Proceedings of the Conference
STRATEGIC HIGHWAY RESEARCH
PROGRAM AND TRAFFIC SAFETY
ON TWO CONTINENTS in
Gothenburg, Sweden,
September 18 – 20, 1991, Part 5

- Asphalt
- Highway Operations and Concrete and Structures
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- Asphalt
- Highway Operations and Concrete and
  Structures
# Abstract (background, aims, methods, results) max 200 words:

**Papers presented at the seminar were as follows:**

- **Asphalt Model:** Results of the SHRP Asphalt Research Program (Jones, D R and Kennedy, T W);
- **SHRP Asphalt-Aggregate Mix Analysis System (AAMAS)** (Kennedy, T W, Cominsky, R J, Harrigan, E T and Leahy, R B);
- **Investigation of Asphalt-Aggregate Interactions and Their Sensitivity to Water** (Curtis, C W, Perry, L M and Brannan, C J);
- **Thermal Fatigue Cracking of Asphalt Concrete Pavements - An Experimental Approach** (Jackson, N M, Vinson, T S and Janoo, V);
- **Development of Test Methods for a Performance-Related Bitumen Specification** (Anderson, D A and Christensen, D W);
- **Characterization of Asphalt by NMR Spectroscopy and High Performance Gel Permeation Chromatography** (Jennings, P W, Pribanic, J A S, Desando, M A, Raub, M F, Moats, R Mendes, T M, Hoberg, J O, Smith, M A and Stewart, F F);
- **Asphalt Research in the Netherlands** (Hopman, P C, Kunst, P A J C, Pronk, A C, Molenaar, J M M and Molenaar, A A A);
- **Closed Track Testing of Maintenance Work Zone Safety Devices** (Hanscom, F R);
- **Innovative Materials for Pavement Surface Repairs** (Shah, S);
- **MINSALT - A 5-Year Study to Minimize the Negative Effects of Salt** (Gustafson, K and Oeberg, G);
- **DEICING SALT - Its Use and Effect on Road Safety and the Living Conditions of Roadside Trees and Shrubs** (Giesa, S);
- **Improving Concrete Pavements Through SHRP Research** (Hanna, A N and Jawed, I);
- **Optimization of Highway Concrete Through Combined Use of Particle Packing Modelling, Rheological Studies, Computer Simulations and Compaction Simulations** (Holm, J and Andersen, P J);
- **High-Performance Road-Surfacing Concrete with Good Resistance to Wear by Tyre Studs** (Nilsson, M);
- **Maintenance and Repair of Highway Concrete Bridges: A Case Study** (Al-Babtain, I and Abbas, A M).
PREFACE

The Swedish Road and Traffic Research Institute (VTI) and the US Transportation Research Board (TRB) of the National Research Council were jointly organising this international conference. The objective was to cover the present and future road research with special emphasis on the Strategic Highway Research Program (SHRP), as well as the research concerning drivers and vehicles as related to highway safety.

SHRP is a fully funded, $ 150 million (US), five year program of research directed at asphalt, concrete and structures, highway operations, and long term pavement performance.

In the sessions on roads there were presentations which highlighted differences between European and US practices and needs, and the discussions were concentrated on how to promote international involvement in SHRP and application of its research, within the areas of Asphalt, Long Term Pavement Performance (LTPP), Highway Operations and Concrete and Structures.

In the different road safety sessions there were presentations of actual research in different countries and discussions of the differences that exist between Europe and the USA, trying to explain the reasons for them and examine whether they are reasonable and acceptable.

Linköping October 1991

Kenneth Asp

Proceedings of the Conference STRATEGIC HIGHWAY RESEARCH PROGRAM AND TRAFFIC SAFETY ON TWO CONTINENTS in Gothenburg, Sweden, September 18-20, 1991:

VTI RAPPORT 372A, Part 1
- Opening
- Motorist Information Systems
- Accident Studies and Safety Management

VTI RAPPORT 372A, Part 2
- Roadside Safety Features
- Human Engineering, Training and Traffic Safety

VTI RAPPORT 372A, Part 3
- Operational Roadway and Workzone Research
- Safety and Mobility of Older Drivers

VTI RAPPORT 372A, Part 4
- Simulation and Measurement of Operator and Vehicle Performance
- Strategies to Increase the Use of Restraint Systems

VTI RAPPORT 372A, Part 5
- Asphalt
- Highway Operations and Concrete and Structures

VTI RAPPORT 372A, Part 6
- Long-Term Pavement Performance
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## ASPHALT

### Asphalt Model: Results of the SHRP Asphalt Research Program

David R Jones and Thomas W Kennedy, University of Texas at Austin, USA

### SHRP Asphalt–Aggregate Mix Analysis System (AAMAS)

Thomas W Kennedy and R J Cominsky, University of Texas at Austin, E T Harrigan and Rita B Leahy, Strategic Highway Research Program, USA

### Investigation of Asphalt–Aggregate Interactions and Their Sensitivity to Water

Christine W Curtis, L M Perry and C J Brannan, Auburn University, USA

### Thermal Fatigue Cracking of Asphalt Concrete Pavements – An Experimental Approach

N Mike Jackson, T S Vinson, Oregon State University and Vincent Janoo, USA CRREL, USA

### Development of Test Methods for a Performance-Related Bitumen Specification

David A Anderson and Donald W Christensen, Jr, The Pennsylvania State University, USA

### Characterization of Asphalt by NMR Spectroscopy and High Performance Gel Permeation Chromatography

P Wyn Jennings, J A S Pribanic, M A Desando, M F Raub, R Moats, T M Mendes, J O Hoberg, J A Smith and F F Stewart, Montana State University, USA

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Asphalt Research in the Netherlands 105
Piet C Hopman, Delft University of Technology, PA J C Kunst, Netherlands Pavement Consultants bv, A C Pronk, J M M Molenaar, Roads and Hydraulic Department of Rijkswaterstaat and A A A Molenaar, Delft University of Technology, The Netherlands

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Closed Track Testing of Maintenance Work Zone Safety Devices 125
Fred R Hanscom, Transportation Research Corporation, USA

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Shashikant Shah, Strategic Highway Research Program (SHRP), USA

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Kent Gustafson and Gudrun Öberg, Swedish Road and Traffic Research Institute (VTI), Sweden

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Siegfried Giesa, Technical University Darmstadt, Germany

Improving Concrete Pavements Through SHRP Research 185
Amir N Hanna and Inam Jawed, Strategic Highway Research Program (SHRP), USA
Optimization of Highway Concrete Through Combined Use of Particle Packing Modelling, Rheological Studies, Computer Simulations and Compaction Simulations
Jens Holm and Per Just Andersen, G M Idorn Consult A/S, Denmark

High-Performance Road-Surfacing Concrete with Good Resistance to Wear by Tyre Studs
Märten Nilsson, Swedish Society of Civil and Structural Engineers, Sweden

Maintenance and Repair of Highway Concrete Bridges: A Case Study
Ibrahim Al-Babtain and Adil M Abbas, Ministry of Communications, Kingdom of Saudi Arabia
OPENING

9.00 - 11.30

Chairman: Mrs Gunnel Färm, Director General, Swedish Road and Traffic Research Institute (VTI), Sweden

Opening Speeches
Mr Kjell A Mattsson, Governor of the Province of Gothenburg and Bohus, Sweden
Mrs Gunnel Färm, Director General, Swedish Road and Traffic Research Institute (VTI), Sweden

Research and the International Transportation Community
Dr C Michael Walton, Chairman, Executive Committee, Transportation Research Board, National Academy of Sciences and Engineering, USA

Transport Policies and Traffic Safety in an Integrated Europe
Dr Jan C Terlouw, Secretary General of the European Conference of Ministers of Transport (ECMT), France

Getting SHRP’s Products Into Practice
Dr Damian J Kulash, Executive Director, Strategic Highway Research Program (SHRP), USA

FHWA Role in SHRP Implementation
Mr E Dean Carlson, Executive Director, Federal Highway Administration, USA
(presented by Charles L Miller)

Recent European Initiatives in Research Collaboration
Mr David F Cornelius, Director, Transport and Road Research Laboratory (TRRL), United Kingdom
WEDNESDAY SEPTEMBER 18

ASPHALT

13.00 - 17.00

Chairman: Tord Lindahl, Swedish Road and Traffic Research Institute (VTI), Sweden

The Asphalt Model: Results of the SHRP Asphalt Research Program
D R Jones and T W Kennedy, University of Texas, Austin, Texas, USA

SHRP Asphalt-Aggregate Mix Analysis System
T W Kennedy and R J Cominsky, University of Texas, Austin, Texas; E T Harrigan and R B Leahy, Strategic Highway Research Program, Washington, USA

An Investigation of Asphalt-Aggregate Interaction and Their Sensitivity to Water
C W Curtis, L M Perry and C J Brennan, Auburn University, Auburn, Alabama, USA

Thermal Fatigue Cracking of Asphalt Concrete Pavements - An Experimental Approach
N W Jackson, T S Vinson, and V Janoo, Oregon State University, Corvallis, Oregon, USA

Development of Test Methods for a Performance-Related Bitumen Specification
D A Anderson, The Pennsylvania State University, USA

Characterization of Self Assemblies in Asphalt by NMR Spectroscopy and High Performance Gel Permeation Chromatography
P A Jennings, J A S Pribanic, T M Mendes, and J M Smith, Montana State University, Bozeman, Montana, USA

Asphalt Research in The Netherlands
P C Hopman, Delft University of Technology, Delft; P A J C Kunst, Netherlands Pavement Consultants bv, Hoevelaken; A C Pronk and J M M Molenaar, Roads and Hydraulic Department of Rijkswaterstaat, Delft, and A A A Molenaar, Delft University of Technology, Delft, The Netherlands

