada N3L 3E1
Mr. Don Kobi
(519) 442-2261
ARAN & PURD

Infrastructure Management Services
3350 Salt Creek Lane
Arlington Heights, IL 60005
Mr. Robert L. Novak
(312) 506-1500
Swedish Laser RST
690DNC Profilometer

K. J. Law Engineers
42300 W. Nine Mile Rd.
Novi, Michigan 48050-3627
Mr. Ken Law
(313) 347-3300
8300A Roughness Surveyor
Friction Testers

Kokusai Kogyo., Ltd.
Abenatak Corporation
1179 Fernwood Drive
Millbrae, CA 94030
Mr. Taizo Abe
Roadman

MHM Associates, Inc.
1920 Ridgedale Road
South Bend IN 46614
Mr. Jerry Mohajeri
(219) 291-4793
ARIA

MAP Inc.
1825 I Street, NW. Suite 400
Washington, D.C. 20006
Mr. Michael Grippon
(202) 429-2089
Gerpho

PASCO USA Inc.
1-J Franssetto Way
Lincoln Park, N.J. 07035
Mr. Wade Gramling
(201) 628-8433
Pasco Road Survey System

Pavement Condition Evaluation Services
1145 Icehouse Avenue
Sparks, Nevada 89431
Mr. Bert Butler
(702) 355-0225
Pavement Distress Imager 1

Pavedex, Inc.
N. 800 Hamilton Ave.
Spokane, Washington 99202
Mr. Don Bender
(509) 483-4126
Pavedex PAS I

South Dakota DOT
700 Broadway Avenue East
Pierre, SD 57501
Mr. David L. Huft
(605) 773-3871
South Dakota Road Profiler

VideoComp
500 Sawtooth Ave.
Boise, Idaho 83709
Mr. Basil Dahlstrom
(208) 385-1575
Distress Survey Trailer

* This list has been prepared on the basis of currently available informati in this office. We would appreciate the readers' input if others are known

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U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: "Addressing Institutional Barriers to Implementing a PMS" ASTM Paper by Dr. Roger E. Smith

Date: AUG 1 9 1991

From: Chief, Pavement Division

Reply to Attn of: HNG-41

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

At the recent American Society for Testing Materials Symposium (ASTM) on Pavement Management, the attached stand-out paper, was given by Professor Roger E. Smith, of the Texas Transportation Institute. Dr. Smith has given his permission for us to share his paper with others interested in Pavement Management Systems (PMS).

The paper addresses institutional barriers to implementing PMS (using the San Francisco Bay Area Metropolitan Transportation Commission (MTC) as an example) and is not only an excellent description of these issues, but is also a perfect lead into the September symposium that we are co-sponsoring with the Illinois Department of Transportation (IDOT). You may wish to share it with our divisions and the State highway agencies.

As the States move from development of their PMS's into implementation, success could well hinge on recognition that these barriers exist, and taking steps to overcome them.

Key issues identified in the paper are:

1. If the PMS is not affecting the ultimate decisions on the highway agency's program, then the agency cannot claim to have a PMS implemented.

2. Many PMS's are not being utilized to their fullest potential--often because of surmountable barriers.

3. The most troublesome barriers to implementation of a PMS are people related.

4. Some of the barriers in capsule are:

   - Fear of exposure - Previous decisions may have been incorrect.

   - Turf Protection - There are formal and informal lines of communication within an organization that need to be recognized.

- Black Box Syndrome - The PMS is not fully understood, therefore its products can not be verified.

- One person show - The champion of the PMS leaves and the system dies.

5. Some of the concepts to be considered in surmounting barriers are:

- Innovation - "It is not the actual newness of the innovation but rather the perception of newness to the potential adopter that influences adoption and use."

- Consider the "Opinion Leader" - Know who they are because without their support the system will not be implemented.

- Compatibility - The new system must be compatible with the existing because a complete overhaul or reorganization is usually impossible.

- Trialability - Innovation needs to be adopted in stages. "New ideas that can be tried on a limited basis are more likely to be adopted."

- Observability - This is the degree to which results of innovations are visible to others. If you can show positive results it goes a long way toward selling PMS.

These are of course, only key paraphrasing of the paper and a full understanding of the issues requires a complete reading. We recommend it to all with responsibility for implementation of PMS's.

Louis M. Papet

Attachment

Order

U.S. Department of Transportation
Federal Highway Administration

Subject

PAVEMENT MANAGEMENT COORDINATION

Classification Code	Date
5080.3	April 13, 1992

Par. 1. Purpose
2. Cancellation
3. Washington Headquarters Coordination
4. Field Coordination

1. PURPOSE. To provide a forum for coordinating the Federal Highway Administration's (FHWA) Pavement Management Program in the Washington Headquarters, and to provide guidance for coordination in FHWA field offices.

2. CANCELLATION. FHWA Order 5080.2, Pavement Management Coordination, dated March 23, 1987, is canceled.

3. WASHINGTON HEADQUARTERS COORDINATION

a. Pavement issues often involve the activities of several Washington Headquarters offices. To ensure coordination among related offices and activities, a Pavement Management Coordination Group (PMCG) was established in 1980. The membership has been revised from time to time to reflect organizational changes in the Washington Headquarters since establishment of the group.

b. The PMCG consists of the following members:

(1) Chief, Pavement Division, Office of Engineering, will Chair the Group.

(2) Chief, Planning Programs Division, Office of Environment and Planning.

(3) Director, Office of Engineering and Highway Operations Research and Development.

(4) Director, Office of Technology Applications.

(5) Director, Office of Policy Development.

(6) Director, Office of Highway Information Management.

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(7) Director, Office of Motor Carrier Information Management and Analysis.

(8) A Regional Administrator appointed by the Executive Director will serve for 2 years. This assignment will be rotated among the Regional Administrators.

(9) Chief, Pavements Division, Office of Engineering and Highway Operations Research and Development.

(10) Chief, Long Term Pavement Performance Division, Office of Engineering and Highway Operations Research and Development.

(11) Chief, Materials Division, Office of Engineering and Highway Operations Research and Development.

(12) Chief, Engineering Applications Division, Office of Technology Applications.

(13) Chief, Strategic Highway Research Program Implementation Staff, Office of Technology Applications.

c. The PMCG has the following responsibilities:

(1) ensure that pavement related activities [including the need for, the gathering of, and the use of pavement data] by the several Washington Headquarters offices are cooperatively developed and properly coordinated;

(2) identify pavement problems or issues which merit attention by the FHWA;

(3) participate in field reviews as requested or needed;

(4) serve as the Research, Development, and Technology Coordinating Group for the pavement area as required by FHWA Order 6000.2, Research, Development, and Technology Advisory Councils;

(5) support the FHWA involvement in the Strategic Highway Research Program (SHRP) and the implementation of the products resulting from SHRP, and coordinate the activities of the various programs, studies, and groups involved in the Long Term Pavement Performance evaluation; and

(6) recommend FHWA policies, programs, or actions to improve effectiveness of pavement-related activities of FHWA, State highway agencies (SHA), and local governments.

d. A Technical Working Group (TWG) will assist the PMCG in developing and monitoring pavement-related activities. Each Office Director in the PMCG will appoint at least one representative to serve on the TWG.

(1) The Chief, Pavements Division, Office of Engineering and Highway Operations Research and Development will chair the TWG.

(2) The chairperson may create working groups for specific technical areas as they deem necessary. Each working group will elect its own chairperson, who shall be a member of the TWG.

4. <u>FIELD COORDINATION</u>

a. To provide necessary emphasis to pavement management activities, and to provide the support required to implement FHWA policies and programs in the pavement area, FHWA field offices should develop coordinating mechanisms and assign specific responsibilities for pavement activities.

b. Each regional office should:

(1) assure coordination of pavement-related activities within the region, including but not limited to research, technology transfer, Highway Performance Monitoring System (HPMS), SHRP, annual conferences, and vehicle weight enforcement program;

(2) develop regional operations plans to assist division offices in aiding States and local governments in the conduct of coordinated pavement management programs;

(3) monitor and evaluate pavement-related activities within the region and provide recommendations to improve their effectiveness;

(4) designate a pavement management coordinator to serve as the focal point for all pavement-related activities in the region; and

(5) promote the full range of resources available to strengthen the technical capabilities of local government, SHA, and FHWA personnel in pavement-related areas. Resources available include annual conferences, training courses, and other technology transfer activities and materials, as well as technical expertise within the FHWA staff.

c. Each division office is expected to:

(1) designate a pavement management coordinator to ensure that pavement-related activities, including new and rehabilitated pavement design and construction, pavement management, research, technology transfer, HPMS, vehicle weight enforcement program, etc., are well coordinated among functional/administrative areas of the division office;

(2) monitor and evaluate pavement activities and programs, determine short and long-term needs, and formulate operations plans for meeting these needs; and

(3) take full advantage of available resources to strengthen the technical capabilities of local government, SHA, and FHWA personnel in pavement-related areas. Resources available include annual conferences, training courses, other technology transfer activities and materials, as well as technical expertise within the FHWA staff.

T. D. Larson
Federal Highway Administrator

The NHS Designation Act of 1995, Section 205(a) - Suspension of Management Systems, made PMS a State option and no longer required by federal law. The interim management systems regulation is being revaluated.

SUBCHAPTER F—TRANSPORTATION INFRASTRUCTURE MANAGEMENT

PART 500—MANAGEMENT AND MONITORING SYSTEMS

Authority: 23 U.S.C. 134, 135, 303 and 315; 49 U.S.C. app. 1607; 23 CFR 1.32; and 49 CFR 1.48 and 1.51.

Subpart A—General

§ 500.101 Purpose.

The purpose of this part is to implement the requirements of 23 U.S.C. 303, Management Systems, which requires State development, establishment, and implementation of systems for managing highway pavement of Federal-aid highways (PMS), bridges on and off Federal-aid highways (BMS), highway safety (SMS), traffic congestion (CMS), public transportation facilities and equipment (PTMS), and intermodal transportation facilities and systems (IMS). Section 303 also requires State development, establishment, and implementation of a traffic monitoring system for highways and public transportation facilities and equipment. This subpart includes definitions and general requirements that are applicable to all of these systems. Additional requirements applicable to a specific system are included in subparts B through H of this part.

§ 500.103 Definitions.

Unless otherwise specified in this part, the definitions in 23 U.S.C. 101(a) are applicable to this part. As used in this part:

Certifying official(s) means the position(s) designated by the Governor of a State or the Commonwealth of Puerto Rico or the Mayor of the District of Columbia to certify that the management system(s) is/are being implemented in the State.

Cooperation means working together to achieve a common goal or objective.

Federal agency(ies) means for the PMS and BMS, the Federal Highway Administration (FHWA); for the SMS, the FHWA and the National Highway Traffic Safety Administration; for the CMS, PTMS, and IMS, the FHWA and the Federal Transit Administration (FTA).

Federal-aid highways means those highways eligible for assistance under title 23, U.S.C., except those functionally classified as local or rural minor collectors.

Highway Performance Monitoring System (HPMS) means the State/Federal system used by the FHWA to provide information on the extent and physical condition of the nation's highway system, its use, performance, and needs. The system includes an inventory of the nation's highways including traffic volumes.

Life-cycle cost analysis means a procedure for evaluating the economic worth of one or more projects or investments by discounting future costs over the life of the project or investment.

Management system means a systematic process, designed to assist decisionmakers in selecting cost-effective strategies/actions to improve the efficiency and safety of, and protect the investment in, the nation's transportation infrastructure. A management system includes: Identification of performance measures; data collection and analysis; determination of needs; evaluation and selection of appropriate strategies/actions to address the needs; and evaluation of the effectiveness of the implemented strategies/actions.

Metropolitan planning area means the geographic area in which the metropolitan transportation planning process required by 23 U.S.C. 134 and section 8 of the Federal Transit Act (49 U.S.C. app. 1607) must be carried out.

Metropolitan planning organization (MPO) means the forum for cooperative transportation decisionmaking for a metropolitan planning area.

National highway system (NHS) means the system of highways designated and approved in accordance with the provisions of 23 U.S.C. 103(b).

Performance measures means operational characteristic, physical condition, or other appropriate parameters used as a benchmark to evaluate the adequacy of transportation facilities and estimate needed improvements.

State means any one of the fifty States, the District of Columbia, or Puerto Rico.

Transportation Management Area (TMA) means an urbanized area with a population over 200,000 (as determined by the latest decennial census) or other area when TMA designation is requested by the Governor and the MPO (or affected local officials), and officially designated by the Administrators of the FHWA and the FTA. The TMA designation applies to the entire metropolitan planning area(s).

Work plan means a written description of major activities necessary to develop, establish, and implement a management or monitoring system, including identification of responsibilities, resources, and target dates for completion of the major activities.

§ 500.105 Development, establishment, and implementation of the systems.

(a) Each State shall develop, establish, and implement the systems identified in § 500.101. Each State shall tailor the systems to meet State, regional, or local goals, policies, and resources, but the systems must meet the requirements as specified in subparts B through H of this part. Documentation that describes each management system shall be maintained by the States for the Federal agencies to determine, on a periodic basis, whether the systems meet the requirements in this subpart and subparts B through H of this part, as applicable.

(b) Each State shall have procedures, within the State's organization, for coordination of the development, establishment, implementation and operation of the management systems. The procedures must include:

(1) An oversight process to assure that adequate resources are available for implementation and that target dates in the work plan(s) are met;

(2) The use of data bases with a common or coordinated reference systems and methods for data sharing; and

(3) A mechanism to address issues related to the purposes of more than one management system.

(c) In developing and implementing each management system, the State shall cooperate with MPOs in metropolitan areas, local officials in non-metropolitan areas, affected agencies receiving assistance under the Federal Transit Act and other agencies (including private owners and operators) that have responsibility for operation of the affected transportation systems or facilities.

(d) In accordance with the provisions of 23 U.S.C. 134(l)(3) and 49 U.S.C. app. 1607(l)(3) and the requirements of 23 CFR part 450, the CMS shall be part of the metropolitan planning process in TMAs.

(e) Within metropolitan planning areas, the CMS, PTMS, and IMS shall, to the extent appropriate, be part of the metropolitan transportation planning process required under the provisions of 23 U.S.C. 134 and 49 U.S.C. app. 1607.

(f) In metropolitan planning areas that have more than one MPO and/or that include more than one State, the establishment, development, and implementation of the CMS, PTMS, and IMS shall be coordinated among the State(s) and MPO(s) to ensure compatibility of the systems and their results.

(g) The results (e.g., policies, programs, projects, etc.) of the individual management systems shall be considered in the development of metropolitan and statewide transportation plans and improvement programs and in making project selection decisions under title 23, U.S.C., and under the Federal Transit Act.

(h) The roles and responsibilities of the State, MPO(s), recipients of assistance under the Federal Transit Act, and other agencies involved in the development, establishment, and implementation of each system shall be mutually determined by the parties involved. A State may enter into agreements with local governments, regional agencies (such as MPOs), recipients of funds under the Federal Transit Act, or other entities to develop, establish, and implement appropriate parts of any or all of the systems, but the State shall be responsible for overseeing and coordinating such activities.

(i) Section 204(a) of title 23, U.S.C., requires the Secretary in cooperation with the Secretaries of the Interior and Agriculture to develop the safety, bridge and pavement management systems for Federal lands highways, as defined in 23 U.S.C. 101(a). To avoid duplication of effort, the management systems required under this part should be used to the extent appropriate to fulfill the requirement in 23 U.S.C. 204(a) regarding establishment and implementation of pavement, bridge, and safety management systems for Federal lands highways. The State, the Federal agencies, and the agencies that own the roads shall cooperatively determine responsibility for coverage of Federal lands highways under their respective jurisdictional control and shall ensure that the results of the PMS, BMS, and SMS for Federal lands highways are available, as appropriate, for consideration in developing metropolitan and statewide transportation plans and improvement programs and are provided to the FHWA for use in developing Federal lands highway programs.

(j) Each management system must include appropriate means to evaluate the effectiveness of implemented actions developed through use of that system. The effectiveness of the management systems in enhancing transportation investment decisions and improving the overall efficiency of the State's transportation systems and facilities shall be evaluated periodically, preferably as part of the metropolitan and statewide planning processes.

§ 500.107 Compliance.

(a) States must be implementing the management systems specified in subparts B through G of this part beginning in Federal fiscal year 1995 (October 1, 1994 to September 30, 1995) and must certify annually to the Secretary of Transportation that they are implementing each of the management systems. A State shall be considered to be implementing a management system if the system is under development or in use in accordance with the compliance schedule for that system as specified in subparts B through G of this part.