VTI RAPPORT 372A
WEDNESDAY SEPTEMBER 18

MOTORIST INFORMATION SYSTEMS

13.00 - 17.00

Chairman: Conrad Dudek, Texas A&M University, College Station, USA

Changes in Driver Behaviour as a Function of Handsfree Mobile Phones: A Simulator Study
Håkan Alm and Lena Nilsson, Swedish Road and Traffic Research Institute (VTI), Sweden

Variable-Message Signs: Legibility and Recognition of Symbols
Colomb, Huberg, Bry, Carta, Laboratoire Central des Ponts et Chaussees, Dore-Picard, Institute National de Recherche sur les Transport et leur Sécurité, France

The Man and His Wheel: Cognitive and Perceptual Factors
Marcel Wierda, Traffic Research Centre, The Netherlands

Measuring Effects of Variable Message Signing on Route-Choice and Driving Behavior
Richard van der Horst, Wiel Janssen and J E (Hans) Korteling, TNO Institute of Perception, Soesterberg, The Netherlands

Acceptance and Benefits of the Berlin Route Guidance and Information System (LISB)
Jürg M Sparmann, SNV Studiengesellschaft Nahverkehr mbH, Berlin, Germany

Automobile Navigation Safety Issues
Robert L French, R L French & Associates, Ft Worth, Texas, USA

(16.30-17.00 Short business meeting of TRB Committee A3B08, User Information Systems - visitors welcome)
WEDNESDAY SEPTEMBER 18

ACCIDENT STUDIES AND SAFETY MANAGEMENT

13.00 - 17.00

Chairman: Gunnar Carlsson, Swedish Road and Traffic Research Institute (VTI), Sweden

Economic Appraisal and Ranking of Road Safety Measures
Karl-Olov Hedman, Swedish Road and Traffic Research Institute (VTI), Sweden

Traffic Safety on Two Continents - A Ten-Year Analysis of Human and Vehicular Involvements
Rüdiger Lamm, University of Karlsruhe, Germany and Elias M Choueiri, North Country Community College, New York, USA

Description and Testing of a Side Impact Protection System
Jan Ivarsson, Volvo Car Corporation, Sweden

A Critical View of Traffic Safety Management in a Developing Country; A Case Study of Jordan
N M Katamine and M A Salem Kiyassat, University of Jordan, Jordan

Development of a Collision Topology for Evaluation of Collision Avoidance Strategies
Kenneth L Campbell, Daniel F Blower, Dawn L Massie, Patricia F Waller and Arthur C Wolfe, UMTRI, Ann Arbor, Michigan, USA

Comprehensive Safety Management
Michael S Collins, Ergotrans, United Kingdom

The Future of Road Traffic Management: Urgent Global Harmonization Will Affect All Governments
Arthur R Olin, Sweden

Implications of Litigation for Highway and Motor Vehicle Safety Research
P Robert Knaff, K B and Assoc., Silver Spring, MD, USA

The Impact of Litigation on the Federal Highway Administration’s Highway Safety Program
Steven E Wermcrantz, Federal Highway Administration, USA
THURSDAY SEPTEMBER 19

ROADSIDE SAFETY FEATURES

9.30 - 17.30

Chairman: Thomas Turbell, Swedish Road and Traffic Research Institute (VTI), Sweden, co-Chairman: Hayes E Ross, Texas Transportation Institute (TTI), USA

Roadside Safety - A Knowledge-based Approach
Abdelkrim Ramache, University of Newcastle Upon Tyne, United Kingdom

Safety Barriers Systems in Germany
Bernd Wolfgang Wink, Volkmann & Rossbach GmbH & Co KG, Germany

Side Impact Crash Testing of Highway Safety Hardware
John F Carney and Malcolm H Ray, Vanderbilt University, Nashville, USA

Safety Assessment of Highway Designs
Malcolm H Ray, Standard & Ray Assoc., Franklin, USA (presented by J F Carney)

The Importance of Using a Range of Vehicle Weights when Testing a Crash Cushion
Michael G Dreznes, Energy Absorption Systems Inc, Chicago, USA

Reliability of Results of Crash Testing Small and Medium Size Cars into Two Segmented Concrete Barriers
Francis P D Navin, University of British Columbia, Vancouver, Canada

13.00 Luncheon

Safe Road Design as Limit State
Francis P D Navin, University of British Columbia, Vancouver, Canada

Status of the United States Efforts in Promoting International Harmonization of Test and Evaluation Procedures for Roadside Safety Features
Harry W Taylor, FHWA, Washington DC, USA

Occupant Risk by Different Severity Criteria
Vittorio Giavotto, Politecnico di Milano, Milan, Italy

Hayes E Ross Jr, Texas Transportation Institute, Texas A&M University, USA

Status of the European Work on Harmonizing Requirements and Test Procedures for Roadside Safety Features
Jacques Boussuge, SETRA, France

WORKSHOP on International Harmonization
Status reports from the ongoing update of the US test procedures and the development of a European Standard within CEN
(This workshop will be followed up in non-public informal meeting between TRB committee A2A04(2) and CEN/TC226/WG1 on Friday morning)

VTI RAPPORT 372A
THURSDAY SEPTEMBER 19

HUMAN ENGINEERING TRAINING AND TRAFFIC SAFETY

9.30 - 13.00

Chairman: Alison Smiley, Human Factors North Inc, Toronto, Canada

Development of a Methodology for Measuring Improper Seat Belt Use
Brian A Grant, Road Safety Directorate, Transport Canada, Jocelyn Pedder and Nicholas Shewchenko, Biokinetics and Assoc. Ltd, Ottawa, Canada

Mandatory Hazard Perception Testing as a Means of Reducing Casualty Crashes Amongst Novice Drivers
Michael Hull and Peter Lowe, Vic Roads, Australia

Eye Scanning Rules for Drivers - How Do They Compare With Actual Observed Eye Scanning Behavior?
Helmut T Zwahlen, Ohio University, Athens, Ohio, USA

The Effects of Moderate Heat on Driver Vigilance in a Moving Vehicle
D P Wyon and F Norin, Volvo Car Corporation, Sweden

Position Accuracy When Pushing Pushbuttons in a Car as a Function of Car Speed and Location: Implications for Design
Helmut T Zwahlen, Nuruddin Abdullah and David Kellmeyer, Ohio University, Athens, Ohio, USA

(9.00-9.30 Short meeting of TRB Committee A3B02, Vehicle User Characteristics - visitors welcome)
THURSDAY SEPTEMBER 19

OPERATIONAL ROADWAY AND WORKZONE RESEARCH

14.00 - 17.30

Chairman: Karl-Olov Hedman, Swedish Road and Traffic Research Institute (VTI), Sweden

Overtaking Behaviour on Single Carriageway Roads in the United Kingdom
J G Hunt and T A Mahdi, School of Engineering, UWCC, Cardiff, United Kingdom

Overtaking Behaviour on Two-Lane Rural Roads
Arne Carlsson, Swedish Road and Traffic Research Institute (VTI), Sweden

Time and Space Criteria of Column Following
Milan Vujanic, University of Belgrade, Yugoslavia

Passing Operations on a Recreational Two-Lane, Tow-Way Highway
A R Kaub, University of South Florida, Tampa, USA

Reducing Risk Taking in Passing on Two Way Roads
Krsto Lipovac, Higher Shcool of Internal Affairs, Yugoslavia

Guidelines for Railroad Preemption at Signalized Intersections
Peter S Marshall, Barton-Aschman Ass Inc, Minneapolis, MN and William D Berg, University of Wisconsin-Madison, USA

VTI RAPPORT 372A
THURSDAY SEPTEMBER 19

SIMULATION AND MEASUREMENT OF OPERATOR AND VEHICLE PERFORMANCE

9.30 - 13.00

**Chairman:** R Wade Allen, Systems Technology Inc, USA

**Traffic Measurements by Means of Computer Vision Techniques**
N O Jørgensen, Institute of Roads, Transport & Town Planning, Denmark

**Dynamic 3-D Highway Modelling**
Arthur Roberts, NJDOT Research, Trenton, USA (Presented by R Pain)

**Validation of Real-Time Man-In-The-Loop Simulation**
R Wade Allen, David G Mitchell, Anthony C Stein and Jeffery R Hogue, Systems Technology Inc, Hawthorne, USA

**Measurement of Driver Performance in Training Simulators**
J E Korteling, TNO, The Netherlands

**Litigation and Driving Simulators**
Slade Hulbert, Ph D, Consultant, Danville, USA

STRATEGIES TO INCREASE THE USE OF RESTRAINT SYSTEMS

**WORKSHOP**

14.00 - 17.30

14.00 Opening

14.05 Illustration of background paper

14.15 National reports on seat belt use and countermeasures (10 minutes each)
   - Canada
   - Finland and other Nordic countries
   - France
   - Germany
   - Great Britain
   - Netherlands
   - United States

15.45 Coffee break

16.00 Discussion with speakers and audience

17.00 Concluding remarks and closure

(17.00-17.30 Short business meeting of TRB Committee A3B06, Simulation and Measurement of Operator and Vehicle Performance)

VTI RAPPORT 372A
THURSDAY SEPTEMBER 19

LONG-TERM PAVEMENT PERFORMANCE

9.30 - 17.30

Chairman: Hans Jørgen Ertman Larsen, Danish Road Institute, Denmark

Early Evaluations of SHRP LTPP Data and Planning for Sensitivity Analyses
J B Rauhut, Brent Rauhut Engineering, Austin, Texas; M I Darter, Eres Consultants Inc, Savory, Illinois; O Pendleton, Texas A&M University, College Station, Texas; and N F Hawks, Strategic Highway Research Program, Washington, USA

The Specific Pavement Studies: Key Issues and Potential Products
A N Hanna and N F Hawks, Strategic Highway Research Program, Washington, USA

Expected Changes to the AASHTO Design Guide
N F Hawks, Strategic Highway Research Program, Washington, USA

Cost Effectiveness of Asphalt Concrete Overlays - The Canadian Approach
G A Sparks, Clayton, Sparks & Ass Ltd, Saskatoon, Canada; D M Nesbitt, Decision Focus Inc, Los Altos, California; and G Williams, Roads and Transportation Association of Canada, Ottawa, Canada

Long Term Pavement Performance Trials and Data Analysis in The United Kingdom
H R Kerali, University of Birmingham and J F Potter, Transport and Road Research Laboratory, United Kingdom

SHRP-NL: A Research Project Parallel to SHRP
G T H Sweere, SHRP-NL, Delft, The Netherlands

Structural Assessment, Performance and Economic Maintenance of Minor Roads
J Roger Duffell, The Hatfield Polytechnic, United Kingdom

Treatment of Bearing Capacity Results
B Leben and A Petkovsek, Institute for Geotechnic and Roads, Ljubljana, Yugoslavia

A Model of IRI for Jointed Plain Concrete Pavements
P Ceza, J David, J Gonzalez and M Poblete, IDIEM, University of Chile; and P Gutierrez, National Highway Administration, Chile

The High Speed Road Deflection Meter
P W Arnberg and G Magnusson, Swedish Road and Traffic Research Institute (VTI), Sweden

PAVUE: A Real-Time Pavement Distress Analyzer
M W Burke and K Råhs, OPQ Systems AB, Linköping; and P W Arnberg, Swedish Road and Traffic Research Institute (VTI), Sweden
FRIDAY SEPTEMBER 20

SAFETY AND MOBILITY OF OLDER DRIVERS

8.30 - 12.30

**Chairman:** John Eberhard, TRB Task Force on Safety and Mobility of Older Drivers, USA

**Old Hands on the Wheel: Exposure, Accident Experience and Problems of Elderly Drivers**
M L Chipman, C G MacGregor, A M Smiley, University of Toronto, M E H Lee-Gosselin, Universite Laval, Quebec, and L Clifford, Ministry of Transportation, Toronto, Canada

**More Safety Thanks to Good Orientation – Nothing Works Without Traffic Signs**
Henriette Reinsberg, 3M Germany, Germany

**Elderly People and Mobile Telephone Use – Effects of Driver Behaviour?**
Lena Nilsson and Hakan Alm, VTI, Sweden

**Driving Performance in Mild Senile Dementia of the Alzheimer Type (SDAT)**
Linda Hunt, Dorothy Edwards, John C Morris and Ada Mui, Irene Walter Johnson Rehabilitation Institute at Washington University Medical Center, St Louis, USA

**Discussant:**
Robin Barr, National Institute on Aging, US Department of Health and Human Services, Bethesda, Maryland, USA

**SYMPOSIUM SESSION:**

**VISUAL AND COGNITIVE CAPABILITIES IN OLDER DRIVERS: PREDICTING ACCIDENT RISKS**

**Visual Function and Eye Health: Their Relationship to Older Driver Problems**
Michael Sloane, University of Alabama at Birmingham, USA

**Attentional and Cognitive Factors in Predicting Older Driver Problems**
Karlene Ball, Western Kentucky University, Bowling Green, USA

**Attention and Driving Performance in Alzheimer’s Dementia**
Raja Parasuraman, Catholic University of America, Washington, USA

**Older Drivers Handling Road Traffic Informatics: Divided Attention in a Dynamic Driving Simulator**
Peter C van Wolffelaar, Wiebo H Brouwer and Talib Rothengatter, Traffic Research Centre, University of Groningen, The Netherlands

**Discussant:**
Harvey Sterns, Institute for Life-Span Development and Gerontology, University of Akron, Ohio, USA
SAFETY AND MOBILITY OF OLDER DRIVERS

13.30 - 16.00

PANEL DISCUSSION:
FEASIBILITY OF INTERNATIONAL PERFORMANCE STANDARDS FOR OLDER DRIVERS

Presiding Officer: John Eberhard, Chairperson, TRB Task Force on Safety and Mobility of Older Drivers, USA

1. A USA Perspective
   Robin Barr, National Institute of Aging, Bethesda, MD, USA

2. A European Community Perspective
   Margaret Greico, Oxford University, United Kingdom
   Kay Axhausen, Imperial College of Science, Technology and Medicine, London, United Kingdom

3. A Scandinavian Perspective: Older Drivers - A Problem for Whom?
   Krister Spolander, Central Bureau of Statistics (SCB), Sweden

4. A Multi-continent Perspective
   Martin Lee-Gosselin, Université Laval Quebec, Canada

Discussion: Invited from prior presenters and all session attendees

(16.00-16.30 Short meeting of TRB Task Force A3T52, Safety and Mobility of Older Drivers, visitors welcome)
FRIDAY SEPTEMBER 20

HIGHWAY OPERATIONS AND CONCRETE AND STRUCTURES

8.30 - 12.30

Chairman: Torkild Thurmann-Moe, Road Research laboratory, Norway

Closed Track Testing of Maintenance Work Zone Safety Devices
S C Shah, Strategic Highway Research Program, Washington and F R Hanscom, Transportation Research Corporation, Haymarket, Virginia, USA

Innovative Materials for Pavement Surface Repairs: Field Installation and Evaluation
S C Shah, Strategic Highway Research Program, Washington, USA

MINSALT - A 5-Year Study to Minimize the Negative Effects of Salt
Kent Gustafson and Gudrun Öberg, Swedish Road and Traffic Research Institute (VTI), Linköping, Sweden

Deicing Salt - Its Use and Effect on Road Safety and the Living Conditions of Roadside Trees and Shrubs
Siegfried Giesa, Technical University of Darmstadt, Germany

Improving Concrete Pavements Through SHRP Research
Amir N Hanna, Strategic Highway Research Program, Washington, USA

Optimization of Highway Concrete Through Combined Use of Particle Packing Modelling, Rheological Studies, Computer Simulations and Compaction Simulations
J Holm and P J Andersen, G M Idorn Consult A/S, Birkerød, Denmark

High Performance Road-Surfacing Concrete with Good Resistance to Wear by Tyre Studs
Mårten Nilsson, Swedish Road Administration, Sweden

Maintenance and Repair of Highway Concrete Bridges: A Case Study
I Al-Babatain and A M Abbas, Ministry of Communications, Riyadh, Saudi Arabia
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<td>Ministry of Communications</td>
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The Asphalt Model:
Results of the SHRP Asphalt Research Program

David R Jones, IV
Ph D
Center for Transportation Research
University of Texas at Austin
U S A

and

Thomas W Kennedy
Ph D, P E
Center for Transportation Research
University of Texas at Austin
U S A
ABSTRACT:

The Asphalt Model:
Results of the SHRP Asphalt Research Program

David R. Jones, IV Ph.D
Thomas W. Kennedy, Ph.D, P.E.
Center for Transportation Research
University of Texas at Austin
Austin, Texas, USA

By the time of this conference, the Asphalt Research Program will have been underway for almost four years. The major asphalt research contracts, located at The Western Research Institute in Laramie, Wyoming; Montana State University at Bozeman; The University of Southern California at Los Angeles; Auburn University in Auburn, Alabama; and SouthWestern Laboratories in Houston, Texas have made significant strides toward understanding the relationship between asphalt's chemistry and its performance on the roadway.

This paper will outline the progress to date, and present the current understanding of the chemistry-performance link.

Due to the complex and highly varied chemical makeup of paving asphalts, the relationship between asphalt chemistry and performance is difficult to state simply. In addition to its difficult chemistry, it is known that the combination of binder, aggregate, construction practices, and service environment will control the ultimate performance of the pavement. In spite of these difficulties, the SHRP researchers have made significant progress in elucidating the fundamental relationships between the composition of the asphalt binder and its ultimate performance.

This progress has been accomplished through the use of an asphalt "model," a set of working hypotheses which describe the predicted behavior of the asphalt due to its chemical constituents. Based on historical data gathered prior to the SHRP program, a "micellar" model was proposed, and used as the hypothesis upon which the experimental designs were based. As the research progressed the original model failed to adequately explain the findings, and it was reexamined and modified. Results were then analyzed in light of the new model, and experimental designs modified to take into account the new data being generated. This iterative process has led us to our current understanding of the relationship between asphalt chemistry and field performance.

Early in the research it was realized that the molecular makeup of the asphalt, rather than its detailed atomic structure, was the best key to understanding the properties of the binder. These "associations" of asphalt molecules have been shown to control most of the performance properties of the asphalt, although several chemical species also have an active role in performance, the details of which will be the subject of this paper.
1. INTRODUCTION -

Of the $150 million USD pledged for the Strategic Highway Research Program (SHRP), approximately one-third ($50 million USD) is designated for Asphalt Binder Research. The SHRP program is a five-year, highly focused, product-oriented research effort. The goal of the asphalt portion of the program is to issue a new binder and mixture specification that will help federal, state and local regulatory and specifying agencies obtain better performing hot-mix asphalt concrete (HMAC) pavements.

By the time this paper is presented, the SHRP Asphalt Research Program will have been underway for almost four years, and will be well on its way toward delivery of the end-product specifications. With just over a year left in the contract, many of the researchers will be in the midst of summarizing their research results, and draft binder and mixture specifications will have been issued for public comment and input from both users and producers of asphaltic paving products.

The SHRP Asphalt Research Program encompasses a wide variety of experimental approaches. All are focused on furthering understanding of how the chemistry of the binder affects the properties of the mix and the resultant performance of the flexible pavement. These research efforts are primarily aimed at the major areas of poor performance affecting roadways in the United States.