(b) The Governor of the State or the Commonwealth of Puerto Rico or the Mayor of the District of Columbia shall notify the FHWA Division Administrator in writing by September 30, 1994, of the title(s) of the certifying official(s) for each management system. If there is a change in designated position(s), the State shall provide documentation of the revised designation with, or prior to, the next annual certification. In those States where responsibility for all of the management systems is within a single agency (e.g., State DOT), designation of one certifying official for all of the management systems is recommended.

(c) The certification statement(s) shall be submitted by the certifying official(s) to the FHWA Division Administrator by January 1 of each year, beginning January 1, 1995. To the extent possible, one certification statement should cover all six management systems. If more than one certification statement will be submitted by a State, the statements should be coordinated at the State level and submitted simultaneously. The first certification statement shall include a copy of the workplan(s), required in accordance with the compliance schedule for each management system, and a summary of the status of implementation of the management system(s). Subsequent certification statement(s) shall include a summary of the status of implementation of each management system and a discussion of planned corrective actions for any management system(s) or subsystem(s) that are not under development or fully operational in accordance with the compliance schedule and work plan for the management system.

(d) The FHWA Division Administrator will provide copies of the certification statement(s) and any relevant supporting documentation and correspondence to other Federal agencies identified for the specific system(s) in § 500.103. Within 90 days of receipt, the Federal agencies will review the certification and the FHWA Division Administrator will notify the State whether the certification is acceptable or if sanctions may be imposed in accordance with the provisions of § 500.109.

(e) A State shall be considered to be implementing the traffic monitoring system for highways (TMS/H), specified in subpart H of this part, if the system is under development or in use in accordance with the compliance schedule in § 500.809. The State shall submit the work plan for the TMS/H to the FHWA Division Administrator by January 1, 1995.

(The information collection requirements in paragraphs (c) and (e) of § 500.107 have been approved by the Office of Management and Budget under control number 2125–0555.)

§ 500.109 Sanctions.

(a) Beginning January 1, 1995, if a State fails to certify annually as required by this regulation, or if the Federal agencies determine that any management system or subsystem, specified in subparts B through G of this part, is not being adequately implemented, notwithstanding the State's certification(s), the Secretary may withhold up to 10 percent of the funds apportioned to the State under title 23, U.S.C., and to any recipient of assistance under the Federal Transit Act for any fiscal year beginning after September 30, 1995. Sanctions may be imposed on a statewide basis, on a subarea of a State, for specific categories of funds or types of projects, or for specific recipients or subrecipients of funds under title 23, U.S.C., or under the Federal Transit Act depending on the adequacy of implementation of the management systems.

(b) While a State may enter into agreements with local governments or other agencies to develop, establish, and implement all or parts of the management systems, in accordance with § 500.105(g), the State shall be responsible for ensuring that the systems are being implemented statewide and for taking any necessary corrective action, including implementing the systems at the regional and local levels if necessary.

(c) Prior to imposing a sanction, a State will be notified in writing by the FHWA of the sanction(s) to be imposed, the reasons for the sanctions, and the actions necessary to correct the deficiencies. After 60 days from the date of notification to the State, the Federal agencies will consider any corrective actions proposed by the State and the FHWA will notify the State if such actions are acceptable or if sanctions are to be applied.

(d) In instances where a State, or responsible sub-unit of a State or recipient of funds under the Federal Transit Act, has not fully implemented all of the management systems, consideration shall be given by the Federal agencies to efforts underway or planned to make the systems fully operational within a reasonable time period.

(e) To the extent that they have not lapsed, funds withheld pursuant to this subpart shall be made available to the State or recipient under the Federal Transit Act upon a determination by the Federal agencies that the management systems are being adequately implemented.

§ 500.111 Funds for development, establishment, and implementation of the systems.

(a) The following categories of funds may be used for development, establishment, and implementation of any of the management and monitoring systems: National Highway System, Surface Transportation Program, FHWA State planning and research and metropolitan planning funds (including the optional use of minimum allocation funds authorized under 23 U.S.C. 157(c) for carrying out the provisions of 23 U.S.C. 307(c)(1) and 23 U.S.C. 134(a)), Federal Transit Act Section 8 (49 U.S.C. app. 1607), Federal Transit Act Section 9 (49 U.S.C. app. 1607a), Federal Transit Act Section 26(a)(2) (49 U.S.C. app. 1622(a)(2)), and Federal Transit Act Section 26(b)(1) (49 U.S.C. app. 1626(b)(1)). Congestion Mitigation and Air Quality Improvement Program funds (23 U.S.C. 104(b)(2)) may be used for those management systems that can be shown to contribute to the attainment of a national ambient air quality standard. Apportioned bridge funds (23 U.S.C. 144(e)) may be used for development and establishment of the bridge management system.

(b) Federal funds identified in paragraph (a) of this section used for development, establishment, or implementation of the management and monitoring systems shall be administered in accordance with the procedures and requirements applicable to the category of funds.

§ 500.113 Acceptance of existing management systems.

(a) Existing State laws, rules, or procedures that the Federal agencies determine fulfill the purposes of a management system, or portion thereof, as specified in this part may be accepted by the Federal agencies in lieu of development and implementation of a new system.

(b) If a State has existing laws, rules, or procedures that it wants to use to meet the requirements of this part, it shall submit a written request to the FHWA Division Administrator that the Federal agencies accept the existing management system in lieu of development of a new system. The request shall include a discussion, and any necessary supporting documentation, that shows how the existing system meets the requirements of this part. The documentation shall reflect the views of the MPOs, transit operators, and other affected agencies, as appropriate, and the actions to be taken to assure that the cooperation required under § 500.105(c) is established.

(c) Upon receipt of a request, the FHWA Division Administrator will coordinate review of the request with the other Federal agencies specified in § 500.103 and with appropriate FHWA offices. Within 90 days of receipt of the State's request, the FHWA will notify the State that the existing system is either fully acceptable, acceptable subject to specific modifications, or unacceptable and that a new system must be developed.

(d) To meet the compliance schedule for a system, the State must submit any requests under paragraph (a) of this section no later than June 1, 1994.

Subpart B—Pavement Management System

§ 500.201 Purpose.

The purpose of this subpart is to set forth requirements for development, establishment, implementation, and continued operation of a pavement management system (PMS) for Federal-aid highways in each State in accordance with the provisions of 23 U.S.C. 303 and subpart A of this part.

§ 500.203 PMS definitions.

Unless otherwise specified in this part, the definitions in 23 U.S.C. 101(a) and § 500.103 are applicable to this subpart. As used in this part:

Pavement design means a project level activity where detailed engineering and economic considerations are given to alternative combinations of subbase, base, and surface materials which will provide adequate load carrying capacity. Factors which are considered include: materials, traffic, climate, maintenance, drainage, and life-cycle costs.

Pavement management system (PMS) means a systematic process that provides, analyzes, and summarizes pavement information for use in selecting and implementing cost-effective pavement construction, rehabilitation, and maintenance programs.

§ 500.205 PMS general requirements.

(a) Each State ~~shall have~~ a PMS for Federal-aid highways that meets the requirements of § 500.207 of this subpart.

(b) The State is responsible for assuring that all Federal-aid highways in the State, except those that are federally owned, are covered by a PMS. Coverage of federally owned public roads shall be determined cooperatively by the State, the FHWA, and the agencies that own the roads.

(c) PMSs should be based on the concepts described in the "AASHTO Guidelines for Pavement Management Systems." [1]

(d) Pavements shall be designed to accommodate current and predicted traffic needs in a safe, durable, and cost-effective manner.

§ 500.207 PMS components.

(a) The PMS for the National Highway System (NHS) shall, as a minimum, consist of the following components:

(1) Data collection and management.

(i) An inventory of physical pavement features including the number of lanes, length, width, surface type, functional classification, and shoulder information.

(ii) A history of project dates and types of construction, reconstruction, rehabilitation, and preventive maintenance.

(iii) Condition surveys that include ride, distress, rutting, and surface friction.

(iv) Traffic information including volumes, classification, and load data.

(v) A data base that links all data files related to the PMS. The data base shall be the source of pavement related information reported to the FHWA for the HPMS in accordance with the HPMS Field Manual.[2]

(2) Analyses, at a frequency established by the State consistent with its PMS objectives.

(i) A pavement condition analysis that includes ride, distress, rutting, and surface friction.

(ii) A pavement performance analysis that includes an estimate of present and predicted performance of specific pavement types and an estimate of the remaining service life of all pavements on the network.

(iii) An investment analysis that includes:

(A) A network-level analysis that estimates total costs for present and projected conditions across the network.

(B) A project level analysis that determines investment strategies including a prioritized list of recommended candidate projects with recommended preservation treatments that span single-year and multi-year periods using life-cycle cost analysis.

(C) Appropriate horizons, as determined by the State, for these investment analyses.

(iv) For appropriate sections, an engineering analysis that includes the evaluation of design, construction, rehabilitation, materials, mix designs, and preventive maintenance as they relate to the performance of pavements.

(3) Update. The PMS shall be evaluated annually, based on the agency's current policies, engineering criteria, practices, and experience, and updated as necessary.

(b) The PMS for Federal-aid highways that are not on the NHS shall be modeled on the components described in paragraph (a) of this section, but may be tailored to meet State and local needs. These components shall incorporate the use of the international roughness index or the pavement serviceability rating data as specified in Chapter IV of the HPMS Field Manual.

§ 500.209 PMS compliance schedule.

(a) By October 1, 1994, the State shall develop a work plan that identifies major activities and responsibilities and includes a schedule that demonstrates full operation and use of the PMS on the NHS by October 1, 1995, and on non-NHS Federal-aid highways by October 1, 1997.

(b) By October 1, 1995:

(1) The PMS for the NHS shall be fully operational and shall provide projects and programs for consideration in developing metropolitan and statewide transportation plans and improvement programs; and

(2) PMS design for non-NHS Federal-aid highways shall be completed or underway in accordance with the State's work plan.

(c) By October 1, 1997, the PMS for non-NHS Federal-aid highways shall be fully operational and shall provide projects and programs for consideration in developing metropolitan and statewide transportation plans and improvement programs.

[1] AASHTO Guidelines for Pavement Management Systems, July 1990, can be purchased from the American Association of State Highway and Transportation Officials, 444 N. Capitol Street, NW., suite 225, Washington, DC 20001. Available for inspection as prescribed in 49 CFR part 7, appendix D.

[2] Highway Performance Monitoring System (HPMS) Field Manual for the Continuing Analytical and Statistical Data Base, DOT/FHWA, August 30, 1993, (FHWA Order M5600.1B). Available for inspection and copying as prescribed in 49 CFR part 7, appendix D.

CHAPTER 10

STRATEGIC HIGHWAY RESEARCH PROGRAM

10.1 Strategic Highway Research Program Product Implementation Status Report, December 1995 (Published Quarterly)

10.2 Strategic Highway Research Program (SHRP), Information Clearinghouse, July 22, 1994.

10.3 Strategic Highway Research Program (SHRP)
- Implementation Plan for SHRP Products, June 3, 1993.
- SHRP Products Implementation Plan, November 22, 1993.
- Asphalt Research Output and Implementation Program, September 1993.

10.4 Reserved

10.5 Office of Technology Applications,
- SHRP Technology Applications Programs - April 1994 (Published Semi-annually)

DEMONSTRATION PROJECTS

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DP-84	Corrosion Survey Techniques
DP-87	Drainable Pavements
DP-87	Drainable Pavement Systems (Phase II)
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CHAPTER 10

STRATEGIC HIGHWAY RESEARCH PROGRAM

10.5 Office of Technology Applications,

TESTING AND EVALUATION PROJECTS

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STATUS REPORT

Strategic Highway Research Program Product Implementation

December 1995

U.S. Department of Transportation
Federal Highway Administration

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FOREWORD

The Strategic Highway Research Program (SHRP) was conceived and funded by State highway departments as a means of developing new technologies for designing and maintaining longer-lasting, safer roadways. During the 5-year program, experts in materials, construction, maintenance, traffic operations, and other areas focused on developing better ways of building and maintaining roads and bridges.

The research program ended in 1993. Since then, the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), and the Transportation Research Board (TRB) have been working with highway agencies and industry on the implementation of SHRP products. This Status Report, which is published periodically, summarizes the activities and projects currently under way for implementing the products of the Strategic Highway Research Program.

If you are familiar with the SHRP technologies and have followed the development of the implementation activities, the information in the Status Report gets right to the heart of the subject. However, if you are not quite so familiar with the subject, the Status Report may actually generate more questions. In those cases where the "bridge" is not complete, we encourage you to pick up the telephone and contact the chairman or secretary of the appropriate technical working group for additional information.

The strategic plan for SHRP implementation is described in the *Implementation Plan—SHRP Products* (June 1993, FHWA-SA-93-054). The plan describes the internal and external organizational structure, partners and partnerships, purposes, roles, and the implementation mechanisms and support functions that are used to accomplish the program. The plan provides the framework under which the partnerships function in developing the detailed product implementation plans.

FHWA provides several sources of information and assistance with SHRP products, including the following:

- Pooled-fund purchases of new test equipment.
- Test and evaluation projects.
- Training, equipment demonstrations, workshops, and exhibits.
- SHRP Information Clearinghouse, a computerized, on-line source of information on FHWA's SHRP implementation activities.
- *Focus*, a monthly newsletter reporting on State, Federal, and industry initiatives for implementing SHRP products.

Technical Working Group Contacts

Asphalt

Chairman: Gerry Eller, Office of Engineering, 202-366-4853 (fax: 202-366-9981; email: geller@intergate.dot.gov).

Secretary: John D'Angelo, Office of Technology Applications, 202-366-0121 (fax: 202-366-7909; email: jdangelo@intergate.dot.gov).

Highway Operations

Co-Chairmen:

Joe Lasek (Work Zone Safety), Office of Highway Safety, 202-366-2174 (fax: 202-366-2249; email: jlasek@intergate.dot.gov).

Jesse Story (Pavement and Winter Maintenance), Office of Engineering, 202-366-1552 (fax: 202-366-9981; email: jstory@intergate.dot.gov).

Co-Secretaries:

Mike Burk (Work Zone Safety), Office of Technology Applications, 202-366-8033 (fax: 202-366-7909; email: mburk@intergate.dot.gov).

Gary Henderson (Pavement and Winter Maintenance), Office of Technology Applications, 202-366-1283 (fax: 202-366-7909; email: ghenderson@intergate.dot.gov).

Concrete and Structures

Chairman: Suneel Vanikar, Office of Technology Applications, 202-366-0120 (fax 202-366-7909; email: svanikar@intergate.dot.gov).

Secretary: Donald Jackson, Office of Technology Applications, 202-366-6770 (fax: 202-366-7909; email: djackson @intergate.dot.gov).

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Assisting in the development of the overall strategy for SHRP implementation is the Transportation Research Board's SHRP Committee. The committee, composed of top-level managers from industry, State highway agencies, academia, and FHWA, provides oversight to the long-term pavement performance studies and serves as a sounding board for ideas for overcoming institutional barriers to SHRP implementation.

Each State and FHWA regional and division office has designated a SHRP implementation coordinator. So that these coordinators can benefit from each others' experiences, FHWA holds a coordinators meeting each January in Washington, D.C.

The technical working groups and their subgroups, known as expert task groups, are key players in shaping the scope, structure, and content of the SHRP implementation program.

The AASHTO Task Force on SHRP Implementation, chaired by Bobbie Templeton of the Texas Department of Transportation, provides coordination and guidance to States in implementing SHRP products.

With local governments responsible for more than 70 percent of our Nation's roads and streets, local highway organizations are prime candidates for implementing SHRP products. FHWA has contracted with Hibbs Highway Engineering Services to assist the Local Technical Assistance Program (LTAP) centers with the delivery of SHRP products to local governments. Toward that end, Hibbs provides the LTAP centers with news articles, technical materials, product exhibits, loaner equipment, and training packages geared to the needs of local highway agencies.

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Long-Term Pavement Performance

Chairman: John Hallin, Office of Engineering, 202-366-1323 (fax: 202-366- 9981; email: jhallin @intergate.dot.gov).

Overall Coordination

Tommy Beatty, Office of Technology Applications, 202-366-8028 (fax: 202-366-7909; email: tbeatty@intergate. dot.gov).

To obtain a copy of the software program needed to access the SHRP Information Clearinghouse, contact Mark Bradley at Tonya (telephone: 202-289-8108; fax: 202-289-8107).

To be added to the *Focus* mailing list, contact Lisa Pope at Harrington-Hughes & Associates (telephone: 202-347-1448; fax 202-347-6938).

Asphalt *UPDATE*

FHWA continues its outreach program to inform the highway community about the Superpave system, which was the primary product of the SHRP asphalt research program.

A new **brochure**, "The Superpave System: New Tools for Designing and Building More Durable Asphalt Pavements," provides an overview of the Superpave system and a list of resources for additional information. The brochure (Publication Number FHWA-SA-96-010) is available from FHWA's Reports Distribution Center (telephone: 703-285-2144, fax: 703-285-2919).

The Superpave system was also the theme of the October 1995 issue of the ***Asphalt Contractor***. FHWA provided several articles for the issue:

- User-Producer Groups Set the Stage for Superpave
- Team Refining Superpave Software
- States Move Forward on Superpave
- Superpave Straight Talk
- Superpave Travels a Rocky Road to Implementation

A new **videotape** on the Superpave volumetric mix design procedures, produced jointly by FHWA and the National Asphalt Pavement Association, will be available in January 1996.

Superpave was very much on the agenda of the recent annual meeting of the American Association of State Highway and Transportation Officials. Augmenting the many presentations and committee meetings on Superpave was FHWA's mobile Superpave laboratory, which was parked outside the meeting site to allow participants a hands-on look at the new test devices.

Superpave

The Superpave (Superior Performing Asphalt Pavements) mix design and analysis system is a significant advancement in hot-mix asphalt pavement design. By taking into account climatic conditions and projected traffic loads, the system allows highway departments and contractors to create pavements that will better resist rutting and cracking and that will last longer.

State highway agencies, roadbuilders, suppliers, and others in the highway industry are in the process of acquiring and learning how to use the battery of new test equipment required for Superpave mixes. This section highlights the progress made by both States and industry in reaching the two target dates for Superpave implementation: adoption of the Superpave binder specification by 1997, and full-scale use of Superpave volumetric mix design by 2000.

Since 1992, FHWA has been providing technical assistance, support, and training in the use of the Superpave system. Those activities are expected to continue until 2000.

Binder Test Equipment

Testing asphalt binders for conformance with the Superpave binder specification requires five principal pieces of equipment:

- Pressure aging vessel, to simulate in-service aging of the binder;
- Rotational viscometer, to determine the flow characteristics of the binder;
- Bending beam rheometer, to measure the binder's low-temperature stiffness;

- Dynamic shear rheometer, to measure the binder's stiffness and phase angle at intermediate and high temperatures;
- Direct tension tester, to measure the low-temperature tensile and fracture properties.

All **States now have** the pressure aging vessel, rotational viscometer, bending beam rheometer, and dynamic shear rheometer. These devices were obtained through a pooled-fund purchase coordinated by FHWA.

In addition, FHWA has **loaned a full set** of the binder test equipment to each of the five regional asphalt user-producer groups. This equipment will be used both for training engineers and technicians and for testing asphalt binder samples provided by State departments of transportation and others.

The prototype for the third generation of the **direct tension tester**, the final piece of necessary binder equipment, is currently undergoing testing and evaluation at FHWA's Turner-Fairbank Highway Research Center (TFHRC). Once this evaluation is complete and necessary changes have been made, FHWA will purchase up to five additional units and loan them to the regional user-producer groups (UPGs) for ruggedness testing. The pooled-fund procurement for the States is expected to begin in late 1996.

Contacts at FHWA

Superpave Training and Field Assistance:
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Superpave Volumetric Mix Design

The Superpave mix design system is based on volumetric proportioning of the asphalt and aggregate materials and laboratory compaction of trial mixes using the **Superpave gyratory compactor**. All 50 States, as well as Puerto Rico and the District of Columbia, have received the Superpave gyratory compactor as part of the pooled-fund purchase.

The Superpave system also includes mix analysis procedures for predicting how well a mix will perform in the field. These procedures are intended for mixes that will be placed in pavements with very high traffic volumes and loads. Two new, sophisticated pieces of laboratory equipment—the Superpave shear tester and the indirect tensile tester—provide the data needed for the performance models.

A prototype of the **Superpave shear tester** is currently being evaluated at the TFHRC and by the five Superpave regional centers (Alabama, Indiana, Pennsylvania, Nevada, and Texas). Because of the high cost and complexity of the device, highway agencies and contractors have expressed interest in a simplified version that would perform only the shear test (no ancillary tests) and would not require a pressure chamber. Once the evaluation of the full-scale Superpave shear tester is complete, FHWA will look into developing a simplified, less costly version.

The first-article **indirect tensile tester** was delivered to the TFHRC in July 1995. It is now undergoing testing and evaluation.

Training Programs

Since 1993, the **Asphalt Institute** has, under contract with FHWA, offered Superpave training courses and technical assistance to State departments of transportation, paving contractors, asphalt suppliers, and others. The Institute's National Asphalt Training Center, located in Lexington, Kentucky, has held sixteen 1-week courses in **binder testing**, drawing 290 participants. The center has also taught fourteen 1-week courses in mix design to 275 engineers and technicians.

FHWA recently awarded the Asphalt Institute a contract for the second phase of Superpave training. Over the next 3 years, the National Asphalt Training Center will provide additional laboratory training in the areas of **mix design and pavement performance prediction**. The center will also work with the Superpave regional centers to provide local on-site training, technical assistance, and workshops.

Two **training manuals** developed for the courses, *Superpave Performance-Graded Asphalt Binder Specification and Testing* (Publication No. SP-1) and *Superpave Level 1 Mix Design* (Publication No. SP-2), are available from the Asphalt Institute.

For more information on training programs, contact the Asphalt Institute's **National Asphalt Training Center:**
Telephone: 606-288-4964
Fax: 606-288-4999

Mobile Asphalt Laboratories

FHWA now has two mobile asphalt laboratories. The laboratories are staffed with skilled technicians who provide assistance and training in Superpave **volumetric mix design and quality control/quality assurance** at construction sites across the country. The mobile laboratories are each equipped with a Superpave gyratory compactor and are used to demonstrate the principles of Superpave volumetric mix design.

This year, the labs have provided assistance at a dozen job sites, including an extended evaluation at FHWA's new test track, WesTrack.

Superpave Software

The Superpave software and performance models are currently being refined in response to evaluations by FHWA and its contractors, as well as a select group of field testers.

The **first version** of the software will be demonstrated at

FHWA's technology fair of SHRP products, which will be held in conjunction with the Transportation Research Board annual meeting in Washington, D.C., in January 1996.

FHWA has contracted with the University of Maryland to refine and manage the software, particularly the performance models.

Test Tracks

The Superpave system is currently being tested and validated through a variety of experimental projects. These include the new WesTrack facility, located at the Nevada Automotive Test Center. The track features 26 hot-mix asphalt pavement test sections. The performance of the various test sections will be evaluated against the Superpave performance prediction models.

FHWA is also collecting performance data, using two accelerated loading facility machines at the TFHRC, to validate the Superpave asphalt binder and mixture specifications.

Regional Coordination and Training

The asphalt user-producer groups continue to play a key role in developing and facilitating the implementation of the Superpave system. They have outlined a sensible, well-planned strategy for adopting the Superpave system on a regional basis.

Superpave centers have been established in each of the five asphalt user-producer group regions. The centers, operated jointly by universities and State departments of transportation, will conduct a thorough and coordinated shakedown of the procedures used with the Superpave shear test and indirect tensile test. They will also provide training on a regional basis.

New Logo Emphasizes Partnerships

To emphasize the partnerships involved in implementing the Superpave system, FHWA recently introduced a new Superpave logo. The logo shows the principal partners in the Superpave implementation program—namely, the American Association of State Highway and Transportation Officials, the highway industry, and FHWA. "Superpave 2000" signifies the target date for nationwide implementation of the Superpave mix design procedures.

Asphalt User-Producer Groups

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Rocky Mountain User-Producer Group
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Rick Holmgreen
Telephone: 713-544-8257
Fax: 713-544-8150

Concrete and Structures *UPDATE*

Showcase workshops, conducted on a regional basis, are one of the principal means of conveying information about the SHRP products for improving construction and maintenance practices for concrete pavements and structures. Each workshop features hands-on training and classroom learning on a group of related SHRP products. In some cases, technical assistance and loaner equipment are available to State highway agencies. After each workshop, participants from State highway agencies, industry, and FHWA meet to discuss how the technologies can be implemented on a regional basis.

Showcase workshops are available or planned in the following six topic areas:

- Alkali-Silica Reactivity (ASR)
- Concrete Durability
- Assessment of the Physical Condition of Reinforced Concrete Structures
- Methodologies for Reinforced Concrete Removal, Repair, Protection, and Rehabilitation
- Electrochemical Chloride Extraction
- High-Performance Concrete for Bridges and High-Performance Rigid Pavements

The pilot **concrete durability** showcase workshop was held June 27-28, 1995, in Arlington, Virginia. Presented by Construction Technologies Laboratories (CTL), the course introduced participants to a number of devices and procedures for evaluating the durability of concrete. The workshop covered five main topics:

- Permeability
- Freeze-thaw resistance
- Quality control
- Nondestructive testing
- Expert systems

Techniques discussed included the impact-echo method for measuring concrete thickness and locating defects, the microwave oven drying method for determining water content, and the hydraulic fracture test. FHWA will begin holding concrete durability workshops on a regional basis in April 1996.

Eight **ASR showcase workshops** were held in 1995. These

More than 40 products were developed under SHRP's concrete and structures program. These products can be classified under the broad categories of bridge condition assessment, bridge protection and rehabilitation, concrete durability, high-performance concrete, and alkali-silica reactivity. Many of these products can be applied to both pavements and bridges. The focus now is on refining the products and, through such means as showcase workshops, introducing them to the State highway departments and highway contractors.

> **Showcase Workshops**
>
> *Alkali-Silica Reactivity*
> The 3-day workshop features several SHRP products for detecting alkali-silica reactivity (ASR) in concrete in the field and in the laboratory. Includes hands-on training in identifying ASR. Target audience: materials engineers in highway departments and industry.
>
> Next workshop: Montreal, Quebec, April 16-18, 1996.
>
> Contact: Roger Surdahl, 202-366-1563 (fax: 202-366-9981; e-mail: rsurdahl @intergate.dot.gov).
>
> Concrete *Durability*
> Covers freeze-thaw durability, concrete permeability, and nondestructive testing of concrete. Target audience: materials and research engineers and technicians.
>
> Schedule: Workshops will commence in April 1996.
>
> Contact: Gary Crawford, 202-366-1286 (fax: 202-366-7909; e-mail: gcrawford @intergate.dot.gov).
>
> *Assessment of the Physical Condition of Reinforced Concrete Structures*
> Features corrosion detection devices, radar units, and rapid chloride test kits and emphasizes using these devices to evaluate bare and covered bridges. Target audience: bridge and construction engineers and technicians.
>
> Schedule: The pilot showcase is tentatively scheduled for March 1996.
>
> Contact: Donald Jackson, 202-366-6770 (fax: 202-366-7909; e-mail: djackson @intergate.dot.gov).

workshops are designed to give participants hands-on training in identifying and mitigating the effects of ASR-induced deterioration in portland cement concrete. The next workshop is scheduled for April 1996 in Montreal, Quebec.

Pilot workshops for the showcases on assessing the **physical condition of concrete structures** and **repairing, protecting, and rehabilitating concrete structures** will be held in spring 1996. The two showcases will run back-to-back during the same week, to make it possible for more engineers and technicians to attend.

Two ground-penetrating radar units for bridge deck evaluations have been ordered for use in both the workshops and field corrosion activities. The equipment is due to be delivered in the spring of 1996.

Three pilot **electrochemical chloride extraction** (ECE) projects have been installed: a bridge deck in Arlington, Virginia, and bridge columns and piers in Charlottesville, Virginia, and Sioux City, South Dakota. ECE is a promising technique for removing chloride ions from reinforced concrete structures, thus slowing deterioration. The pilot projects are designed to provide more information on the results of the ECE process, including how long a treatment can be expected to last and under what conditions ECE treatment is advised.

Open houses held at the pilot projects attracted a diverse group of attendees from State and Federal governments, private industry, and academia.

The pilot workshop on ECE was held in Arlington, Virginia, in July 1995. A field trip to the Arlington bridge project was included as part of the workshop.

Equipment Evaluations

Field evaluations of the **impact-echo device** are under way. The devices have been loaned to the highway departments in Wisconsin, New York, Iowa, California, South Dakota, Missouri, Virginia, Texas, Mississippi, West Virginia, New Jersey, Nevada, North Carolina, Massachusetts, and Pennsylvania, as well as the University of Washington and the University of Texas. In addition, Kansas, South Dakota, Indiana, and the University of Louisville have each purchased the equipment.

Initial evaluation reports of the device have been turned in by Missouri, Wisconsin, West Virginia, and Virginia. Users have reported difficulties in taking measurements and interpreting data with the device and have recommended additional research and development. The biggest problem they encountered was measuring the pavement thickness within the desired accuracy of ±5 mm; results to date have been in the range of ±13 mm. To address this problem, FHWA has begun testing a new production unit that allows users to measure pavement thickness more accurately (±4 mm).

Five small **hydraulic fracture test chambers** have been purchased for round-robin testing. The units have been sent to Kentucky, Iowa, Missouri, North Dakota, and Maryland.

Additional **air permeability test devices** have also been purchased, bringing the total available for loans to five. To date, the equipment has been loaned to Florida, New Jersey, Nevada, Arkansas, Missouri, the University of Nebraska, the Virginia Transportation Research Council, and South Dakota.

High-Performance Concrete

Officials from FHWA and the American Association of State Highway and Transportation Officials, together with representatives from private contractors and consulting agencies, recently toured the Northumberland Strait Crossing Project in Prince Edward Island, Canada. They met with Canadian officials and had an opportunity for a first-hand look at the bridge that is being built with high-performance concrete (HPC).

The first **HPC for bridges showcase workshop** will be held March 25-27, 1996, in Houston, Texas. It will cover the advantages and disadvantages of high-performance concrete, mix proportioning, structural design considerations, and evaluation of bridge component performance.

There are currently five **HPC bridge projects** being constructed in four States: Texas (2 bridges), Virginia, Nebraska, and New Hampshire. The projects are funded jointly by the Office of Technology Applications, the Office of Engineering R&D, the Office of Advanced Research, and the participating States. In addition, 10 States (California, Georgia, Iowa, Massachusetts, Minnesota, New York, Ohio, Pennsylvania, Texas, and Washington) have pooled a portion of their research funds to help finance two of the projects. projects. Seven more HPC for bridges projects have been proposed by Georgia, Colorado, Ohio, Washington, North Carolina, Nevada, and Indiana.

FHWA is making arrangements to host an **international HPC conference** in 1997.

Members of the expert task group (ETG) on **high-performance rigid pavements** (HPRP) held their first meeting in April 1995. As a result of their discussions, FHWA, through its regional offices, has invited State highway agencies to submit proposals for modifying or developing concrete paving projects to incorporate high-performance features.

Methodologies for Reinforced Concrete Removal, Repair, Protection, and Rehabilitation

The workshop features a variety of SHRP and non-SHRP products (including software, specifications, test procedures, and reference documents). Target audience: bridge and construction engineers and technicians.

Schedule: The pilot workshop is tentatively scheduled for March 1996.

Contact: Donald Jackson, 202-366-6770 (fax: 202-366-7909; e-mail: djackson @intergate.dot.gov).

Electrochemical Chloride Extraction

Demonstration projects in Delaware and Maryland will provide the basis for discussion in the workshops. Target audience: bridge and construction engineers and technicians.

Scheduled for Wilmington, Delaware, summer 1996, and Baltimore, Maryland, fall 1996.

Contact: Donald Jackson, 202-366-6770 (fax: 202-366-7909; e-mail: djackson @intergate.dot.gov).

High-Performance Concrete for Bridges

This workshop covers mix proportioning, structural design considerations, and advantages and disadvantages of high-performance concrete (HPC). Target audience: materials, bridge, and design engineers, as well as concrete inspectors.

Contact: Terry Halkyard, 202-366-6765 (fax: 202-366-7909; email: thalkyard@ intergate.dot.gov).

Highway Operations UPDATE

The SHRP highway operations program developed a wide range of test methods, design guides, and products addressing such areas as pavement repair, preventive pavement maintenance, snow and ice control, and work zone safety. Some of these products are undergoing further evaluation and refinement. Others, such as most of the work zone safety devices, have been readily adopted by State highway agencies.

Showcase workshops will be used to introduce many of these products on a regional basis.