When the SHRP program was being organized in 1986 and the goals and objectives were being decided upon, the "Brown Book" (1) identified six areas of pavement distress that were to be the focus of SHRP's Asphalt Research Program. They were:

- Low-Temperature Cracking
- Fatigue Cracking
- Permanent Deformation
- Aging
- Moisture Damage
- Adhesion
The program addresses these distress areas with a variety of fundamental and practical approaches. The unique part of the SHRP Asphalt Research Program is the practical, goal-oriented emphasis of the research and its focus on generating the required products, namely a binder and mixture specification.

The major tenant of the asphalt portion of the SHRP program was the use of an asphalt model as the basis for the experimental plan. The major asphalt research contract was awarded in 1987 to Western Research Institute (WRI) in Laramie, Wyoming. WRI used the micellar model (Figure 1), which has since been discredited, as a starting point for its experimental designs.

**Figure 1**
The Micellar Model of Asphalt

![Diagram of the micellar model of asphalt with fractions and oxidation processes.]

At the time the SHRP program began the micellar model was accepted as providing the best available explanation of how asphalts structured themselves. In the authors' opinion the most significant result of the five-year asphalt portion of the SHRP program is the entirely new understanding of asphalt chemistry that has been achieved and the resultant asphalt model that has been developed to explain asphalt's performance. This new fundamental explanation of how and why asphalts behave as viscoelastic materials will enable highway engineers to design pavement structures that take full advantage of asphalt's chemical and physical properties. It will also enable refiners and manufacturers to modify and beneficiate asphalts in a rational manner, which has never been possible before.
2.  THE ASPHALT MODEL: HISTORICAL PERSPECTIVE

The original micellar model had been constructed from interpretations of historical data in which researchers looked for relationships between chemical parameters and pavement performance. Other surrogate measurements, such as penetration and viscosity, and mathematical permutations of these constants, such as penetration index (PI), penetration-viscosity number (PVN), and aging index (AI) were used to discover or define the chemistry-property-performance relationship for asphalts. Often stirring impassioned debate by dedicated scientists and engineers, the arguments and discussion have gone on for many years.

As a result of the many man-years of effort dedicated by the SHRP contractors, a clearer picture of the relationship between asphalt chemistry and pavement performance is evolving rapidly. Several very powerful analytical techniques have been applied to asphalts for the first time, and they are yielding startling results. These new chemical results, in combination with an exhaustive examination of the rheological properties of asphalts, lead us to the present understanding of the chemistry-property-performance relationships of asphalts, and the definition of a new asphalt model. This model will allow, for the first time, to predict pavement performance based on asphalt chemistry.

Having stated that the results of the SHRP program will give us the tools to predict pavement performance based on asphalt chemistry, it is prudent to add several qualifiers. Pavements are composites of multiple materials. Both the asphalt and aggregate will contribute to pavement performance, but construction practices, service environment, traffic loading and duration, and base characteristics all play an important role in the ultimate performance of the pavement. For the purposes of this paper the authors will assume that there are no confounding effects from these additional variables when considering the linkage between chemistry and performance.

3.  ANALYTICAL METHODS AND THE MODEL

In reality, the historical model was defined and limited by the analytical tools applied to asphalt. Without better definition of the constituents of asphalts, researchers were limited in determining the chemical relationships that govern performance. Separation methods such as those developed by Corbett and Rossler gave data that were ambiguous, and there were always asphalts that were exceptions to the general trends indicated by these methods. In a similar vein, surrogate measurements such as penetration and viscosity didn't correlate with
performance because they were not specific enough to define a desired physical trait. It was with these limitations in mind that analytical chemistry was applied to asphalts in the (then) new SHRP program.

3.1 **Size-Exclusion Chromatography**

One of the first methods to be brought to bear was size-exclusion chromatography (SEC). At both Montana State University and WRI new SEC techniques were applied to asphalts to follow up on earlier work by Jennings and Pribanic (2). The method was refined by WRI to use toluene as the solvent, which caused less disruption to the asphalt structure than the original tetrahydrofuran. This was accomplished on a preparative scale, and for the first time allowed subsequent chemical analysis and characterization of separated SEC fractions.

The SEC technique evolved as the SHRP program progressed, and it finally resulted in two fractions (SEC I and SEC II) being isolated from the asphalts. SEC I has been given the title "associated phase," and SEC II is labeled the "solvent phase." While in principle SEC separates the asphalt molecules based on their hydrodynamic volume or molecular size, additional analyses have conclusively demonstrated that the majority of the polar materials present in the native asphalts are found in SEC I, and the non-polar materials are in the SEC II fraction. While many researchers have long felt that the polar materials in asphalts played a major role in pavement performance, they had been unable to demonstrate it conclusively. The adoption of SEC was a first key to proving that relationship. Examination of the SEC II "solvent phase" has also led to some interesting conclusions, and has allowed SHRP to establish the first chemistry-performance relationships, which are illustrated in Figure 2.
Analysis of SEC II fractions of many asphalts showed conclusively that this fraction consisted of non-polar molecules that played two critical roles in asphalt:

1. The non-polar "solvent" makes a critical contribution to the low-temperature properties of the pavement. In fact, low-temperature cracking in pavements is virtually independent of any other variable in the pavement system. At low temperatures the non-polar asphalt molecules align and order themselves, and there is a free-volume collapse in the pavement. This collapse takes place without crystallization and will cause thermal cracking if too severe. It is primarily a function of the molecular weight of SEC II, but chain branching in the molecules is also important, hindering collapse and retarding low-temperature cracking.

2. The second contribution of the SEC II fraction to pavement properties is due to its role as a solvent for the polar materials. It is important to note that there is no solvent "phase" per se in asphalt. Asphalt is a single-phase, complex mixture of molecules. There are no micelles, no networks, and no floating "islands" of materials. However, while there are no true phases in asphalts, there is a mixture of molecules of widely varying polarity and molecular weight, and the materials in this mixture interact with one
another to form what the authors will call associations. These associations form among the polar molecules in the asphalt, which create weak electrostatic bonds between polar sites on the molecules. These associations take place within the non-polar portion of the asphalt (SEC II fraction), and the molecular weight distribution and amount of non-polar materials affect the associations of the polar materials.

As a result of nearly four years of intensive research we now know that the polar molecules in the asphalt form weakly linked associations within the bitumen, and these associations are affected by the non-polar molecules surrounding them. Figure 3 attempts to display this new model of asphalt structure graphically.

This figure illustrates the authors' concept of the asphalt structure as it is now understood. The associations of the polar molecules (shown in this drawing as the different shapes) take place at the polar sites of the asphalt molecules through electrostatic forces or hydrogen bonding, and other interactions take place to a lesser extent through pi-pi bonding of aromatic rings and Van der Waals interactions of long-chain hydrocarbons. While these forms of interaction take place simultaneously, the majority of the viscoelastic properties of asphalts are a result of the polar-polar interactions of the molecules. The evidence which led to this conclusion is discussed below.
This new model has gone by several names during its development, and several SHRP researchers have independently come to the same conclusions when interpreting the data generated by the asphalt research portion of the SHRP program. The model has, at various times, been called the "spider," "spaghetti and sauce," "polar-dispersed fluid," and "microstructural" model, among others. It is the authors' opinion that the terminology of "microstructural model" perhaps is the most descriptive, and should be adopted by the industry.

It is important to remember that all of these interactions between asphalt molecules are weak, and the bonds may be broken through the action of heat or shear forces. This concept of weak interactions between the molecules explains why asphalt behaves as a Newtonian fluid at elevated temperatures, and also explains why asphalt exhibits constantly changing behavior. Due to the weak nature of the polar-polar bonds, the bonds are constantly being broken and reformed, each time in a unique way that never yields quite the same material.

Perhaps the most persuasive demonstration of this is shown when the asphalt is dissolved in solvent. Even if oxygen is rigidly excluded and loss of volatiles is carefully controlled, the physical properties of the reconstituted asphalt are never the same as the original material. This is because the solvent breaks the weak polar-polar bonds, and when the solvent is removed the bonds reform in a unique new pattern, yielding a "new" asphalt and a new set of physical properties. It is critical to be aware of this unique property of asphalt when examining its physical properties, especially in the case of solvent-recovered asphalts from pavements.

3.2 Ion Exchange Chromatography

Ion Exchange Chromatography (IEC) is the second powerful analytical technique that has been applied to asphalt to help understand its chemistry-physical property-performance relationships. The seminal work by Green, et al. (3) was modified by the researchers at WRI to tailor the method to asphalts. As discussed in an earlier paper (4), for the first time researchers have an analytical method that allows the polar molecules in the asphalt to be separated by polarity. Earlier methods such as Corbett and Rossler separations did not depend solely on polarity for the separation of molecular species. The authors believe the failure of these analytical methods to separate discreet chemical fractions is demonstrated by the lack of correlation between single-separated fractions and asphalt performance. The use of IEC to separate asphalt into fractions based solely on their polar nature makes the technique unique in providing a means of unravelling the chemistry-performance link. It also strengthens the validity of the asphalt model.
IEC has been used to generate a unique new fraction of asphalt that is believed to be its "key" building block. These materials have unique physical properties that explain the tendency of polar materials in asphalt to agglomerate and form associations. They have been named "Amphoterics."

4. THE AMPHOTERICS

The Webster's New Collegiate Dictionary defines an amphoteric material as one that can exhibit either acidic or basic character. In the case of asphalt the term is used to mean that an asphalt molecule has both an acid and base group on the same molecule. Preliminary data from WRI (4) have provided strong circumstantial evidence for the importance of the role the amphoterics play in building the polar-polar bonds that give asphalt its unique properties. Additional experiments to verify and expand our understanding of the amphoteric molecules are underway as this paper is written. The preliminary data suggest that the presence of two or more functional groups on the asphalt molecule make it capable of forming "chains" of weak polar-polar interactions, as illustrated in figure 4.

Figure 4
Chain Building in Asphalts

It can also be seen in this illustration that molecules containing single functional groups would serve as chain terminators or "capping" agents, and would prevent the building of long networks of associated molecules. These two tendencies can be used to explain several of the behavioral traits of asphalts as they age.
5. AMPHOTERICS AND ASPHALT AGING

As asphalts age they incorporate oxygen into molecules where there are heteroatoms (N,O,S) or where there are chemically active carbon atoms such as benzylic carbon. These sites may well be non-polar in the unaged or tank asphalt. Aliphatic sulfur will be non-polar, but following oxidation the resultant sulfoxide will be weakly basic, and can participate in the polar-polar associations. Similarly, benzylic carbons, which are non-polar, can be oxidized to carbonyl, which can take part in hydrogen bonding and participate in the aged asphalt as a polar site.

But it has long been shown in the literature that there is no correlation between asphalt properties such as aging index (AI) and sulfoxide and/or carbonyl content. Asphalts such as SHRP asphalt AAG (California Valley) produce large amounts of carbonyl, but don't gain 140 F viscosity rapidly when oxidized. Conversely, asphalts such as AAK (Boscan) don't form much carbonyl, but have large 140 F viscosity changes upon aging.

The new asphalt model allows to explain this previously anomalous behavior. The lower amounts of amphoterics in AAG mean that most of the molecules in AAG only have one oxidizable site, and hence cannot form many associations. Conversely, AAK has a higher level of amphoterics, and thus can build networks of associations and viscosity as it is oxidized. This model explains the lack of correlation between oxidation products and aging behavior. It has also led us to look for the presence of additional oxidation products that may contribute to aging properties, but may have not been measured by classical analytical techniques applied to aged asphalts such as infra-red spectroscopy (IR).

A series of experiments to determine the oxygen balance of asphalts as they age is currently underway at SRI in Palo Alto, California. This research will assure that no unknown pathways play a role in the oxidative behavior of asphalts. Additional experiments are also underway at a number of institutions to determine the role of aggregate in the aging process.

The lack of agreement between the AI of asphalts and their chemical composition can also be interpreted in another important way. If we assume that analytical techniques such as IR have accounted for all the oxidation products that occur in asphalt, then we must conclude that the aging index is not measuring a property that has anything to do with pavement performance. This argument is made more compelling when one realizes that the AAK asphalt, which has a high AI, is historically seen as a good field performer and is not susceptible to the premature field aging effects its high AI would indicate.
The measurement of viscosity at 140 F only defines one point on a continuous curve. Therefore we are not defining performance. Some of the other empirical indices such as PI and PVN have used more than one temperature to try to anchor this curve. The final SHRP asphalt specification fully addresses this problem by measuring and defining the master rheological curve for the aged and unaged asphalt over the full range of its service temperatures. Only in this way can we fully define asphalt's performance, and only with this knowledge base can we understand the interactions between the different chemical species affecting the rheological properties of asphalt.

6. CONCLUSIONS

Through the efforts of dozens of researchers at many different institutions we have developed a new model to explain asphalt's chemistry-physical property-performance relationships. This new microstructural model of asphalt has made the micellar model obsolete and explains many of the anomalies present in earlier asphalt models.

Asphalt is a single-phase homogeneous mixture of many different molecules, which may be differentiated into two broad classes: polar and non-polar. The non-polar molecules serve as a matrix or solvent for the polar molecules, which form weak "networks" of polar-polar associations that give asphalt its elastic properties. There are no micelles or "cores" of asphaltenes in asphalt. The polar materials are uniformly distributed throughout the asphalt, and upon heating the weak interactions are broken to yield a Newtonian fluid. When perturbed these interactions break and reform to produce a new combination of interactions that gives a "new" asphalt.

"Good" asphalts have a proper balance of polar and non-polar molecules. The true molecular weight of the non-polar molecules is also important in asphalt performance, especially in low-temperature performance. Asphalts that have too much polar material will be subject to fatigue cracking in thin pavements, brittleness, and thermal cracking. Asphalts that have too much non-polar material, or asphalts in which the non-polars are too low in molecular weight, will suffer from fatigue cracking in thick pavements, moisture sensitivity, and rutting.

The objective of the SHRP Asphalt Research Program in its final months is to continue to determine and expand our understanding of the link between asphalt chemistry and pavement performance. Much has been accomplished; much remains to be done.
7. REFERENCES

1. Transportation Research Board, Strategic Highway Research Program Research Plans, NCHRP, 1986


SHRP Asphalt—Aggregate Mix Analysis System (AAMAS)

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The principal goal of the SHRP asphalt research program is to develop performance-based specifications for asphalt binders and asphalt-aggregate mixes. These specifications will allow the engineer to select an asphalt binder and asphalt-aggregate mixture on the basis of the performance level required of the pavement under the present and predicted traffic and environmental conditions.

The performance-based specification for asphalt-aggregate mixes will require methods or protocols for the following: 1) mixing and compaction of laboratory specimens that realistically simulate the effects of plant mixing and field compaction; 2) selection and proportioning of materials to obtain a job mix formula that provides the required pavement performance level in view of the expected environmental conditions and traffic volume; and 3) evaluation of modifiers or modification techniques if it is determined that unmodified mixes cannot meet the performance demands. The preceding represents an asphalt-aggregate mix analysis system (AAMAS) that, when used in conjunction with the performance-based specifications and accelerated test methods, allow a rational selection of materials for a mix compactible over a wide range of temperatures which is resistant to permanent deformation, fatigue and thermal cracking, aging and moisture damage.

Initially, SHRP will use some of the AAMAS concepts identified by NCHRP 9-6-(1) and build upon these as SHRP research findings support inclusion of more profound technical advancements related to materials selection; specimen size, configuration, compaction and conditioning; and supportive tests. In its current format the AAMAS logically proceeds with consideration of the following: asphalt selection; asphalt-aggregate compatibility; initial proportioning and trial mix design; specimen fabrication; volumetric analysis; evaluation of fundamental engineering properties; and comparison to structural design requirements.
SHRP's ASPHALT-AGGREGATE MIX ANALYSIS SYSTEM (AAMAS)
Rita B Leahy
Senior Staff Engineer
SHRP

1. INTRODUCTION

The sole purpose of present day mix design methods is to determine acceptable asphalt contents for paving mixes. Mixture properties such as stability, flow and air void content obtained from those methods have only an intuitive link with pavement performance. It is not possible, for example, to specify a minimum Marshall or Hveem stability that will eliminate the possibility of permanent deformation (rutting) on a high volume, urban freeway in a hot, dry climate.

SHRP's strategy is to develop a performance-based specification for asphalt-aggregate mixtures, the AAMAS, and supportive tests based upon fundamental engineering properties of the mix from which reasonable estimates of pavement performance can be computed. Within specified environmental regimes and estimated traffic loadings, the specification will address the following characteristics related to performance: permanent deformation, fatigue, low-temperature and thermal cracking; aging, water sensitivity); and constructability. The performance-based specification should allow selection of an optimal job-mix formula that will provide for satisfactory pavement performance by accommodating a wide range of environmental, construction and traffic loading conditions.

The AAMAS supportive tests will be correlated and validated with extensive field performance data by mathematical models and rigorous statistical design procedures. By this very strategy, the nature of the tests and mix design procedures will be more complex. However, the advantage of this approach is that the performance-based mix specifications and the AAMAS will allow engineers to choose those materials that will be most cost effective given specific traffic volume and environmental conditions.

Initially, SHRP will employ some of the concepts identified in NCHRP Report # 338 (1) and build upon these as the research findings support inclusion of the profound technical advancements and refinements related to material selection, mix design, and the supporting tests. Table I presents a comparison of the laboratory test systems identified in support of the NCHRP, SHRP Level I and SHRP Level II AAMAS. The NCHRP tests were intuitively adopted to evaluate mixtures that individually or interactively have an influence on pavement performance but were not necessarily validated with extensive field performance data as only five projects were used to correlate laboratory and field results. Accelerated laboratory tests that relate to field performance will be utilized in SHRP's Level I and Level II AAMAS and will be correlated with short-term or accelerated-simulated field performance and ultimately with data accumulated in the long-term pavement performance (LTPP) program.

Fig. 1. Concept for the AAMAS

2. SHRP LEVEL I AAMAS

2.1 Aggregate Selection

Aggregate selection will consider both natural and manufactured characteristics such as those shown below.

<table>
<thead>
<tr>
<th>Natural Characteristics</th>
<th>Manufactured Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>absorption</td>
<td>gradation</td>
</tr>
<tr>
<td>durability</td>
<td>shape (crushed faces)</td>
</tr>
<tr>
<td>wettability</td>
<td>filler (passing No 200 sieve)</td>
</tr>
</tbody>
</table>

Recognizing the importance of gradation, it is likely that SHRP will establish guidelines for voids in the mineral aggregate (VMA) as well as use of the FHWA 0.45 power gradation curve relationship.

2.2 Asphalt Selection

Initial asphalt selection will be made in accordance with SHRP's Asphalt Binder Performance-Based Specification. This specification addresses the selection of the asphalt grade that best meets the particular requirements dictated by traffic and environmental conditions. The grading is based upon the properties of asphalt aged to simulate a specific pavement service period. The grading primarily controls the response of the asphalt to factors that cause low-temperature cracking and permanent deformation. Use is made of rational performance indices that are established considering exposure of the pavement to temperatures ranging from less than -20°F to greater than 100°F. Thus, a precise grade can be selected based upon the laboratory tests to accommodate the need to control low-temperature cracking, permanent deformation, or both, in a particular construction project. Comparison of laboratory test results to specification values for fatigue cracking and water sensitivity/adhesion may also be considered in asphalt selection.