Workshop Contacts

Snow and Ice Control
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Pavement Preventive Maintenance
Contact: Michael Smith, 202-366-4057 (fax: 202-366-9981; email: mrsmith@intergate.dot.gov).

Innovative Pavement Maintenance
Contact: Patrick Bauer, 202-366-1554 (fax: 202-366-998); email: pbauer @intergate.dot.gov).

Pavement Preventive Maintenance

More than 100 persons attended the May 1995 pilot **showcase workshop** on pavement preventive maintenance, held in Denver, Colorado. Designed for pavement, construction, and maintenance engineers, the workshop covered preventive treatments for both hot-mix asphalt and portland cement concrete pavements.

Regional workshops are tentatively scheduled to begin in early 1996. Workshop leaders will explain and demonstrate promising treatments that have been found to extend pavement service life. Test and evaluation plans for preventive maintenance treatments will be developed, and technical assistance will be provided to those State highway agencies participating in the evaluations.

Innovative Pavement Maintenance

The pilot showcase workshop on innovative pavement effectiveness was held in August 1995 in Washington, D.C. The workshop covered the **four maintenance areas** studied under SHRP:

- pothole repair in asphalt concrete pavements,
- crack sealing and filling in asphalt concrete pavements,
- spall repair in portland cement concrete pavements, and
- joint resealing in portland cement concrete pavements.

The workshop was divided into six sessions. The first two sessions were aimed at upper management and emphasized the importance of pavement maintenance to a sound pavement management strategy. The other sessions were geared for maintenance engineers and provided more detailed information.

Regional workshops are scheduled to begin early in 1996.

Snow and Ice Technology

FHWA recently wrapped up its 2-year **anti-icing test and evaluation project** (T&E Project 28). The study, which consisted of extensive field testing of various anti-icing technologies, culminated in a symposium in Estes Park, Colorado, in October

1995. The symposium drew more than 200 maintenance engineers and managers from State and local highway agencies, academia, consultants, suppliers, and manufacturers. The 15 State highway agencies that participated in the study reported the strategies they used and the benefits they gained. The contractor for the project, the U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL), summarized the overall findings and described the methodologies used in the study.

Based on data collected in the study, CRREL has developed a **guidance manual** for anti-icing operations under a variety of storm conditions. Highway agencies will be able to use the manual to develop their own localized anti-icing strategies. A draft of the manual was distributed at the Colorado symposium, and a final version is expected in early 1996.

Beginning in 1996, FHWA will conduct a series of 2-day **regional workshops** to showcase the snow and ice technologies. In addition to anti-icing strategies and technologies, the workshops will cover

- methods for evaluating chemical deicers,
- ice disbonding,
- road weather information systems,
- customized weather prediction,
- snow drift control,
- snowplow cutting edge,
- snowplow design, and
- snowplow scoop.

FHWA is currently seeking participants for five **test and evaluation projects:**

- Anti-icing—to evaluate how well spreader equipment distributes a finely graded salt prewetted with a liquid chemical.
- Road weather information systems—to determine the integration and interoperability between systems from different vendors and to establish a standard protocol.
- Road weather information systems—to test and evaluate snow and ice control management systems that are based on road weather information systems.
- Cutting edge—to evaluate a plow blade coated with a high cobalt grade of tungsten carbide to resist wear from shock.
- Plow design—to evaluate a plow that combines the SHRP-developed cutting edge, snowplow scoop, and moldboard design.

Snow and Ice Test and Evaluation Project

To solicit interest in the snow and ice test and evaluation projects, FHWA has distributed a brochure, *Better Snow and Ice Control Using State-of-the-Art Technologies: An Invitation to Test and Evaluation and Winter Workshops.*

For a copy of the brochure, which includes a response card to indicate interest, contact Salim Nassif at FHWA (telephone: 202-366-1557; fax: 202-366-9981; email: snassif@intergate dot.gov).

Work Zone Safety Devices

Since 1992, the SHRP work zone safety devices have been displayed at 41 major events, including such recent ones as the Texas Municipal League 1995 Convention and the 1995 annual meeting of the American Association of State Highway and Transportation Officials. Each FHWA region and most Local Technical Assistance Program (LTAP) centers have received a full set of the safety devices, allowing the devices to be shown at many regional and local events. To make it easier for the regions and LTAP centers to demonstrate the SHRP products to local and State highway agencies, FHWA has provided utility trailers that can easily store and transport the entire complement of work zone safety devices.

FHWA is encouraging highway agencies to try out the products in actual field applications. Technical assistance and funding support have been provided to participating States.

Work Zone Safety Brochure

Highway work zones are dangerous places. The need to perform critical road repairs often conflicts with the need to maintain traffic flow, leading to increased potential for work zone accidents. The SHRP work zone safety devices were designed to address these opposing needs.

The SHRP work zone safety devices are described and portrayed in an FHWA brochure, *Innovative Devices for Safer Work Zones*. The brochure covers the flashing stop/slow paddle, portable rumble strip, portable all-terrain sign and stand, direction indicator barricade, opposing traffic lane divider, intrusion alarm, remotely driven vehicle, portable crash cushion, truck-mounted attenuator for salt-spreaders, and queue detector.

The brochure also includes a listing of the SHRP work zone safety device contacts in each of the FHWA regions.

To request a copy of the brochure, contact Jacques Jenkins at 202-366-8025 (fax 202-366-7909; email: jjenkins@intergate.dot.gov).

Availability of Devices

Seven work zone safety devices are now commercially available.*

Five companies currently manufacture **intrusion alarms**. The Safety Line Infrared Alarm (ASTI Transportation Systems, Newark, Delaware) consists of an infrared transmission unit housed in a traffic cone; the alarm unit is housed in a second cone. It provides both longitudinal and transverse detection.

The Safety Sentinel Microwave Alarm (Traffic Management Systems Corporation, St. Louis, Missouri) is a two-unit system housed in plastic drums. Solar cells are mounted on top of the drums to recharge the batteries as needed. The system uses a microwave beam to provide longitudinal detection. It also includes a drone radar transmitter that sets off radar detectors in vehicles within 600 meters of the unit, helping to slow approaching traffic.

The Model 10 two-unit intrusion alarm (Safe Lite System, Newtown, Pennsylvania) runs on rechargeable batteries and uses a radio communications linkage between the units. A pneumatic tube laid on the pavement is used to detect intruding vehicles and provides transverse detection at the lane closure.

The intrusion alarm manufactured by the Columbia Safety Sign Company (Woodland, Washington) also uses a pneumatic tube to detect intruding vehicles.

The Watchdog (Kenco International, Ligonier, Pennsylvania) consists of a series of pneumatic hoses hard-wired to the alarm unit.

*The U.S. Government does not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

Impact Recovery Systems (San Antonio, Texas), Flexstake, Inc. (Ft. Meyers, Florida), and Flasher Handling Corporation (Depew, New York) currently manufacture the **opposing traffic lane divider.** All three products feature a similar two-arrow face design, with the main difference between the three being the support systems for returning the divider to an upright position when hit.

Three companies currently manufacture devices that meet the basic criteria for SHRP's **direction indicator barricade.** The product from WLI Industries, Inc., (Villa Park, Illinois) has a horizontal arrow on a type II barricade, while Flasher Handling Corporation (Depew, New York) and Carsonite, Inc. (Carson City, Nevada) place the sign panels on a support with a weighted base. The device's primary objective is to provide guidance during lane closures. Currently, the barricade is still considered experimental and thus requires permission from FHWA for use.

Poly Enterprise (Monrovia, California) has produced a molded version of the **portable rumble strip** using virgin and recycled plastic in place of the neoprene laminated version developed by SHRP. The rumble strip works best under low speed traffic conditions; under high traffic speeds or heavy truck volume, the strip is subject to rotation and movement.

The original SHRP-designed **flashing stop/slow paddle** is currently being produced by a Canadian firm, Detronics, and distributed by Graham-Migletz, Inc. (Independence, Missouri). In addition, Columbia Safety Sign Corporation (Woodland, Washington), Action West (Kelso, Washington), A/C Enterprise (Vancouver, Washington), Medifax, Inc. (La Center, Washington), and Brittney Safety Sign (Copper Country Safety Sales, Phoenix, Arizona) are each manufacturing a paddle that is based on the SHRP concept but that uses strobe lights or bulbs rather than high-intensity halogen bulbs.

Napoleon Fabricators, Inc. (Napoleon, Ohio) and AdraCorp. (Huntsville, Alabama) both manufacture the **portable all-terrain sign and stand.** AdraCorp's product is a tripod version that weighs just over 3 kilograms (7 pounds).

The **queue detector**, which consists of a transmitter, receiver, and electronics module, is available from ASTI Transportation Systems (New Castle, Delaware). The detector alerts drivers to stopped or slow traffic ahead, giving them more time to react and prevent accidents.

Still Under Development

The portable crash cushion is currently being modified so that it uses a small trailer for more maneuverability in loading and unloading. Three trailer units are currently being manufactured for testing and evaluation by State highway agencies.

MUTCD Approval

The revised Part VI of the *Manual on Uniform Traffic Control Devices* includes three of SHRP's work zone safety devices.

Section 6E-4 includes a discussion of stop/slow paddles and approves the use of SHRP's flashing stop/slow paddle.

SHRP's portable rumble strip meets the specifications set forth in Section 6F-8d, which describes allowable types of rumble strips and their proper application.

Section 6F-8g covers opposing traffic lane dividers, which are used as center lane dividers to separate opposing traffic on a two-lane-two-way operation.

Long-Term Pavement Performance UPDATE

Designed to give States the information and products they need to build and maintain longer lasting pavements, the 20-year long-term pavement performance (LTPP) program is almost at its midpoint. The program is, however, already delivering products, such as the modified Georgia faultmeter and the falling weight deflectometer calibration procedures.

Some of the products now available relate to materials testing, pavement performance monitoring, and equipment standards and calibration procedures. Still under development are products directed at the selection and effectiveness of maintenance strategies, performance of various rehabilitation techniques and materials, and the selection of design features for new construction or total reconstruction.

Strategic Plan Published

In September 1995, the LTPP program published *The Long-Term Pavement Performance Roadmap: A Strategic Plan*. The plan was developed with input from State and provincial highway agencies, the American Association of State Highway and Transportation Officials (AASHTO), the Transportation Research Board (TRB), industry, academia, and FHWA.

The *Roadmap* contains a data analysis plan for developing LTPP products, and it identifies critical issues facing the LTPP program. The *Roadmap* also provides a brief history of the LTPP program, its partners, and their roles. It charts a course to the program's near-term and longer term destinations.

The *Roadmap* is being widely distributed to help inform the highway community about the projects and products of the LTPP program. AASHTO has sent copies of the *Roadmap* to each State.

Just as the LTPP program is a dynamic process, so too is the *Roadmap*; the report will be updated periodically to reflect changing needs and priorities.

A new pocket-sized brochure describing the LTPP program was published by FHWA in October 1995. The brochure, titled *Improving Pavement Technology: A 20-Year Journey*, consists of a series of commonly asked questions and answers about the LTPP program.

National Conference To Be Held in March

To provide an update on the LTPP program's accomplishments and the products being developed by the program, FHWA will convene a conference in Irvine, California, in March 1996. The conference, "Improving Pavements with LTPP: Products for Today and Tomorrow," will be held March 26-28 at the Arnold & Mabel Beckman Center of the National Academies of Science and Engineering.

The conference will focus upon LTPP products that contribute to increased pavement life; early products available from the LTPP program, and the path to developing additional anticipated products.

The conference is intended primarily for State, Federal, and industry engineers and managers with responsibilities for delivering pavement programs. The conference will also be of interest to engineers involved in the conduct of the LTPP studies or other pavement research programs.

The conference is cosponsored by:

- American Association of State Highway and Transportation Officials
- American Concrete Pavement Association
- American Trucking Associations
- Canadian Strategic Highway Research Program
- National Asphalt Pavement Association
- National Stone Association
- Transportation Research Board

LTPP Product Preview

In January 1996, FHWA will distribute a new brochure containing a list of the available and planned LTPP products. The *LTPP Product Preview* will include a description of each product, its status, and a name of the person to contact for more information.

Products will be grouped in four categories: materials testing, design guidelines, pavement monitoring procedures, and equipment standards and calibration. The Product Preview will be used to develop implementation plans for the products. Engineers and managers who desire to be among the earlier users of the products will also find the brochure helpful.

SPS-3/4 1995 Field Evaluations Completed

Expert teams of engineers from State highway agencies, industry, and FHWA have completed their evaluations of the performance of various preventive maintenance treatments constructed in 1990 as part of SHRP. Regional teams conducted on-site field evaluations of the specific pavement studies (SPS) experiments (flexible pavements, SPS-3, and rigid pavements, SPS-4) during August, September, and October 1995. Each field review was 6 to 10 days in length. More than 81 experimental sites and 405 test sections were visited.

The review teams' subjective evaluations will be used to complement the LTPP data analysis now under way on the 5 years of performance data collected at the sites. The objective of this analysis effort is the formulation of sound conclusions

Publication Requests

To request a copy of

- *The Long-Term Pavement Performance Roadmap: A Strategic Plan* (FHWA-RD-95-200)
- *LTPP Product Preview*
- LTPP Conference Brochure

contact the Pavement Performance Division, Office of Engineering R&D, at 703-285-2355 (fax: 703-285-2767).

and recommendations on the performance and use of these preventive maintenance treatments—that is, what works, and what doesn't. A national summary report detailing the observations, conclusions, and recommendations of the review teams is being developed by an FHWA contractor, Nichols Consulting Engineers, and should be available in early 1996. A final report on the entire SPS 3&4 project is also being prepared. Technology transfer materials and manuals of practice will be developed to assist highway agencies in implementing the study findings.

Monitored Traffic Data Now Included in National Information Management System

The LTPP National Information Management System now includes actual traffic data collected at monitored general pavement studies (GPS) sites. State and provincial highway agencies have been collecting the data since 1990, but access to the data was delayed until standardized processing procedures could be developed.

The newly available traffic data covers the 1990-1993 period and contains information on

- traffic and truck volumes,
- weight distributions of axle groups by vehicle type, and
- equivalent single-axle load estimates.

The information is based on vehicle counts collected at more than 470 GPS sites and vehicle weights measured at nearly 400 GPS sites in 48 States and provinces.

LTPP Activities at the 1996 TRB Annual Meeting

The LTPP program will be very visible at the 1996 Transportation Research Board Annual Meeting in Washington, D.C. The activities start on January 6 with the Data Analysis Working Group meeting. At the SHRP Coordinators meeting on January 7, highlights of the LTPP program will be presented in the plenary session. An LTPP exhibit will be set up at the technology fair that follows the coordinators meeting.

The international LTPP coordinators will meet on January 7. Participants will share the status of their LTPP activities and explore opportunities for further cooperative efforts.

On January 8, Session 42 will feature a series of presentations on the *Roadmap*, related AASHTO activities, and LTPP products.

Data Sampler Software

To obtain a copy of the Data Sampler and Data Request software program, contact Barbara Ostrom at 703-285-2514 (fax: 703-285-2767; email: bkostrom@intergate.dot.gov).

The program is furnished on a single 90-mm (3.5-inch) disk. It requires a computer running under Windows version 3.0 or higher, 2 megabytes of hard disk space, and 4 megabytes of RAM.

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U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: SHRP Information Clearinghouse

Date: July 22, 1994

From: Associate Administrator for Safety and System Applications
Washington, D.C. 20590

Reply to Attn of: HTA-3

To: Regional Administrators

One of the challenges in conducting the SHRP implementation program is communication, within FHWA, and with all of our partners regarding the structure and status of the program, and about the numerous opportunities to participate. One communication tool is the FHWA's SHRP Product Implementation Status Report. Prepared quarterly, the Status Report captures the highlights of the SHRP implementation program. Attached is the June issue. To date, the FHWA has utilized its traditional communication mechanisms supplemented by extensive use of E-mail directly to the FHWA SHRP coordinators in the regions and divisions. The Status Report is one example of the information that is distributed via E-mail to our field offices. National and regional meetings have also been used to tell the story. The FHWA also publishes the SHRP FOCUS monthly newsletter which is sent to 8,500 individuals nationally and internationally.

One of the recommendations which the FHWA received regarding SHRP implementation communication was to establish a computer based information system. One that would allow any interested party to learn what is planned, who is doing it, and when it will happen. The SHRP Information Clearinghouse contains: (1) Status Report, (2) Product Information, (3) Calendar, (4) Directories, and (5) SHRP Report Abstracts.