2.3 Aggregate-Asphalt Compatibility

The net adsorption test will be used to determine the adsorptive nature and water sensitivity of different siliceous aggregate. The test provides a quantitative measure of the degree to which asphalt adheres to aggregate after exposure to water. Conducted in three steps asphalt is first adsorbed onto aggregate by means of a toluene solution in a recirculating column. Water is then circulated through the column, possibly desorbing asphalt from the aggregate. The asphalt remaining on the aggregate after water circulation is indicative of the net adsorption.

2.4 Initial Proportioning and Trial Mix Design
Figure 1. Framework for an Asphalt-Aggregate Mixture Analysis System (AAMAS)
<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>SHRPE LEVEL II</th>
<th>SHRPE LEVEL I</th>
<th>NCIRP 9-6 (1)</th>
<th>Diametral stiffness (2.5 x 4 φ)</th>
<th>Axial creep (4 φ x 8)</th>
<th>Indirect tensile strength and strain at break (2.5 x 4 φ)</th>
<th>Relative cost (acceptability)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fatigue</td>
<td>Flexural beam</td>
<td>(1.5 x 1.5 x 15)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$220,000</td>
</tr>
<tr>
<td>Permanent Deformation</td>
<td>Cyclic shear</td>
<td>(2.5 x 4 φ)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Low/Intermediate</td>
</tr>
<tr>
<td>Thermal Cracking</td>
<td>Thermal stress and strain at break plus stiffness (2.5 x 4 φ)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>High</td>
</tr>
<tr>
<td>Aging</td>
<td>1) Short-term</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>High $100,000-150,000</td>
</tr>
<tr>
<td></td>
<td>2) Long-term</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Low/Intermediate $150,000-200,000</td>
</tr>
<tr>
<td>Water Sensitivity</td>
<td>Water conditioning and freeze/thaw cycling under vacuum and load</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Stiffness</td>
<td>Cyclic shear modulus (2.5 x 4 φ)</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Laboratory Compaction</td>
<td>Gyratory (Texas)</td>
<td>(4 x 8 φ)</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Acceptability</td>
<td>Intermediate/high</td>
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</tbody>
</table>

| Table 1: AAMAS Test Systems |
Initial selection of the aggregate gradation will be reasonably close to the FHWA 0.45 power gradation curve. Guidelines for VMA will be based on the nominal maximum size of aggregate. The amount of filler (passing No 200 sieve) should be limited to 3 to 7 percent by weight of aggregate. Target air void values should range between 3 and 5 percent. Asphalt contents will include a target value based on the aggregate grading and acceptable VMA and air void ranges. Two asphalt contents above and below the initial target value will be considered in the initial trial mix design. To simulate short-term aging (plant mixing) the loose mix will be conditioned using a modified rolling thin oven (RTO). This short-term age conditioning is discussed in the section on mix conditioning procedures.

2.5 Test Specimen Fabrication (Compaction)

Initially, SHRP views compacting specimens at each asphalt content using the Texas Gyratory Shear Compactor. The specimens will be compacted to refusal density at each asphalt content. After compaction, density measurements will be made by gravimetric means.

2.6 Volumetric Analysis

From the gravimetric density measurement, values of the VMA and air voids will be determined. These calculated values must be within the allowable VMA and air void specification ranges in order to establish the design asphalt content. Figure 2 illustrates the concepts graphically. If the calculated values are not within the allowable specification limits, the trial mix must be redesigned. Compacted specimens will be subjected to additional age conditioning, to simulate long-term aging, and moisture conditioning as outlined in the following section.

2.7 Mixture Conditioning Procedures

2.7.1 Aging

Two aging tests will be utilized. The first simulates the short-term aging which occurs during plant mixing and construction; the second, the aging that occurs during five to ten years of service.

1. Mixture Rolling Thin Film Oven Test

The aging that occurs during mixing and construction will be simulated by using a modification of the RTO test. This modification involves placing a cylinder containing loose asphalt mix on the shaft of the rotating element and subjecting the mix to a specified time and temperature. Another possibility, as suggested by NCHRP Report # 338, is the use of a forced draft oven in which the loose mix is held at a temperature of 275°F for 4 hours.

2. Modified Triaxial Aging Test

To simulate long-term aging (5 to 10 years) the compacted mix specimens are placed in a triaxial cell and air is pumped through the cell under controlled temperature and pressure conditions. An alternative procedure is that recommended by NCHRP Report # 338: loose mix initially held in a forced draft oven at a temperature of 140°F for 48 hours, then at 225°F for 120 hours.

2.7.2 Moisture Sensitivity

A single moisture sensitivity conditioning procedure will be utilized. This test procedure involves subjecting the compacted specimens to pressure and moisture at elevated temperature. The test apparatus is a triaxial compression type cell. A minimum retained stiffness and/or tensile strength will be specified. Additionally, a permeability measurement may be required.

2.8 Mechanical Tests Evaluating Fundamental Engineering Properties of the Mix

If the volumetric proportions, short-term and long-term aging and water sensitivity results are within specification requirements, the compacted specimens will then be subjected to a series of tests to evaluate the fundamental engineering properties of the mix related to the following: permanent deformation; fatigue; low-temperature cracking and stiffness. Figure 3 illustrates graphically the concepts involved with the analysis of fundamental engineering properties. The specific performance-based tests are as follows.

2.8.1 Low-Temperature Cracking

Low-temperature cracking results when the tensile stresses, caused by a drop in temperature, exceed the mixture's fracture strength. The indirect tensile test with cylindrical specimens (2.5 in x 4.0 in diameter) will be used to determine tensile strength and strain at break, as well as stiffness. A possible alternative, if a demonstrated advantage is shown, would be a thermal stress restrained specimen test. This test involves the use of a beam specimen, restrained at its ends to prevent movement, while subjecting it to monotonic or cyclic cooling.

2.8.2 Permanent Deformation

The test method utilized to evaluate permanent deformation characteristics is the confined axial compression-shear test. This test involves cylindrical specimens (2.5 in x 4.0 in diameter) which are subjected to a confining pressure. During testing, the specimen is subjected to a vertical axial stress and a repeated shear stress. The shear stress is applied for 1000 cycles and the accumulation of permanent strain is measured.

2.8.3 Fatigue
1.) Initial design asphalt content from (a) and (b).

\[ A_0 = \frac{A_1 + A_2}{2} \]

2.) Check design asphalt content with VMA requirements.

Figure 2. Volumetric proportioning analysis concepts to establish design asphalt content.
Figure 3. Conceptual Analysis of Fundamental Engineering Properties to Establish Design Asphalt Content.
Because fatigue failure usually is caused by repetitions of tensile stresses and strains, it is logical that a relatively simple apparatus should provide for testing in a tensile mode. The specification must consider both controlled (or constant) stress and controlled (or constant) strain which simulate thick and thin layers, respectively. The repeated-load indirect tensile test using cylindrical specimens (2.5 in x 4.0 in diameter) will be used to characterize fatigue properties for both thick and thin pavement sections. Application of a haversine or other suitable waveform load at a preselected frequency of 0.33, 0.50 or 1.0 Hz for a minimum of 50 to 200 load repetitions is proposed. A possible alternative, if a demonstrated advantage is shown, will be the bending beam test. The test would be conducted on beam specimens (2 in x 2 in x 16). Testing would involve four point loading using a sinusoidal load pulse at a relative high load frequency (e.g. 20 Hz). Testing would be conducted to failure or to a specified number of load repetitions (e.g. 50,000) at which time measurements would be made to allow stiffness and strain to be estimated relative to the fatigue characteristics of the mixture.

2.8.4 Stiffness

The stiffness of asphalt concrete is a critical parameter in determining pavement performance and is considered a fundamental property essential for the analysis of pavement response to traffic loading. The AAMAS must consider the determination of dynamic modulus and phase angle over a range of frequencies (0.01 to 20 Hz) and temperatures (0 to 80°C). These tests would be performed using cylindrical specimens (2.5 in x 4.0 in diameter) under confined pressure.

In summary, the Level I approach is to test cylindrical specimens employing the static indirect tensile, repeated-load indirect tensile and the confined axial compression-shear tests. SHRP envisions that a total of twenty-four specimens will be required for this analysis: six specimens each (2 at each asphalt content) for fatigue, permanent deformation, low-temperature cracking and stiffness. Results of the tests at each asphalt content are compared to performance-based specification limits considering the environmental conditions and traffic level to which the pavement may be subjected. This process is used to further optimize the design asphalt content by evaluating the asphalt content that provides engineering properties prior to the final selection of the design asphalt content. If any one of the test criterion is not met, the mix must be redesigned.

2.9 Comparison to Structural Design Requirements

The mix design must be related to the pavement structural design for the asphalt layer. SHRP is utilizing initially the approach identified in NCHRP Report # 338. This approach uses the minimum total resilient moduli defined by the relationship between the structural layer coefficient and total resilient moduli in accordance with the AASHTO 1986 Guide for the Design of Pavement Structures (2).

Average total resilient modulus for each asphalt content may be determined from lab tests. Those asphalt contents that result in the minimum total resilient modulus which meets or exceeds the layer coefficient assumed for design are deemed adequate.

The layer thickness in the AASHTO Design Guide are determined by using structural layer coefficients, which do not consider different types of distress separately. SHRP Contract A-005 will advance the required mathematical models that are needed to predict mix behavior based on validated performance-based, accelerated laboratory tests related to various distresses, and identify the ranges or tolerances of the asphalt-aggregate properties that significantly affect pavement performance. This contract will also relate the traffic loading, environmental conditions and material properties to pavement distress for improved input to the AASHTO design procedures.

3. SHRP LEVEL II AAMAS

As noted above, SHRP's Level II AAMAS considers test equipment and methods which should provide a direct link between the measured fundamental engineering properties (stress, strain, etc.) in the laboratory and those measured directly in the field. Several of these tests may be more complex and time-consuming than current laboratory practices. However, they should provide a better estimate of in-place performance which in turn will enable the engineer to design a mix and structural section which will significantly increase in service-life and minimize maintenance and rehabilitation costs. Although initial costs may be higher, when amortized over the extended life of the pavement, this may result in a more economically attractive engineering investment strategy based on the Level II AAMAS performance-based laboratory tests. Shown in Table II are the accelerated lab tests (ALTs) currently being considered the by the A-003A contractor in the development of the Level II AAMAS.

As with the Level I AAMAS, the concept involves an initial development of a mix design based on volumetric properties and optimization of the design considering fundamental engineering properties based on pavement performance. For the Level II AAMAS, guidelines for materials selection, initial proportioning of the trial mix design, and volumetric analysis are identical to those identified in the Level I AAMAS. Similarly, the asphalt-aggregate compatibility test is as proposed for the Level I AAMAS. The essential differences between the Level I and II AAMAS are in the specimen fabrication, age and moisture conditioning, and the performance-based lab tests used to evaluate mechanical properties of the mix. In the Level II AAMAS specimens are initially compacted at three different asphalt contents, after which shear tests are conducted to assess permanent deformation potential. Based on the results of this preliminary testing, a detailed evaluation is conducted on the mix at a single asphalt content.

3.1 Test Specimen Fabrication (Compaction)

Compaction of the test specimens will utilize the rolling wheel compaction concept. Intuitively, the rolling wheel compaction should more closely reproduce field compaction. Indeed, a preliminary evaluation by the SHRP A-003A contractor indicates that the rolling wheel compactor provides a mix similar to field compacted mix in terms of structure and particle orientation. Specimens produced by rolling wheel compaction are cored or sawed from a larger mass, yielding smooth, cut surfaces. Smoothly cut surfaces are advantageous for several reasons: air voids can be more accurately measured; comparisons with specimens extracted from in-service pavements are more appropriate; and the specimens are more homogeneous thus reducing test variability.
## TABLE II - CANDIDATE METHODS FOR THE SELECTION/DEVELOPMENT OF PERFORMANCE BASED TESTS FOR AAMAS -- LEVEL II

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Advantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fatigue</td>
<td>Repeated flexure or diametral</td>
<td>Results usable for mix design and pavement analysis.</td>
</tr>
<tr>
<td></td>
<td>Direct tension</td>
<td>Eliminates need for fatigue tests.</td>
</tr>
<tr>
<td></td>
<td>Fracture mechanics</td>
<td>Eliminates need for fatigue tests.</td>
</tr>
<tr>
<td>Rutting</td>
<td>Uniaxial, triaxial, or simple shear</td>
<td>Can be conducted in creep, repeated, and dynamic modes.</td>
</tr>
<tr>
<td></td>
<td>Wheel tracking</td>
<td>Simulates field loading.</td>
</tr>
<tr>
<td>Thermal Cracking</td>
<td>Direct or indirect tension</td>
<td>Extensive experience with method.</td>
</tr>
<tr>
<td></td>
<td>Coefficient of thermal expansion</td>
<td>Significant prior use.</td>
</tr>
<tr>
<td></td>
<td>Thermal stress (restrained specimen)</td>
<td>Simulates field conditions.</td>
</tr>
<tr>
<td></td>
<td>Thermal fatigue (restrained specimen)</td>
<td>Simulates field conditions.</td>
</tr>
<tr>
<td>Aging</td>
<td>Short Term</td>
<td>Extended heating</td>
</tr>
<tr>
<td></td>
<td>Extended heating</td>
<td>Simulates plant conditions.</td>
</tr>
<tr>
<td></td>
<td>Extended mixing</td>
<td>Simulates field conditions.</td>
</tr>
<tr>
<td></td>
<td>Long Term</td>
<td>Extended heating</td>
</tr>
<tr>
<td></td>
<td>Low pressure oxidation</td>
<td>Significant aging at high temperatures.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Significant aging at low temperature.</td>
</tr>
<tr>
<td>Water Sensitivity</td>
<td>Compacted Mixtures</td>
<td>Modified triaxial cell AASHTO T-283</td>
</tr>
<tr>
<td></td>
<td>Potential for various conditions (e.g., environment, load)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose Mixtures</td>
<td>Boiling water</td>
</tr>
<tr>
<td></td>
<td>Potential for screening test.</td>
<td></td>
</tr>
<tr>
<td>Stiffness</td>
<td>Repeated load, axial</td>
<td>Extensive experience.</td>
</tr>
<tr>
<td></td>
<td>Repeated load, diametral Dynamic</td>
<td>Extensive experience.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Provides measure of stiffness over a range of times of loading and temperatures.</td>
</tr>
</tbody>
</table>
Specimen fabrication involves preparing a large slab with a rolling wheel compactor. Initially, the slab was compacted to dimensions of 16 in x 16 in x 8 in (Figure 4), from which the following specimens were extracted:

a) 2 - 4 in x 8 in cores for stiffness testing (unaged);
b) 6 - 2 in x 4 in cores for permanent deformation testing (2 as compacted, 2 after moisture conditioning and 2 after traffic conditioning);
c) 4 - 2 in x 2 in x 15 in beams for fatigue tests (2 unaged and 2 aged);
d) 3 - 2 in x 2 in x 8 in beams for thermal stress tests (all aged); and
e) 4 - 4 in x 4 in x 8 in cores for moisture sensitivity tests.

After analysis of substantial test data, however, the A-003A contractor has recommended two alternatives for compaction of slab specimens as shown in Figures 5 and 6. From the former, which is compacted to dimensions of 24 in x 26 in x 3\% in, the following specimens would be extracted:

a) 3 - 2 in x 2 in x 8 in beam specimens for dynamic modulus - axial tension-compression (short-term aged);
b) 5 - 6 in dia x 3 in cores for permanent deformation testing (2 as compacted, 1 after moisture conditioning and 1 after traffic conditioning);
c) 3 - 2 in x 2 in x 15 in beams for fatigue tests (short-term aged);
d) 3 - 2 in x 2 in x 8 in beams for thermal stress restrained tests (long-term aged); and
e) 4 - 4 in dia x 3\% in cores for water sensitivity tests.

The alternative slab specimen (Figure 6) is compacted to dimensions of 24 in x 17 in x 3\% in, from which all cylindrical specimens would be cored for use in the rutting, stiffness and water sensitivity tests.

3.2 Mixture Conditioning Procedures

3.2.1 Aging

Two aging tests will be utilized. The first simulates the short-term aging which occurs during plant mixing and construction; the second, the aging that occurs during five to ten years of service.

1. Forced Draft Oven

The aging that occurs during mixing and construction will be simulated by holding the loose mix in a forced draft oven at 135°C for 4 - 8 hours. Preliminary data indicate that until field validation suggests otherwise, 4 hours may be sufficient. Immediately after aging, the mix is allowed to cool to a temperature corresponding to a viscosity of 6 Poise (based on unaged asphalt properties) and then compacted.

2. Modified Triaxial Aging Test

The long-term aging is simulated by placing the compacted mix in a triaxial cell and pumping oxygen through the sample at a rate of 4 ft³/hr for 2 - 5 days at a temperature of 60 - 85°C. An alternative procedure would entail holding the compacted mix for 2 - 5 days in a forced draft oven at 85°C. Based on extensive modulus ratio data (before and after aging) it is hypothesized that the long term-aging test may not be necessary as the short-term aging may adequately characterize the mix response.

3.2.2 Moisture Sensitivity

To assess water sensitivity of a mix two conditioning procedures will be utilized: a hot soaking at 60°C for warm climates; or a hot soaking followed by one or more freeze-thaw cycles for cold climates. Prior to fatigue, permanent deformation and thermal cracking testing, compacted specimens would be subjected to wet conditioning at 25°C for 24 hour under 20 in Hg vacuum. The test apparatus is a triaxial compression type cell instrumented to monitor not only resilient modulus, but also volume change and permeability. The advantage of the proposed the environmental conditioning system (ECS) and test apparatus is numerous:

- Instrumented to monitor the permeability and resilient modulus of the specimens after each conditioning cycle: wetting, thawing or freezing;
- Eliminates leaking and specimen deformation during testing;
- Minimizes test variability as specimens require only one "setup;"
- Eliminates specimen handling (ie, transferring specimen from water bath to testing device) which is a major source of experimental error;
- Volume change of the specimen (swelling or shrinkage) is monitored during the conditioning cycle in terms of permeability rather than by thickness or bulk volume;
- Allows application of repeated loads throughout the duration of the test; and
- Simulation of field conditions (traffic and environmental) can be tested independently.

At this time it is anticipated that any or all of the following may be included in the specification: resilient modulus ratio; tensile strength ratio or minimum tensile strength; and maximum permeability.

3.3 Performance-Based Tests

Extensive evaluations are being conducted to identify tests to define the effects of the various factors, including asphalt binder on the fatigue, permanent deformation, thermal cracking, aging, and water sensitivity/adhesion characteristics of asphalt-aggregate mixes. For each of these distress modes, a number of tests was identified from which the specific performance-based accelerated laboratory tests are being selected, refined and/or developed. In addition, since stiffness is an important mix characteristic, tests to define this parameter over a broad temperature-frequency domain are also being evaluated.