The Clearinghouse, which is operated by the Office of Technology Applications is currently accessible to all users via a modem and an 800 telephone line. The only requirement for operation of the system is that a user execute a series of computer commands on his or her initial entry. These instructions have been E-mailed directly to the region and division SHRP Coordinators. We are currently exploring options to access the Clearinghouse on the FHWA WAN and AASHTO VAN.

As the principal potential users of the SHRP products, the State highway agencies need to be introduced to the Clearinghouse and provided the computer instructions. To strengthen the SHRP implementation partnership, we are requesting that the division offices inform the State highway agencies about the Clearinghouse. To assist the divisions, attached are:

A suggested letter from the division office to the State introducing the Clearinghouse - please modify the letter to suit local conditions,

Sufficient quantity to provide two computer diskettes to each State, and

An information page describing the Clearinghouse.

The letter to the State should also go to the Local Technical Assistance Program (LTAP) Technology Transfer Centers in each State and in Puerto Rico. A limited number of SHRP products are of interest to small and local governments. The FHWA is funding a contract to promote SHRP products to local governments through the LTAP technology transfer centers. Information on the implementation efforts for local governments is also contained in the Clearinghouse data bases and each center is being sent directly a copy of the diskette. A separate distribution will be made to the four technology transfer centers for American Indian tribal governments.

Industry, national associations and trade publications, academia, and international users will be informed about the Clearinghouse through magazine articles in FOCUS, PUBLIC ROADS, other magazines, and general advertisements. Please feel free to inform regional and local industry and publications regarding the availability and access to the Clearinghouse.

The regions, divisions, and States have all cooperated enthusiastically and significant progress has been made toward the adoption of the SHRP products. However, a lot remains to be accomplished and your continued support and participation is critical to the overall success of the implementation effort. The Headquarters SHRP implementation team is available to assist you. Please do not hesitate to contact any of the individuals identified in the Status Report for assistance.

Dennis C. Judycki

Dennis C. Judycki

Attachments

U.S. Department of Transportation
Federal Highway Administration

SHRP Information Clearinghouse

shrp

As part of its SHRP implementation program, FHWA has initiated numerous activities, including workshops, exhibits, technical assistance, and test and evaluation projects. Keeping track of all that information is a formidable task.

FHWA created the SHRP Information Clearinghouse to make it easier for State departments of transportation, industry, academia, the international community, and others to check the status of the SHRP products and to get information about FHWA's implementation activities.

The Clearinghouse is actually a set of five databases, housed in an IBM-compatible computer. A customized software program links the databases and provides a graphical user interface. FHWA regularly reviews and updates the data.

The Clearinghouse includes:

- The full text of the most recent version of FHWA's SHRP Implementation Status Report

- Product Information
  - -Historical and current information
  - -Information on the showcase workshops and contracts
  - -Information on the States participating in test and evaluation projects for SHRP products

- Calendar of SHRP-related exhibits, workshops, training programs, and meetings

- A directory of FHWA contractors, technical working group and expert task group members, technical assistance sources, SHRP coordinators, and others involved in SHRP implementation activities

- Abstracts of all SHRP reports - as well as information on ordering the reports

The Clearinghouse runs in a user-friendly Windows environment. It is easy to navigate; the user selects from a series of menus. There are no special computer hardware or software requirements, but a mouse is recommended.

The SHRP Information Clearinghouse became operational in July 1994. You can reach the Clearinghouse through FHWA's local-area network or by using a high-speed (9600 baud or faster) modem to dial directly into the host computer. To request a copy of the self-installing software (which you will need to dial in to the Clearinghouse), contact Tonya Inc. at 202-289-8108. For more information about the SHRP Information Clearinghouse, contact FHWA's Office of Technology Applications (fax 202-366-7909).

U.S. Department of Transportation

Federal Highway Administration

Memorandum

Subject: Implementation Plan for the Strategic Highway Research Program (SHRP) Products

Date: June 3, 1993

From: Executive Director

Reply to Attn of: HTA-3

To: Associate Administrators
Regional Administrators
Federal Lands Highway Program Administrator

The Federal Highway Administration (FHWA) continues to put a priority on the implementation program for the SHRP products. Most recently, the attached plan on SHRP products implementation was developed under the direction of the FHWA SHRP Implementation Coordination Group (SICG). The plan describes the overall approach, the partnerships that are considered essential to the successful implementation of the SHRP products and the roles of the involved organizations, including our field offices. Also, attached is a companion document that lists the organizational memberships of the various committees and task forces associated with this program.

The plan was developed with the understanding that it is a living document that would grow and change in response to the needs of the users of the SHRP products. It provides the framework by which the specific individual product(s) implementation plans, both national, regional and State, will be developed. To be successful, the specific product implementation plans must be tailored to meet regional and State conditions. It is strongly recommended that the regions and divisions be active participants with the States and industry in the development of these implementation plans.

During the coming months, FHWA will continue to put in place the SHRP products implementation mechanisms and activities such as the four technical working groups, the development of specific national plans and the showcase contracts referred to in the plan. However, within the framework described in the plan you are encouraged to begin planning the development of regional strategies and possible organizational structures that include our partners. I strongly encourage you to become actively involved in this process and in the subsequent implementation activities.

The Office of Technology Applications (OTA) is available to provide additional information regarding the SHRP implementation plan and to assist your staff in the development of regional and State plans. During this summer and fall visits by OTA staff to your Region, meetings will be held to discuss the program with your staff.

E. Dean Carlson

E. Dean Carlson

Attachments

Federal Highway Administration
HTA-3:CChurilla:ljp:366-6626:5/26/93
cc: HOA-1
HOA-2
HOA-3
HOAES
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HTA-1
HTA-3 Official File

US Department of Transportation
Federal Highway Administration

Memorandum

Subject SHRP Products Implementation

Date November 22, 1993

From Executive Director

Reply to Attn of HTA-3

To Associate Administrators
Regional Administrators
Federal Lands Highway Program Administrator

The Federal Highway Administration (FHWA) has made significant progress in the SHRP implementation activities at the national level. The four Technical Working Groups (TWGs) have been formed and are addressing the development of product-specific implementation plans, contracts for various SHRP implementation support functions are in place, and the first of the showcase contracts has been awarded. Attached for your information is the SHRP Implementation Status Report that describes the FHWA activities. This report is routinely distributed on E-mail to the region and division office SHRP coordinators.

One of the SHRP support activities is a Speakers Bureau that provides FHWA a mechanism to respond to the many requests for presentations on SHRP products. When FHWA staff is unable to respond to a request for a SHRP presentation, the Speakers Bureau can provide a knowledgeable individual from the private sector. The FHWA also has other means available when we wish to utilize an individual from a State highway agency as a SHRP products speaker. Please contact Charlie Churilla (202-366-6626) in the Office of Technology Applications if we can help in this regard.

One of the field office SHRP implementation activities that is extremely important is working with the State highway agencies to establish or foster the operation of SHRP implementation activities. A number of States have established SHRP implementation committees as a means to coordinate the evaluation and adoption of SHRP products. In those States that have such a committee, the region and division offices can play valuable roles as an information source on the products and a champion for the many implementation activities being offered by FHWA. I am requesting that you encourage the Division Administrators to discuss SHRP implementation with their State counterparts. In those instances where an implementation process does not exist, the importance of taking action now should be stressed.

Federal Highway Administration 1893 1993

In the many instances where such a committee or process already exists, the discussion should focus on the strengthening of the State-FHWA implementation partnership. To assist you in this effort, attached are copies of a SHRP Implementation videotape prepared by FHWA.

During the life of the SHRP, an annual State Coordinators' meeting was held in conjunction with the Transportation Research Board (TRB) Annual Meeting. The SHRP meeting is being continued by FHWA, with the support of TRB, and will focus on the implementation activities and the continuation of the Long Term Pavement Performance program. In the past, this meeting has been extremely well attended with representatives from 70+ percent of the States. Attendance by a regional office representative, and at your discretion from one of your division offices, is recommended. Washington Office Directed Travel has been approved for the SHRP Coordinators' meeting.

Also, during the fall, representatives from the Headquarters offices involved in the SHRP implementation efforts have visited most of the regional offices to provide firsthand information on the SHRP implementation activities and to discuss the region and division offices' roles in these activities. One of the items specifically addressed during several of these meetings was the funding for the SHRP implementation activities at the regional and State levels. As the national implementation plans are developed by the TWGs, each region will have the opportunity to develop regional plans for specific products or showcase group of products. Activities in the regional plans may include test and evaluations, regional equipment purchases, and associated administrative costs for the regional technical committees. The Office of Technology Applications is available to assist your office in the development of these regional plans and to provide the funding for these field-led implementation activities. Detailed information regarding the funding of the regional plans will be forthcoming.

For the SHRP implementation to be a success, it requires the active participation by all the partners. At the national level, TRB, AASHTO, and FHWA have taken a number of significant steps towards this goal. However, to ultimately reach the goal, the States in cooperation with the FHWA field offices and local industry must act. I, again, want to strongly encourage you and your staff to continue to be active participants in the implementation process.

E. Dean Carlson

E. Dean Carlson

2 Attachments

Technical Report Documentation Page

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16. Abstract

The Intermodal Surface Transportation Efficiency Act of 1991 fully supported the implementation of research results from the $150 million Strategic Highway Research Program (SHRP). Successful implementation of SHRP by and large will be measured by successful implementation of the asphalt research.

In a unique cooperative spirit, the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), and the SHRP Project Management Office have worked together to develop a plan that ensures the research will indeed be implemented. This paper describes important aspects of this partnership and focuses on key elements of the plan. These elements include:

- The large scale procurement and evaluation of new equipment.
- Integration of equipment procurement with national training agenda.
- Use of mobile laboratory support.
- Integrated activities with standards setting functions of AASHTO.
- Integrated use of other funding sources for followup research and implementation - National Cooperative Highway Research Program (NCHRP), FHWA Administrative Funds, and Federal-aid Planning and Research Funds.
- The unique role of users-producer groups and technical working groups that represent public and private interests.

Finally, the paper discusses the very critical function of Specific Pavement Study 9 (SPS-9), Validation of the SHRP Superpave™ and Innovations in Asphalt Pavements, in the continuing refinement of the Superpave™ performance models and design methods.

17. Key Words	18. Distribution Statement
Strategic Highway Research Program, asphalt concrete pavement, asphalt, asphalt binder, asphalt mixture	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

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Unclassified	Unclassified	17	

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US Department of Transportation

Federal Highway Administration

Memorandum

Subject: INFORMATION: Distribution of Publication

Date: March 23, 1994

From: Associate Administrator for Safety and System Applications

Reply to Attn of: HTA-13

To: Regional Administrators
Federal Lands Highway Program Administrator

Distributed with this memorandum is Federal Highway Administration Technology Applications Program, April 1994, Publication No. FHWA-SA-94-028, an update of the July 1993 publication (FHWA-SA-93-075). This provides a current listing of all technology transfer projects and an up-to-date status on the activities within the project. The Office of Technology Applications (OTA) will continue to update and distribute this publication periodically in order to keep the field offices, States, and Technology Transfer Centers up to date on the technology transfer activities underway.

Sufficient copies of this publication are being distributed to provide 6 copies to each regional office and 10 to each division office. Direct distribution is being made to the division offices; copies for State highway agencies are included with the copies for the division offices. Two copies are also being sent to each Local Technical Assistance Program Technology Transfer Center.

A limited number of additional copies are available from the FHWA Research and Technology Report Center, HRD-11, Room A-200, 6300 Georgetown Pike, McLean, Virginia 22101-2296.

Dennis C. Judycki

Dennis C. Judycki

Attachment

DP-75 Mobile Concrete Laboratory (SHRP)

DESCRIPTION : The project's goals include demonstration of state-of-the-art concrete technology in materials selection, mix designs, laboratory testing, and field testing. Project activities include guidance for updating specifications and use of computer technology for design, testing, and data storage. A partnership with manufacturers, contractors, industry associations, and academia is maintained in all of the project's activities.

This project demonstrates the use of innovative laboratory and in situ testing equipment, and promotes high-performance concrete and the use of chemical admixtures. This project also supports the activities of SP-201, "Accelerated Rigid Paving Techniques."

BACKGROUND : With today's construction heavily involved in rehabilitation and reconstruction, highway engineers place ever greater demands on Portland cement concrete. These demands include lower permeability, higher and earlier strength, and improved workability. Many concrete admixtures are available today that specifically address these demands. However, to understand and effectively use these admixtures, innovative mix designs, testing equipment, and techniques are a prerequisite.

With the use of a mobile concrete laboratory, 26 field demonstrations have been performed in the last 5 years. Two-day workshops on state-of-the-art concrete technology have been conducted in 44 States. Twenty 1-day seminars on "Concrete Admixtures" have been conducted. Many presentations, including the mobile concrete laboratory, have been given at national, regional, and local FHWA and industry meetings. More than 2,500 State DOT and FHWA engineers have attended workshops, seminars, and field demonstrations. Under the equipment loan program, in situ testing equipment has been loaned to 20 States.

PROJECT MANAGERS : Suneel Vanikar, HTA-21, (202) 366-0120 and Gary Crawford, HTA-21, (202) 366-1286

STATUS : In 1995, mobile laboratory field demonstrations were conducted in Texas, Ohio, and Virginia. One-day nondestructive testing (NDT) workshops were held in Missouri and Iowa. This NDT workshop will be presented in several States over the next few years. This workshop includes some SHRP-developed products. A Concrete admixtures seminar was presented in Hawaii.

The remaining States will be visited over the next several years, with many States asking for repeat visits as the SHRP-developed products are included in the laboratory. The 1-day admixture seminars will continue for a few more years. Additionally, this mobile laboratory will support efforts related to implementing SHRP-developed concrete technology. The major emphasis for the next several years will be on field demonstrations of the SHRP-developed products and implementation of Performance Related Specification for Concrete Pavements.

TECHNOLOGY TRANSFER AIDS : Mobile laboratory, telephone and on-site assistance, speakers, specialized workshops and seminars, and nondestructive equipment loan program. A new mobile concrete laboratory was acquired in 1995.

PUBLICATIONS : FHWA reports on several field studies available through the Office of Technology Applications.

DP-84 Corrosion Survey Techniques

DESCRIPTION : The objective of this project is to demonstrate and document the latest concepts and test procedures for corrosion surveys on reinforced concrete structures. A secondary objective is to work in conjunction with States to collect data on structures that already have protective systems and to determine their effectiveness. The project is divided into three distinct modules:

- Executive Presentation Slide presentation and some equipment demonstration.

- Equipment Demonstration Slide presentation on bridge evaluation techniques and 1- to 2-day equipment demonstrations.

- Hands-on Training and Testing. Three to four days of hands-on experience with equipment.

- A loan program for States that are interested in a particular piece of equipment.

Several products developed under the Strategic Highway Research Program (SHRP) are being demonstrated as part of this project.

BACKGROUND : Deterioration of reinforced concrete by corrosion of the reinforcing steel is the most frequent cause for needing maintenance, rehabilitation, or replacement of concrete structural elements. The ability to identify an active corrosion process in the early stages is the most important factor in minimizing the cost of corrosion-related repairs.

Today's equipment is lighter, stronger, more durable, and is capable of interfacing with microcomputers through CADD-like software. Additionally, with the growing attention paid to concrete substructure corrosion, this equipment solves some of the difficulties of surveying vertical surfaces over rivers, coastal waters, and freeways. Some tests that will be performed are half-cell potential survey, delamination mapping, rapid field measuring, chloride content, concrete cover survey, rebar corrosion rates, and crack measurement.

PROJECT MANAGER : Donald Jackson, HTA-22 (202) 366-6770

STATUS : This project was announced late in 1991. DP-84 has been presented 36 times since then. Interested States may request demonstrations from the project manager.

DP-87 Drainable Pavements

DESCRIPTION : This project was developed to help State highway agencies and industry partners become more familiar with new techniques in permeable base and edgedrain system design and construction. This project concentrates on the use of permeable bases with concrete pavements and consists of a workshop that features a slide presentation, design manual, and field construction technical assistance. It also incorporates a hydraulic demonstration model that presents the drainage rate of various aggregate materials used in road building, including permeable bases.

BACKGROUND : Water in the pavement section is recognized as a major factor in pavement deterioration and early loss of pavement service life. In recent years, highway engineers have recognized the cost benefits of providing permeable bases to drain the pavement section. New aggregate gradations and stabilizing materials for base courses have been used to provide a balance between drainability and stability. Construction engineers also have developed new techniques for placing and compacting permeable base material.

PROJECT MANAGER : Robert Baumgardner, HNG-42, (202) 366-4612

STATUS : More than 40 workshops have been completed to date. Scheduled presentations concluded in March 1994. The scope of the workshop portion of this project will be expanded in a future NHI course to include retrofit edgedrains and drainage of flexible pavement. (See DP-87 Phase II, page under Asphalt Pavement Design and Construction.)

TECHNOLOGY TRANSFER AIDS : Workshop available on request (subject to long-range planning). specifications from Wisconsin, technical assistance, construction evaluation monies (limited), computer software available from PCTrans, University of Kansas, and McTrans, University of Florida.

DP-87 Drainable Pavement Systems (Phase II)

DESCRIPTION : This project was developed to help State highway agencies and industry partners become more familiar with new techniques in permeable base and edgedrain system design and construction for concrete pavements. This phase of the project will concentrate on the use of permeable bases with asphalt pavements and, as with concrete pavements under Phase I, consists of a workshop that features a slide presentation, design manual, and field construction technical assistance.

BACKGROUND : Water in the pavement section is recognized as a major factor in pavement deterioration and early loss of pavement service life. In recent years, highway engineers have recognized the cost benefits of providing permeable bases to drain the pavement section. New aggregate gradations and stabilizing materials for base courses have been used to provide a balance between drainability and stability. Construction engineers also have developed new techniques for placing and compacting permeable base material.

PROJECT MANAGER : Robert Baumgardner, HNG-42, (202) 366-4612

STATUS : This project is being expanded in an NHI course to include retrofit edgedrains and drainage of flexible pavement. In addition, a contract has been awarded to Applied Research Associates to develop a microcomputer program to calculate pavement subsurface drainage.

TECHNOLOGY TRANSFER AIDS : Workshop available on request (subject to long-range planning), equipment demonstration.

DP-89 Quality Management

DESCRIPTION : The goal of this project is to build top-level support and awareness of quality management and to provide training to State highway agencies in statistical quality control techniques. It is part of the National Quality Initiative. This project involves four quality management activities.

- Participate on a joint FHWA/AASHTO/industry steering committee to guide and help focus efforts on the quality of construction, performance, and quality management with emphasis on a partnership effort.

- Develop (jointly) and issue broadly based national policy/goals.

- Hold high level seminars for upper management of Federal, State, industry, and others to educate and gain support.

- Provide technical training, guidance, and tools to others responsible for implementation.

BACKGROUND : There has been a conscious effort within the United States during the past decade to promote a correlation between American products and quality. In general, this effort has been focused in the manufacturing industry. The United States has begun to promote the concept of American quality because quality is an important factor in maintaining global competitiveness.

With the emphasis on quality again moving toward national significance, this project will provide direction and address a broader role of quality in the highway environment.

PROJECT MANAGER : Don Tuggle, HNG-21, (202) 366-1553

PROJECT COORDINATOR : Gary Henderson, HTA-22, (202) 366-1283

STATUS : In an effort to widely disseminate the principles and ideals begun at the National Quality Initiative Seminar in Dallas/Ft. Worth, Texas on November 10, 1992, four AASHTO Regional NQI Seminars involving well over 10,000 people nationwide have been conducted. Additional support of state-level NQI activities has been provided.

An "NQI National Conference" will be held in Alexandria, VA on November 14 and 15, 1995. The first-ever NQI Achievement Award will be presented for the best highway project at this conference.

A 5-day training course (*Materials Control and Acceptance: Quality Assurance*) and a 2-day workshop (*Quality Management for Managers*) is being co-sponsored with the National Highway Institute. Approximately 38 of the 50 available five-day courses and 41 of the 56 available two-day workshops have been presented. Several statistical quality assurance computer programs have been developed by the New Jersey DOT. A technical review of the user manual has been completed, and distribution of the manuals and programs is expected by the end of 1995. In addition a number of workshops and seminars have been supported such as a technician training and certification workshop in Platteville, Wisconsin and a quality assurance specifications development workshop in Little Rock, Arkansas.

TECHNOLOGY TRANSFER AIDS : One-week course, two-day workshops, technical assistance, speakers, and computer programs.

DP-90 Mobile Asphalt Laboratories

DESCRIPTION : This project is a major Office of Technology Applications initiative to promote Strategic Highway Research Program (SHRP) findings in the asphalt area. This project uses two mobile laboratories to provide State highway agencies with a hands-on demonstration of the SHRP SUPERPAVE design system and field management techniques.

The major objective of the project is to promote the Super Pave Mix design system and mix verification / volumetric quality control in the field.

The typical project centers on transplanting a mobile lab to an active paving project at the invitation of the State. Once it is on site, State, contractor, and Federal engineers can witness, compare, and critique the test procedures and sequences.

PROJECT MANAGERS : Thomas Harman, HTA-21, (202) 366-0859; John D'Angelo, HTA-21, (202) 366-0121; and John Bukowski, HTA-21, (202) 366-1287.

STATUS : The use of mobile laboratories for asphalt mix is ongoing. The concepts of Mix Verification and Voids Acceptance have been demonstrated and field simulated in more than 38 States in the last 8 years. As an additional service, more than 50 Federal and State contractors, engineers, and technicians have spent 2 to 5 days in a mobile laboratory learning and strengthening their skills in the asphalt mix area. In 1991, a formal 2-day workshop was added to the demonstration. In 1993, key elements of the SHRP SUPERPAVE mix design system were also added to the workshop. During 1994 and 1995 the laboratory provided field control on several projects using SUPERPAVE designed mixes.

A report detailing the results of the field simulation was voted the "Best Paper of the Year 1991" by the Association of Asphalt Paving Technologists. This report, *Summary of Simulation Studies*, is available from the project managers.

The remaining States will be visited over the next several years. With the addition of the SUPERPAVE system, many States are expected to request repeat visits as they explore the adoption of the new techniques. The mobile laboratory has supported other OTA activities, such as stone matrix asphalt (SMA), and is expected to perform this support activity more frequently in the next few years.

TECHNOLOGY TRANSFER AIDS : Mobile laboratory (subject to scheduling), telephone and on-site assistance, speakers, and specialized workshops and seminars.

PUBLICATIONS : *Summary of Simulation Studies*, by J. D'Angelo and T. Ferragut, 1991.

DP-108 Pavement Management Analysis

PURPOSE : To demonstrate how various PMS prioritization methods are used to identify justifiable and cost-effective pavement preservation strategies for various funding levels and develop multi-year prioritized list of pavement preservation projects.

To demonstrate how PMS pavement performance data is used to perform engineering analyses that could evaluate pavement design, construction, materials and maintenance procedures as they relate to performance of pavements.

BACKGROUND : The ISTEA Interim Final Rule for management systems requires each State Highway Agency to develop a PMS for the National Highway System capable of performing various pavement analyses.

These analyses included pavement performance analysis to analyze the current and predicted performance of specific pavement types, investment analyses to estimate total cost for present and projected conditions across the network, and investment strategies to prioritized pavement preservation projects with recommended preservation treatments that span single and multi-year periods using life-cycle cost analysis.

The regulation also requires the PMS to be capable of performing engineering analyses for appropriate network sections that could evaluate pavement design, construction, rehabilitation, materials, mix designs, and preventive maintenance as they relate to performance of pavements.

State examples of pavement performance, multi-year prioritization methods, cost analyses and engineering analyses will be used to develop two to three-day demonstration sessions. The project consists of two demonstration activities.

- The first activity consists of a series of PMS outreach sessions to provide one-on-one discussions and technical assistance to States that are developing the analyses required to perform **multi-year prioritization of pavement preservation projects.**

- The second project consists a demonstration of the use of **PMS performance data in engineering applications.**

The main topics to be demonstrated in the multi-year prioritization demonstration activity are:

- Pavement Performance Analysis
- Selection of Pavement Preservation Strategies and Treatments
- Cost Analyses
- Effects of Budget Constraints
- Project Selection Process

The main topics to be demonstrated in the use of PMS performance data in engineering applications demonstration activity are:

- Historical Performance Data

- Evaluation of Pavement Design Procedure
- Evaluation of Pavement Construction Practices
- Materials Performance Analysis
- Pavement Preservation Analysis

PROJECT MANAGER : Luis Rodriguez, HNG-41, (202) 366-1335.

STATUS : A contract has been awarded for the multi-year prioritization demonstrations. Demonstration sessions are expected to begin in the first quarter of 1996.

Bids are currently being evaluated for a contract to perform PMS engineering analysis demonstrations. The contract should be awarded by the end of 1995 and sessions are expected to begin in early 1997.

Bridge Design and Construction

Bridge design, as many other segments of civil engineering, has evolved from early art forms to a sophisticated science. A hundred years of experience have been assimilated into the engineering practice, and modern research and development findings have been re-examined, tested, proven in service, and codified into bridge specifications and practice. The traditional design philosophies and methods, such as Working Stress Design (WSD) and Ultimate Strength Design (USD), are still used in bridge design. However, recent developments in bridge design specifications have departed from the traditional approaches to incorporate more rational methods.

Load Factor Design (LFD) was a first step toward implementing a bridge design code based on statistical factors accounting for variability of loads, lack of accuracy in the analysis, and the probability of simultaneous occurrence of different loads. Load and Resistance Factor Design (LRFD) extended the philosophy to include resistance factors that account for the variability of material properties, structural dimensions and workmanship, and the uncertainty in the prediction of resistance. The LRFD code, properly applied, is expected to lead to more rational bridge designs that will produce more economical and durable highway bridges. A concerted effort to train bridge designers in the concept of load and resistance factors, as well as the application to bridge design, is crucial to the successful implementation of the new codes.

The LRFD specifications are ideal for assimilating new developments in bridge materials and construction methods, such as electroslag welding and high performance concretes, since resistance factors can be modified as necessary to represent uncertainties in material properties. Part of this project will involve promoting new bridge materials and construction methods and also implementing the LRFD code in bridge design software.

Recent innovative developments in bridge design codes, bridge materials, and construction methods have led to the establishment of 10 milestones.

1. Develop and initiate formal training sessions for the design of bridge superstructures and bridge foundations using the LRFD code.

2. Develop and initiate formal training sessions for the use of nondestructive load testing to determine load ratings of bridges.

3. Develop and initiate a demonstration project on electroslag welding for steel bridges.

4. Approve the LRFD specifications as the sole AASHTO code for design of highway bridges.

5. Upgrade major bridge design, analysis, and rating software with LRFD code: BRASS, AASHTO BDS.

6. Use High-Performance Concrete in a prestressed concrete bridge in Virginia.

7. Prepare Technology Transfer material and conduct a regional seminar on the use of High-Performance Concrete in a prestressed concrete bridge in Texas.

8. Use High-Performance Concrete in parallel structures conventional concrete in one, HPC in the other.

9. Establish an equipment loan program for SHRP-developed High-Performance Concrete test equipment.

10. Establish design and construction guidelines for High-Performance Concrete.

AP-21 Geotechnical Microcomputer Programs

DESCRIPTION : This project has involved the development of several geotechnical programs under contract with geotechnicai microcomputer programming firms. These programs have been made available to the States by the OTA.

BACKGROUND : The microcomputer industry has undergone rapid changes in recent years. New developments in hardware and software make the use of the microcomputer in civil engineering applications more feasible, practical, and almost indispensable.

The microcomputer can be used to solve many geotechnical problems that need repetitive and yet complicated calculations, such as analyzing embankment and foundation deformations, estimating pile behavior under static and dynamic forces, and calculating foundation settlements. Five of the microcomputer programs developed or under development are:

COM624P: Analyzes the behavior of piles or drilled shafts, subjected to lateral loads using the p-y method.

EMBANK: Determines one-dimensional compression settlement because of embankment loads.

SPILE: Calculates the ultimate static pile capacity in cohesive and cohesionless soils.

RSS: Analyzes stability of slopes that contain soil reinforcement. The analysis is performed using a two-dimensional limiting equilibrium method.

MSEW: Designs and/or analyzes required reinforcement for mechanically stabilized retaining walls, which does not consider specific facing configurations.

DRIVEN: This program is the updated version of the SPILE Program.

PILE FOUNDATION : This program will be developed based on the University of Florida program - LPGSTAN which is capable of analyzing bridge foundations subject to extreme events (hurricanes, ship and ice imports). The program will extend its capabilities to include the analysis and design of sound walls, retaining walls, signs and high mast lighting structures.

PROJECT MANAGER : Chien-Tan Chang, HTA-22, (202) 366-6749

STATUS : The SPILE Program has been upgraded, the new program is called Driven. This program is estimated to be completed by the end of 1995. RSS Program has been completed. It will be tested for about 2 months and will be distributed early December 1995. Contracts are being negotiated to develop a new version of MSEW program and a multiple faceted program called Pile Foundations.

AP-102 SHRP Distress Identification Manual

DESCRIPTION : The *Distress Identification Manual* is a pictorial rating manual for distress identification on highway pavements. The manual's photographs, descriptions, and illustrations provide a reference for the consistent identification and quantification of the severity and extent of pavement distress. It also provides a common language for describing cracks, potholes, rutting, spalling, and other pavement distresses. As a "distress dictionary," the manual has the potential to improve inter- and intra-agency communication while leading to more uniform evaluations of pavement performance.

The manual is divided into three sections that focus on particular types of pavement: (1) asphalt concrete surfaced, (2) jointed Portland cement concrete, and (3) continuously reinforced Portland cement concrete. Each distress is clearly labeled, described, and illustrated.

BACKGROUND : In 1987, the Strategic Highway Research Program (SHRP) began its largest and most comprehensive pavement performance the Long-Term Pavement Performance (LTPP) program. The *Distress Identification Manual* was developed as a tool for the LTPP program. It allows States and others to provide accurate, uniform, and comparable information on the condition of LTPP test sections. Moreover, it enables individuals and agencies to interpret LTPP data or to correlate LTPP findings with their own research efforts.

PROJECT MANAGER : James Walls, HNG-42, (202) 366-1339

STATUS : The SHRP distributed multiple copies of the latest color version of the *Distress Identification Manual* in July 1993. NHI will offer several training courses on the Manual to State and local highway agencies starting in the Fall of 1995.

Copies of the training materials will be made available to academia and the Technology Transfer Centers.

TECHNOLOGY TRANSFER AIDS : The project manager will continue to provide technical advice and participate in conferences, seminars, workshops, and user training sessions. Test and evaluation by a limited number of States is also anticipated.

PUBLICATIONS : *The Distress Identification Manual for the Long-Term Pavement Performance Project* can be purchased from the Transportation Research Board. Telephone: (202) 334-3214; Fax: (202) 334-2519 Cost: $20.

AP-118 Falling Weight Deflectometer Quality Assurance Software (SHRP)

DESCRIPTION : This project develops, markets, and distributes generic versions of the Strategic Highway Research Program's (SHRP's) Falling Weight Deflectometer (FWD) Quality Assurance software for use by State highway agencies. The generic versions accommodate various FWDs, sensor numbers, sensor spacings, and test protocols.

BACKGROUND : The SHRP FWD Quality Assurance Software is a spinoff product of SHRP's Long-Term Pavement Performance (LTPP) studies. It is one of four spinoff products SHRP recommended for FHWA implementation activities in 1992.

Falling Weight Deflectometers are used widely by highway agencies to collect pavement response data used in pavement rehabilitation design, pavement management systems, and forensic examinations of failed pavements. The overall goal of the SHRP FWD Quality Assurance Software is to ensure the consistent collection of high-quality pavement deflection data.

To provide quality assurance for FWD data collection, SHRP developed four software programs and established reference calibration centers at several State highway agencies to provide for quality measurement and data collection.

Since many of the State highway agencies either own or contract for deflection testing services by an FWD, the use of this quality assurance software should provide improved testing data. Unfortunately, all of this software was written specifically for SHRP and its methods. As an example, the programs are written to read data files from Dynatest FWD with seven sensors at the prescribed SHRP sensor spacing.

PROJECT MANAGER : Max Grogg, (518)431-4224.

STATUS : A Technical Working Group was established in 1993. During 1994 the LTPP Division continued to revise these software packages based upon their need, experience, and input from the Technical Working Group. These modifications should be completed by October 1995. In 1996 a consultant contract will be executed to perform the software modification. Additional funding will provide for training on the software and the calibration centers. Limited field testing by the SHAs will be conducted, and modified generic software will be marketed.

TE-14 Innovative Contracting Practices

DESCRIPTION : The objective of this project is to identify innovative contracting practices for evaluation and documentation that have the potential to reduce life-cycle costs to State highway agencies, while maintaining product quality and an acceptable level of contractor profitability. Practices tested under this contract include design/build, warranties, guarantees, lane rental, cost plus time bidding, and incentives/disincentives.

BACKGROUND : This project resulted from the work of a 1988 Transportation Research Board (TRB) task force that spent 3 years exploring innovative practices in the U.S. and abroad. Its findings were released as Transportation Research Circular Number 386, titled "Innovative Contracting Practices" (1991).

Another initiative relative to innovative contracting practices resulted from an asphalt pavement study group's 1990 European tour. The group was impressed with what it saw and recommended three innovative practices that could be pursued through a test and evaluation effort:

- Functional contracts (design/build),
- Warranties of riding surfaces, and
- Lane rental.

In addition, a fourth practice, cost-plus-time bidding, has gained widespread acceptance from State highway agencies.

PROJECT MANAGER : Wady Williams, HNG-22, (202) 366-0606

STATUS : This project has been operational for over 5 years and approximately 65 percent of the SHA's have participated at least once.

By far, the most popular technique used has been cost-plus-time bidding. Twenty-six States and the District of Columbia have used this method thus far. Six SHA's have either completed design/build contracts or have initiated such contracts. Contacts have been completed in Arizona and Colorado with favorable results. Total project time was substantially less than would have been expected for conventional design-bid-build projects, there was no significant change in design costs, and claims were essentially eliminated. Six SHA's have undertaken projects using the lane rental concept to reduce road-user impacts and, eight SHA's have chosen to use and evaluate warranty provisions.

In 1995 FHWA published *Rebuilding America: Partnership For Investment*, FHWA publication No. FHWA-PD-95-028, which contains descriptions of innovative practices and a list of projects using these practices.

TECHNOLOGY TRANSFER AIDS : Lane rental specifications, background information on warranties and guarantees (from the Transportation Research Board), and telephone and speaker assistance.

TE-18 Stone Matrix Asphalt

DESCRIPTION : The goal of this project is to test and evaluate the use of Stone Matrix Asphalt (SMA) on several test sections of U.S. highways to determine its construction feasibility and cost-effective performance. DP-90's mobile asphalt laboratories, its staff, and the Turner-Fairbank Highway Research Center staff are available to assist other States with SMA mix design information. The mobile asphalt laboratories provide materials analysis on-site while supporting quality control and compliance.

BACKGROUND : In 1990, a team of State, industry, and Federal engineers from the U.S. participated in a European Asphalt Study Tour. Their mission was to identify promising asphalt technologies. Of the asphalt mixture technologies studied, SMA had great promise for use in this country.

SMA is an asphalt mixture developed in the 1980's in Germany to provide a rut-resistant pavement surface layer. SMA's proven performance is attributed to a "gap graded" aggregate gradation that provides a stone-to-stone structure held together by a durable asphalt cement, mineral filler, and fiber matrix. SMA is routinely used in many parts of Europe.

PROJECT MANAGER : John Bukowski, HTA-21, (202) 366-1287

STATUS : Interest in SMA remains strong. To date, project presentations have been made at nearly 100 locations to thousands of government and industry individuals interested in the various aspects of material selection, design, construction, and performance. Continuing interest in SMA is evident by the increasing number of States that participate and the tonnage of SMA used in projects.

Year	Number of States	Tons of SMA
1991	4	less than 50,000
1992	12	100,000
1993	15	200,000
1994	23	300,000
1995	27	400,000

Extensive monitoring is under way on more than 50 separate test sites constructed in Maryland, Georgia, Virginia, Texas, California, Alaska, Arkansas, New Jersey, Kansas, Illinois, Ohio, Michigan, Wisconsin, Indiana, and Missouri. Data from these projects are being analyzed and model specifications have been disseminated. Further evaluation is targeting mixture design, cost reduction, quality control, and predictive performance of the SMA pavements. SMA sites are being visited and evaluated by a contractor, which should lead to a greater understanding and more systematic evaluation approach. A mix design research effort funded by the NCHRP 9-8 is underway at the National Center for Asphalt Technology and Auburn University. Efforts are also underway to use some of the Superpave mix technologies in designing SMA.

TECHNOLOGY TRANSFER AIDS : Telephone and on-site assistance, speakers, mix design assistance (based on laboratory availability), and mobile laboratory (subject to long-range planning).

PUBLICATIONS : SMA Model Materials Selection and Construction Guidelines are available through the Office of Technology Applications and are also being distributed by the industry. Copies of material on European SMA Synthesis also are available upon request.

TE-21 Pavement Condition Measurement (SHRP)

DESCRIPTION : This project evaluates and promotes state-of-the-art pavement condition evaluation equipment and consolidates previous ongoing activities with SHRP implementation efforts related to pavement condition measurement. The project will be expanded to include new technology as it becomes available.

Three kinds of equipment have been evaluated through field test and evaluation:

- SHRP Ground Penetrating Radar
- SHRP Seismic Pavement Analyzer
- Fully Automated Pavement Distress Measuring Equipment

PROJECT MANAGERS : Luis Rodriquez, HNG-41, (202) 366-1335 and George Jones, HNG-41, (202) 366-1338.

STATUS : The final report on the fully automated pavement distress measuring equipment has been completed and distributed to all State highway agencies. Reports on additional equipment analysis will be issued upon completion of field test and evaluation. A follow-up test was conducted in North Carolina during December 1994. North Carolina DOT is currently completing the data analysis from that test.

The Technical Working Group met and decided not to fund any additional testing of either the ground penetrating radar or the seismic pavement analyzer. The developers of both pieces of equipment are continuing with the equipments' development. Commercial development through the private sector is encouraged.

TECHNOLOGY TRANSFER AIDS : Test and evaluation in selected States through work orders and equipment loan. A follow-up program of workshops, seminars, and literature is envisioned.

TE-25 Strategic Highway Research Program Work-Zone Safety Devices

DESCRIPTION : To improve safety and efficiency of day-to-day maintenance and operations of work zones, the Strategic Highway Research Program (SHRP) produced 12 devices that are applicable in work zones, especially for maintenance activities.

1. Salt Spreader Truck Mounted Attenuator (TMA)
2. Portable Crash Cushion **
3. Ultrasonic Detection Alarm
4. Infrared Intrusion Alarm **
5. Queue-Length Detector **
6. Portable Rumble Strip **
7. Direction Indicator Barricade **
8. Opposing Traffic Lane Divider **
9. Diverging Lights
10. Flashing STOP/SlOW Paddle **
11. All-Terrain Sign & Stand
12. Remotely Driven Vehicle

** Interest indicated by commercial fabricators.

The Salt Spreader Truck Mounted Attenuator is commercially produced and marketed exclusively by private industry. Six of the other devices, representing the basic SHRP developed concepts, are commercially available and are ready for trial field use. These include the Opposing Traffic Lane Dividers, Portable Rumble Strip, Flashing STOP/SLOW Paddle, Direction Indicator Barricades, Work Zone Intrusion Alarms, and the All-Terrain Sign Stand with Signs. The Portable Crash Cushion and the Remotely Driven Vehicle are being modified to improve their performance. The Queue-Length Detector and Diverging Lights have had technical problems that remain unsolved and also appear to have a limited market demand. Further work on these two devices is on hold.

PROJECT MANAGER : Joe Lasek, HHS-11, (202) 366 2174

PROJECT COORDINATOR : Peter Hatzi, HTA-31, (202) 366 8036

STATUS : Most of the devices have been exhibited by the FHWA and SHRP staff at many national and regional conferences and technical shows. The purpose of showcasing the devices during fiscal years 1992 through 1994 is to acquaint potential users with these new devices and to develop interest in their use.

FHWA supports activities to provide the various devices to State highway agencies for trial use and evaluation. A solicitation of interest was made to the State DOTs through FHWA division offices. Based upon responses, funds were provided to the States to acquire limited numbers of the devices for trial use under actual work conditions. In return information on the overall performance of the devices will be provided to FHWA.

Some additional funding will be made available in FY 1994 for acquiring Intrusion Alarms and other devices that may become available for trial use and evaluation. The funding will be provided under normal Federal aid procedures. Through this evaluation method, FHWA will accumulate an information base on the in-service performance of the various devices, while allowing the States to gain experience with them.

TE-27 Innovative Pavement Materials & Treatments

DESCRIPTION : This project provides States an opportunity to evaluate SHRP pavement maintenance products and techniques by introducing preventive maintenance technology and principles. Technical assistance will be provided on surface treatments and guidance will be available in the use of innovative materials. SHRP technology in two areas is included:

- Effectiveness of pavement preventive maintenance: management concepts, optimum timing of various surface treatment applications, guide specifications for preventive maintenance, and a 1-day workshop.

- Innovative materials: pothole patching, crack sealing, joint sealing, spall repair and other materials and surface repair guidelines, introduction of objective data collection techniques for joint seal effectiveness, and a 1-day workshop.

PROJECT MANAGER : Patrick Bauer, HNG-21, (202) 366-1554 and Michael Smith, HNG-42, (202) 366-4057.

PROJECT COORDINATORS : Jim Sorenson, HNG-42, (202) 366-1333 and Gary Henderson, HTA-21, (202) 366-1283.

STATUS : Showcase contracts have been awarded for Preventive Maintenance and Innovative Materials, and pilot workshops have been conducted. Test and Evaluation programs are under development. The first pilot workshop was held in May, 1995, in Colorado. The second pilot is being held in September, 1995, in Arizona. It is anticipated that workshops for both technologies will be available in the late Fall of 1995.

TECHNOLOGY TRANSFER AIDS : Seminars, technical assistance, and field test and evaluation work orders.

TE-28 SHRP Snow and Ice Technology

DESCRIPTION : This project tests and evaluates SHRP snow and ice technology products in five major areas: snowplow cutting edges, snow fences, roadway weather information systems, anti-icing technologies, and de-icing chemicals. The project will provide an opportunity for States to test and evaluate better designed snowplows and snow fences, improved storm forecasting and communication methods, and more efficient and effective snow removal and ice control methods.

The primary products emerging from this SHRP technology area are design guides, manual of practice for anti-icing operations, research reports, handbooks, evaluation methodologies, and improved snow removal equipment. Guidelines have been developed for evaluating equipment, materials, and methods for utilizing anti-icing technology. FHWA's implementation effort of the SHRP technology has three parts:

- Anti-icing Technology through a technical services support agreement with U.S. Army Corp of Engineers (Cold Regions Research and Engineering Laboratory CRREL).

- Showcasing contract incorporating workshops, field test and evaluation, and equipment loans.

- Field Test and Evaluations through work orders with State highway agencies.

PROJECT MANAGERS : Salim Nassif, HNG-21, (202) 366-1557;Chung Eng, HNG-21, (202) 366-1555.

PROJECT COORDINATOR : Gary Henderson, HTA-21, (202) 366-1283

STATUS : Product/technologies currently being evaluated include weather information systems for highway operations, anti-icing operations, innovative snow fence design and construction, and snow scoops. Additional products/technologies and participants will be added through the showcasing contract. Work orders were established with 15 State highway agencies to evaluate the effectiveness of SHRP anti-icing techniques over the 1993/94 and 1994/95 winter period. Work orders were also established with an additional seven State highway agencies; four to evaluate the Wels portable interactive weather prediction system, and several other weather services in terms of usefulness and accuracy for highway operations; two to evaluate snow fences designed in accordance with SHRP guidelines; and one to evaluate the effectiveness of the snow scoop retrofitted to their existing plows.

A showcase contract has been executed to package the various technologies and develop a series of workshops and seminars focusing on snow and ice technologies. Additional field trials will be initiated with selected States to further evaluate various products by winter 1995/96. Workshops will begin during the first quarter of 1996.

TECHNOLOGY TRANSFER AIDS : Workshops on snow and ice technology will be available in the near future. Following standard work order procedures, States may participate in field tests and evaluations of selected products. Technical assistance will be available to guide participants on proper application and evaluation of products/technology. Limited funding is available.

Pavement Management Technology : This technology group focuses on those technologies related to identification, evaluation, and testing for pavement distress and collection of pavement performance data. It includes a Distress Identification Manual and several pieces of equipment developed under the Strategic Highway Research Program's Long Term Pavement Performance (LTPP) program. Programs under this group will establish a continuing effort to test and evaluate emerging equipment and technology and will provide comprehensive reports of testing results to the industry. This effort will result eventually in more accurate and consistent distress identification and performance data.

TE-30 High Performance Rigid Pavements (HPRP)

DESCRIPTION : The immediate goal of the HPRP Program is to construct some selected highway projects to explore the applicability of other innovative concrete pavement design and construction concepts in the United States. The long range goal is further improvement of cement concrete pavement design, materials, and construction technology and equipment through innovation, research, training, and following pavement technology developments in other nations.

BACKGROUND : In 1992 a team of State, industry, and Federal engineers participated in the U.S. Tour of European Concrete Highways. Their mission was to review European concrete pavement experiences and obtain information relating to finance, research, design, construction, maintenance, and performance to assist with development of appropriate actions for enhancing the U.S. highway system. The follow-up visits to Germany and Austria obtained sufficient information to construct experimental sections using German design and Austrian exposed aggregate surface treatment technique to reduce tire/pavement noise..

PROJECT MANAGER : John M. Becker, HNG-40, (202) 366-1340

PROJECT COORDINATOR : Suneel Vanikar, HTA-21, (202) 366-0120

STATUS : In 1993 a 1-mile test section was constructed on I-75 (Chrysler Freeway) in downtown Detroit, Michigan. The design and construction procedures of the experimental pavement section were similar to those used in Germany and Austria. The project will be monitored for 3 years and evaluation reports have and will be prepared. An open house was organized during construction to demonstrate the European design and construction technology. FHWA plans to participate in additional projects incorporating some of the European and other innovative design features.

State Highway Agencies have been asked to submit proposals for HPRP projects by October 10, 1995. Expert Working Groups will be formed to select projects for FY 1996 funding, to evaluate HPRP performance and to oversee open house activities and to develop T^2 workshops.

TECHNOLOGY TRANSFER AIDS : Telephone and on-site assistance, speakers, and mobile laboratory.

PUBLICATIONS : *Report on the 1992 U.S. Tour of European Concrete Highways*. 1992, and *Summary Report of Follow-up Tour of Germany and Austria*, 1993. Both reports are available through the Office of Technology Applications. A video-tape on the Michigan project is available from the Office of Technology Applications..

TE-34 SHRP Concrete Showcase Contracts

CONCRETE MIX DESIGN AND CONSTRUCTION AIDS (SHRP)

DESCRIPTION : This project provides State DOTs and industry with SHRP-developed information on concrete mix design and curing tables along with providing technical assistance for implementation. Curing tables will aid resident engineers and contractors in their decision process.

BACKGROUND : Packing diagrams have been developed by SHRP to get dense concrete. The diagrams are used as mix design techniques. Properly used, the mix design may improve tensile strength and durability. Curing tables have been developed and include temperature, cement content, and critical dimensions to aid proper curing. The goal of these efforts is to obtain dense, impermeable, and durable concrete with minimum cracks.

PROJECT MANAGER : Suneel Vanikar, HTA-21, (202) 366-0120

STATUS : A Work Order was provided to the Indiana DOT in 1992 to perform field verification of packing diagrams, and field testing and evaluation are complete. A work order was provided to the University of Louisville for additional testing and evaluation in 1994 and is underway. Minnesota DOT conducted their own packing handbook evaluation in 1994. In 1994, the Missouri HTD examined the packing handbook for possible use in mix design.

In 1994, these products were promoted through presentations, and they will be incorporated into other SHRP-related implementation efforts for concrete durability and high performance concrete.

In 1995, the draft Packing Handbook evaluation report and the Curing Tables evaluation report were sent to AASHTO and distributed to members of the Technical Working Group.

TECHNOLOGY TRANSFER AIDS : Presentations are available upon request from the Office of Technology Applications.

CONCRETE DURABILITY (SHRP)

DESCRIPTION : This project will showcase SHRP-developed products and provide education and technical assistance to State DOTs and the industry by developing and presenting workshops and providing testing equipment to State DOTs through an equipment loan program.

This implementation effort includes new test procedures for D-Cracking potential of aggregates, a revised test procedure for freeze-thaw durability, and specifications for aggregates. It will also include an expert system for rehabilitation strategy. The durability of concrete structures and pavements is a key issue in rebuilding infrastructure.

PROJECT MANAGER : Gary Crawford, HTA-21, (202) 366-1286

PROJECT COORDINATOR : Suneel Vanikar, HTA-21, (202) 366-0120

STATUS : Five impact echo devices, five in situ surface air flow permeameters and five hydraulic fracture devices have been purchased and are available through an equipment loan program. The impact-echo device has been loaned to ten agencies, the surface air flow permeameter has been loaned to eight agencies, and the hydraulic fracture device has been loaned to five interested highway agencies. The products are being promoted through a manual, workshops, equipment loans, and technical assistance. Consultant services were obtained in 1994 to develop and present workshops, showcase products, manage the equipment loan program, and provide technical assistance. A pilot workshop was held in Virginia in June 1995. Regional workshops will start in late 1995 and continue through 1996. Some products will also be demonstrated in the FHWA mobile concrete laboratory.

TECHNOLOGY TRANSFER AIDS : Workshops, equipment loans, and technical assistance through consultant services. A manual will be developed for the workshops.

ALKALI-SILICA REACTIVITY (ASR) AND FLORESCENT MICROSCOPY (SHRP)

DESCRIPTION : This project will provide education and technical assistance to State DOTs and the industry while showcasing SHRP-developed products relating to alkali-silica reactivity (ASR) and florescent microscopy.

ASR is a problem for many States, particularly those with concrete pavements. This implementation effort includes identification of ASR, field and laboratory tests, mitigation of ASR in existing structures, and mix design procedures to reduce potential for ASR.

The project will develop and present workshops, provide testing equipment to State DOTs through an equipment loan program, and provide technical assistance.

PROJECT MANAGE : Roger Surdahl, HNG-23, (202) 366-1563

PROJECT COORDINATOR : Suneel Vanikar, HTA-21, (202) 366-0120

STATUS : Six ASR field detection test kits have been purchased. The consultant contract to develop a 3-day workshop and other showcase activities was awarded in 1993. A pilot workshop was held in Pennsylvania in late 1994. Workshop presentations started in 1995, and workshops were presented in Nebraska, New Jersey, North Carolina, Wyoming, Nevada, Oregon, Minnesota, and New Mexico. An equipment loan program has been established, and technical assistance is provided under the contract. Equipment loan and technical assistance were provided to Pennsylvania, Nevada, Idaho, Delaware, Oregon, and Indiana DOT's. Field testing of lithium compounds to minimize ASR is underway in New Mexico, Nevada, New Hampshire, and Pennsylvania.

In 1996, the products will be promoted through a manual, additional workshops, product showcasing, and technical assistance. Some products will continue to be demonstrated in the FHWA mobile concrete laboratory.

TECHNOLOGY TRANSFER AIDS : Workshops, equipment loans, and technical assistance through consultant services.

Concrete Pavement Design and Construction

The concrete pavement design and construction technology group focuses on innovative designs and construction techniques that provide immediate solutions to specific Portland cement concrete pavement problems. The range of technologies addresses water in pavements, faulting joints and cracks, paving under limited time restrictions, pavement durability and economy, and methods of achieving improved overall performance through performance-related specifications.

Several projects incorporating emerging technologies for design and construction are in development stages. These include high-performance rigid pavement design and construction methods, various concrete pavement texturing techniques to minimize noise and enhance safety, and evaluation and implementation of performance-related specifications for concrete pavements.

TE-36 High-Performance Concrete

DESCRIPTION : This national effort will include seminars, workshops, equipment loan programs, demonstration bridges, and technical assistance to evaluate, showcase, and promote high performance concrete and SHRP research products in high performance concrete. The initial goals are to obtain all equipment, specifications, test procedures, and reference documents related to the subject; organize the materials; develop seminar and workshop technology transfer materials; and plan an equipment loan program. The secondary goals are to present seminars and workshops, implement the equipment loan program, provide technical assistance, and construct Demonstration Bridges.

BACKGROUND : The Strategic Highway Research Program (SHRP) supported considerable research into high performance concrete. As a result of this research, new testing methods have been developed and some existing testing methods have been modified to 1) determine the validity of existing test methods; 2) give greater uniformity to test results; and 3) give engineers greater confidence in the material properties of high performance concrete.

A major goal of SHRP was to develop improved criteria and testing methods for the mechanical properties and behavior of high-performance concrete. The training and dissemination of information to personnel (governmental and industry) required to perform tests and mixture design is an essential step for the effective use of new field identification procedures, test procedures, and mixture design methods.

PROJECT MANAGER : Terry D. Halkyard, HTA-22, (202) 366-6765

PROJECT COORDINATOR : John M. Hooks, HTA-22, (202) 366-6643

STATUS : A national multi-year effort is planned that would target a maximum number of interested government and private industry engineers and technicians. This effort will promote the use of high performance concrete and the thorough evaluation of SHRP-developed products to transfer technology to a wide audience throughout the United States. High performance concrete is being used in bridges under construction in Nebraska, Texas and Virginia, and plans are being made for its use in bridges in New Hampshire, Ohio, Colorado, Georgia and Washington. A workshop on the use of high performance concrete in the Texas bridge is planned for early 1996.

TECHNOLOGY TRANSFER AIDS : Workshops on High Performance Concrete, technical assistance, speakers, and presentation materials.

Bridge Inspection and Bridge Management

More than 40 percent of the Nation's 575,000 highway bridges are functionally obsolete or structurally deficient. These deficient structures represent significant impediments to the safe, economical use of the highway system and result in safety hazards, high user costs, and huge outlays for preservation and replacement. Balanced against this backlog of bridge needs is a generally inadequate level of funding by public agencies for infrastructure needs.

The collapse of the Silver Bridge in 1967 was the immediate catalyst for what became a comprehensive bridge safety inspection program mandated by the National Bridge Inspection Standards (NBIS). Every bridge on a public road must be inspected at least every 2 years and highway agencies across the Nation have inspection staffs and programs that collect and update critical bridge inventory and inspection data. After almost 20 years, there is still a manifest need to more effectively analyze this data, to better define bridge needs, and to find effective solutions.

The complexities and costs associated with preserving the Nation's bridge infrastructure demand innovative approaches to collection and analysis of data and prediction of current and future bridge preservation actions. These needs, coupled with the availability of modern analytical methods and high-speed computers, are leading to the development of comprehensive bridge management systems. Prior to the late 1980s, there were no existing management systems adaptable to the management of bridge programs nor was there any clear definition of key bridge management principles or objectives. Therefore, in cooperation with AASHTO, California DOT, and a specially formulated technical working group (TWG) representing several State DOT's, OTA was able to establish the following primary requirements of a comprehensive Bridge Management System (BMS):

General Procedures

1. Identify and establish responsibility for data collection and management and for bridge decision making based on a comprehensive BMS.

2. Coordinate program and project-level decisions and coordinate bridge maintenance and improvement actions and a process of priority programming.

3. Ensure a clear method of communicating needs and programs to outside audiences.

Functional Needs

1. Automated database of bridge inventory, condition data, and a historical data file.

2. Deterioration models for projecting future condit[illegible] of bridge elements with or without intervening actions.

3. Identify costs related to feasible actions, user costs associated with a deficient bridge condition, and budget and other key constraints.

4. Develop multi-period procedures and reporting capabilities.

Efforts to define modern bridge management led to a cooperative effort with California DOT and the TWG to be develop the PONTIS BMS. With Pontis under development, and with the added incentive of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, six milestones were established:

1. Publish Version 2.0 of PONTIS, the BMS jointly developed by FHWA, California DOT and the TWG (complete); accomplish transfer of PONTIS support to the AASHTOWare software system (complete).

2. Develop and begin formal BMS training sessions for bridge inspectors and bridge managers (sessions to be underway beginning in October 1995).

3. Establish an FHWA network of BMS specialists and regional TWGs to provide BMS training and support to SHA and local agency bridge managers (underway).

4. Implement a Commonly Recognized (CoRe) Element system to define standard bridge elements

(complete); establish uniform method of converting core element condition data to NBI format (ready for adoption).

5. Each State implement a comprehensive BMS (underway).

6. Organize a new demonstration project to promote innovative computer hardware and software to improve efficiency and quality of bridge data collection and management (scheduled to begin in FY 1997).

TE-39 SHRP Asphalt Support Projects

This project supports a multitude of activities to promote the SHRP asphalt program.

PROJECT MANAGERS : The managers for all TE-39 projects are: John D'Angelo, HTA-21, (202) 366-0121; Thomas Harman, HTA-21, (202) 366-0859; and John Bukowski, HTA21, (202) 366-1287.

POOLED FUND EQUIPMENT STUDY SUPPORT (SHRP)

DESCRIPTION : FHWA, in cooperation with AASHTO and SHRP, initiated a pooled fund study that gives the participating States the opportunity to acquire SUPERPAVE asphalt binder and mix test equipment. Since the pooled fund announcement on January 10, 1992, States have committed at least a portion of the estimated $335,000 to purchase the equipment. The pooled fund study allows each State to use its Federal SP&R monies without matching funds.

STATUS : Procurement of the equipment is scheduled for a 4-year period. All participating States have received the SUPERPAVE binder equipment. The mix design equipment must go through further development with a series of first article testing. This process should allow for a more rigid analysis of the equipment prior to the purchase. The States have received the gyratory compaction equipment to begin work on the SUPERPAVE mix design system.

Procurement of the mixture analysis equipment and the SUPERPAVE Shear Tester and Indirect Tensile Tester will initially be limited to six units. These will be evaluated at SUPERPAVE Centers established in PA, AL, TX, NV, and IN as well as at the FHWA TFHRC. Equipment procurement for all State DOTs of these devices is scheduled for 1996.

TECHNOLOGY TRANSFER AIDS : Equipment on loan (subject to availability), State reports available through the Office of Technology Applications (subject to availability), and telephone assistance.

SHRP ASPHALT EQUIPMENT LOAN PROGRAM

DESCRIPTION : This project evaluates asphalt binder equipment developed to support the binder specification under the Strategic Highway Research Program (SHRP). The Office of Technology Applications (OTA) has five sets of asphalt cement testing equipment, plus one set for OTA and one set for FHWA's Research and Development. This equipment includes:

- Bending beam rheometer with computer
- Dynamic shear rheometer with computer
- Pressure aging vessel
- Direct tension tester with computer
- Brookfield viscometer

Ruggedness and precision/bias data are being collected for the final specifications (a secondary but very important purpose of this project). OTA is working closely with the AASHTO Subcommittee on Materials to accomplish this expeditiously.

STATUS : All equipment has been delivered and will continue to be loaned to States within each user-producer group. Funding also involves workshops (that include the user-producer group concept) and evaluation monies, as required.

TECHNOLOGY TRANSFER AIDS : Equipment specifications, vendor list, and provisional test procedures. Binder technicians are available for on-site training, three-day workshops, and telephone assistance.

SHRP F[illegible] IMPLEMENTATION ASPHALT

DESCRIPTION : This project w[illegible] provide technical assistance to the States in the local use of Superpave equipment provided under the pooled fund buy. A competitive contract was awarded to the Asphalt Institute for field engineers and technicians to assist the States. Assistance will include equipment setup, testing, test interpretation, local workshops, training in the design and construction of mixes, and guidance for the construction of Special Pavement Section (SPS) 9 design and construction. This project will be closely integrated with LTPP.

STATUS : The contract was be let in FY 1995 and will last for 3 to 5 years.

TECHNOLOGY TRANSFER AIDS : On site training, field and telephone technical assistance.

SHRP SUPERPAVE MODELS

DESCRIPTION : This project will assist in completing the SHRP work on the model ~ that underpin SUPERPAVE. The effort will be completed through a competitive bid contract. The work will include software support, model documentation, and further refinement and documentation. The contract for technical assistance win be let in 1993 and operate for 3 to 4 years.

STATUS : Procurement is on hold until the SHRP reports on the models are made available to include in procurement documents.

GEORGIA LOADED WHEEL TESTER (LWT)

DESCRIPTION : This project supports SHRP asphalt implementation efforts by evaluating innovative asphalt testing equipment. Products under consideration include the nuclear asphalt content gauge, indirect tensile test, moisture sensitivity tests, and most significantly, the Georgia Loaded Wheel Tester (LWT). While not directly associated with SHRP, this project will finance additional evaluations of SHRP-developed products not specifically identified in the pooled fund buy.

BACKGROUND : The Georgia LWT was developed by Dr. Jim Lai at Georgia Tech, in cooperation with the Georgia DOT. It is a quick, efficient, and inexpensive method for determining rut susceptibility of surface mixes. Georgia DOT has developed a specification that is used on all high-traffic roadway projects and other projects where rutting susceptibility is a concern.

FHWA sponsored a round-robin test program with six State DOTs to evaluate the Georgia device, which was found to be repeatable and reproducible. A Work Order with Georgia DOT was issued by FHWA to modify the device to make it semiautomatic and controlled electronically. The modified device is capable of testing multiple samples at one time and handling 75 by 125 by 375 mm samples. The temperature and the hose pressure also are adjustable.

A second round-robin test program is planned to evaluate the modified device.

STATUS : Five States have evaluated the Georgia LWT and will report their findings during the next several years. Georgia Tech has upgraded several features of the LWT to make it semiautomatic and electronically controlled. This modified device is being tested currently. An Expert Task Group was assembled in late 1993 as States completed their evaluations. Funding for this project considers additional State evaluations of this and as yet undefined equipment and techniques that show promise.

TECHNOLOGY TRANSFER AIDS : Equipment loans, field and telephone technical assistance.

Asphalt Pavement Design and Construction

The asphalt pavement design and construction technology group focuses on innovative techniques for design and construction of high performance asphalt pavements used in new construction, reconstruction, rehabilitation, restoration, or resurfacing.

Since 1987, the Federal Highway Administration (FHWA) has supported the "Development of Performance-Related Specifications for Highway Construction" as one of its high priority research areas. Performance-related specifications (PRS) require materials and construction tests, the results of which correlate to a known degree with the performance of the completed product. A series of FHWA, National Cooperative Highway Research Program (NCHRP), and State Planning and Research (SP&R) studies have produced the initial framework and at least a partial system of PRS for hot mix asphalt pavement construction.

The focus in the PRS is on quality control of construction selecting the best available materials and establishing the mix and pavement designs. PRS addresses three questions:

- What quality control tests need to be run *during* construction to minimize premature fatigue cracking or rutting?
- What is the impact on the subsequent performance of deviations from the target values of properties such as density or asphalt content, or both?
- What payment adjustments are appropriate when such deviations are encountered?

The focus of other projects under this technology group is to evaluate these specific technologies to determine the optimum procedure to achieve quality construction and high performance asphalt pavements.

TE-44 Electrochemical Chloride Extraction from Reinforced Concrete Structures

DESCRIPTION : The objective of this project is to demonstrate and document the results established under the SHRP Study. A secondary objective is to work in conjunction with States, private sector, and academia to collect data on new structures protected using the chloride extraction method. Pilot projects will include installations on both the decks and substructures.

BACKGROUND : Corrosion of reinforcing steel is recognized as one of the major contributors to the deterioration of reinforced concrete structures, and the chloride ions that penetrate to the level of the reinforcing bars are a critical element in the corrosion process. One technique for dealing with this problem is chloride extraction. The electrochemical extraction of chloride from concrete structures is accomplished by applying an anode and an electrolyte to the concrete surface and passing direct current (DC) between the anode and the reinforcing steel, which acts as a cathode. Since anions (negatively charged ions) migrate toward the anode, it is possible to cause the negatively charged chloride ions to migrate toward the anode and away from the steel. Chloride extraction is similar in principle to cathodic protection (CP). The major difference is in the magnitude of the current, which is about 100 to 500 times that used for cathodic protection. The total amount of charge (current time) applied for chloride extraction is about the same as a CP system would deliver over a period of about 10 years. The other important difference is that chloride extraction is a short-term treatment, whereas cathodic protection is normally intended to remain in operation for the life of the structure.

PROJECT MANAGER : Donald R. Jackson, HTA-22, (202) 366-6770

STATUS : A work order with Virginia and South Dakota Departments of Transportation to install and evaluate the electrochemical chloride extraction procedure was approved for a bridge carrying 34th Street over I-395 into Arlington, Virginia, and a bridge in Sioux City, South Dakota. The procedure was installed on three sections of the Virginia deck and three piers of the South Dakota bridge in the early spring of 1995. The procedure was also installed on three substructure piers on a structure in Charlottesville, Virginia, in the Spring of 1995.

Open houses were held for the Virginia and South Dakota installations in August 1995. The Open Houses were well attended. Ten States were represented at the Virginia Open House, and five at the South Dakota Open House. The South Dakota Open House took place on August 9, 1995 in Sioux City. Fifty guests, representing Federal, State, academic and private sector organizations, attended each Open House.

CHAPTER 11
REGION / DIVISION ITEMS