The test equipment has been designed as either a series of independent "stand alone" components, as required for each procedure, or modules which can be integrated into a stationery system, (ie, loading frame) suitable for multi-purpose applications.
Figure 4. Rolling Wheel Compacted Slab—approximate dimensions.
Figure 5. Rolling Wheel Compacted Slab - #1
Figure 6. Rolling Wheel Compacted Slab - #2.
3.3.1 Low-Temperature and Thermal Cracking

The proposed test for use in controlling thermal cracking is a device in which rectangular specimens are fully restrained while subjected to controlled cooling, either monotonically or cyclically. During cooling the induced thermal stress is recorded as is the temperature at failure. Details of the test are summarized below:

1) 2 in x 2 in x 8 in specimens are extracted from compacted slab.
2) Specimens are epoxied to the end caps.
3) Specimens are placed in the test frame and cooled at a rate of 10°C/hr.
4) The resulting thermal stress is continuously recorded until fracture, at which time the temperature is also recorded.

The test can also be used to measure thermal fatigue behavior as well as to evaluate the effects of stress relaxation. For example, cycling the temperatures between 20°F and 70°F will gradually cause a reduction in the induced thermal stress with eventual fracture. Stress relaxation at cold temperatures can also be evaluated. The sample is cooled to a low temperature (e.g., 10°C), then held at that temperature while the resulting stress is monitored. These results may be used to relate the test results to field performance and to interpret the rate of loading effects on test results. The primary reasons for the selection of this test device are as follows:

1) It is sensitive to mixture variables, particularly asphalt type. Not only does it discriminate between asphalt types, but also between asphalt grades (asphalt stiffness).
2) It is sensitive to aggregate type.
3) It closely simulates field conditions.
4) The results from this test can be used directly in performance models such as COLD.
5) User-friendly software is readily available for computer control of the test and data acquisition.
6) No problems are anticipated in establishing precision statements.

3.3.2 Permanent Deformation

A number of different tests have been evaluated including axial creep and repeated load, shear creep and repeated load, and the VESYS repeated axial creep tests. In addition, a small linear wheel tracking device has been utilized (TRRL type) to test small slabs (approximately 1 ft square). The last is considered a "torture test" and has been included to assist in the validation of the test procedure(s) ultimately selected.

The most likely candidate is the confined axial compression shear test since both shear and normal stresses can be applied. Furthermore, both axial and shear stresses may be applied continuously (sustained for creep testing) or repeatedly applied with short loading durations to simulate the dynamic effects of traffic. The test apparatus can accommodate 6 in diameter specimens, thus permitting the evaluation of mixes containing aggregates with a maximum size of 1.5 in. The primary reasons for the selection of this device are as follows:

1) The repeated application of shear and normal stresses simulates the dynamic effects of traffic and approximates the state of stress in the upper portion of the pavement near the tire edge.
2) Specimens of minimal height can be tested, thus permitting evaluation of field cores as well as laboratory compacted specimens.
3) The versatility of the equipment is such that taller specimens can be tested in axial creep and/or repeated loading with or without confining stress.

3.3.3 Fatigue

Two types of flexural testing are under consideration at this time. The first is a four-point bending test in which the load is applied for about 0.1 second with a rest period between load applications of 0.5 seconds corresponding to a rate of 100 repetitions per minute. The second involves the use of pyramidal shaped, cantilevered specimens, loaded at a frequency of 20 Hz. For both test configurations, the following information is obtained:

1) stress and strain versus repetitions to failure;
2) strain energy to failure; and
3) dissipated energy per load cycle.

The slow cycle beam test appears, at this time, to be the leading candidate. Alternatively, a sinusoidal loading applied at 10 - 20 Hz in controlled strain may be used. However, the cantilever test on pyramidal specimens is still under consideration and may prove to be a suitable alternate. Use of the dissipated energy concept may prove to be the most appropriate method of analysis as data presented in this form are independent of the mode of loading, temperature, stress or strain levels and loading frequency. Reasons for selection of the beam test include the following:

1) The test is sensitive to mixture variables. Both controlled-stress and controlled-strain modes reflect the influence of the physical properties of the asphalt.
2) The test simulates field conditions since the specimen is subjected to bending stresses as is the asphalt-bound layer in-situ.
3) Results of the test can be used in performance models to define mixture response in pavement structures and can be used in performance related specifications to control mixture properties.
4) Specimens may be subjected to both aging and moisture conditioning prior to testing.
5) The test(s) is/are computer controlled and is/are relatively straightforward to conduct. After placement of the beam specimen in the test apparatus no further manual adjustments are required.
6) Use of the dissipated energy concept may eliminate the need to consider mode of loading, temperature, stress or strain levels and loading frequency.

3.3.4 Stiffness

Although the fatigue test and the permanent deformation tests can be used to measure stiffness, an independent study of stiffness testing has been included in the research program. Tests considered include the following:
1) axial dynamic modulus;
2) axial and diametral resilient modulus; and
3) shear dynamic modulus.

Preliminary analysis of the data indicates that some form of an axial dynamic modulus test may be the most appropriate. The A-003A contractor is recommending axial loading (tension and compression) of 2 in x 2 in x 8 in beam specimens. However, a temperature-frequency sweep with the confined axial compression shear test to determine dynamic shear modulus has not been eliminated.

In summary, the Level II approach is to use prismatic specimens for evaluation of load associated-fatigue, low-temperature and thermal-fatigue cracking properties. Cylindrical specimens will be utilized for evaluation of permanent deformation. For stiffness, prismatic and/or cylindrical specimens may be recommended. Results of the tests are compared to the performance-based specification limits considering the environmental conditions and traffic level to which the pavement may be subjected.

3.3.5 Comparison to Structural Design Requirements

Utilizing the fundamental engineering properties derived from the Level II AAMAS, SHRP Contract A-005 will advance the required mathematical models that are needed to predict mixture behavior based on validated performance-based accelerated laboratory tests related to different types of distress factors and identify the ranges on tolerances of the asphalt-aggregate properties that significantly affect pavement performance. This contract will also relate the traffic loading, environmental conditions and material properties to pavement distress for improved input to the AASHTO design procedures.

4. COMPARISON OF ASPHALT-AGGREGATE MIX ANALYSIS SYSTEMS

The preceding narrative has outlined the strategies of the various AAMAS approaches and the supporting tests. Clearly, all of the AAMAS procedures are somewhat more complex than the current, purely empirical, mix design procedures (eg: Marshall and Hveem). SHRP's strategy is to develop a performance-based specification for asphalt-aggregate mixtures, the AAMAS, and supportive tests based upon fundamental engineering properties of the mix from which reasonable estimates of pavement performance can be computed. Although the nature of the tests and mix design procedures may be more complex this approach will allow the engineer to optimize pavement performance considering available materials, estimated traffic and environmental conditions. The following discussion reviews the key factors that are an integral part of all AAMAS procedures: conditioning, compaction and mechanical evaluation.

4.1 Conditioning

Both the NCHRP and SHRP test concepts include considerations of the effects of moisture and aging on the fatigue, permanent deformation and thermal cracking potential of asphalt-aggregate mixes. To this end the proposed tests must be conducted on specimens which approximate those found in situ. In the area of moisture conditioning, the NCHRP AAMAS was developed from a relative limited study. The modified AASHTO T 283 procedure does not consider the effects of repeated loading which is significant. SHRP will advance a water conditioning approach in which compacted specimens are subjected to freeze/thaw cycling under vacuum and repeated loading in a triaxial compression-type cell.

The NCHRP AAMAS study has recommended a specific methodology for specimen conditioning to consider the effects of aging. Two methods were examined by the NCHRP AAMAS group, the Pressure Oxygen Vessel (POV) and the forced draft oven aging at an elevated temperature. For short-term aging, SHRP is proposing the use of a forced-draft oven; for long-term aging, the preliminary recommendation is the low pressure oxidation of compacted samples in a triaxial compression type cell.

4.2 Laboratory Compaction

Based on a limited study within the framework of the NCHRP project, NCHRP Report # 338 has identified the Corps of Engineers gyratory equipment for laboratory compaction. It is important to note that much of the NCHRP AAMAS study focussed on a comparison of the response of field cores (taken immediately after construction) to laboratory compacted specimens. Moreover, it is important to note that the field conditions which were duplicated in the NCHRP AAMAS Study did not necessarily represent good field practice since several of the projects exhibited high void contents.

A preliminary study conducted by SHRP Contract A-003A indicates that the rolling wheel compaction concept provides a mix structure similar to field compacted mixtures. Additionally, the A-003A's findings indicate that the rolling wheel compaction influences the mixture's resistance to pavement distress factors, especially permanent deformation.

The French are also strong advocates of laboratory compaction employing the rolling wheel concept. They have observed that compaction by a pneumatic-tired roller yields specimens which exhibit rheological behavior similar to that of material sampled in situ.

Although rolling wheel compaction may simulate field compaction, the British do not consider it suitable for the routine fabrication of specimens. Similar misgivings may be held by a State Highway Agency (SHA). Since mix designs may be the responsibility of the SHA's central laboratory, district laboratory, or the contractor, the compaction procedure recommended by SHRP must be appropriate for any of the aforementioned entities.

SHRP's strategy at this stage is to advocate the use of the Texas gyratory with Level I AAMAS, while the rolling wheel compaction concept will be advanced with the Level II AAMAS. In concert with the Level II AAMAS, SHRP is continuing to investigate which laboratory compaction procedure best simulates field compaction. A statistically-designed experiment has been developed whereby laboratory fabricated specimens are being prepared using a rolling wheel compactor and the Texas gyratory compactor. Field cores are being collected from the wheel paths and between the wheel paths of SHRP's LTTP sections. SHRP will attempt to bracket the void content from each of the samples groups. The variation in air voids in the laboratory-prepared specimens will be achieved by variation of the compactive energy for each compaction device.
4.3 Performance Based Laboratory Tests

In the area of thermal cracking, the NCHRP AAMAS study utilized the procedure termed CRACK3. It is not apparent that any investigations were conducted of a much more widely used methodology termed COLD, a well documented procedure developed in Canada. In addition, no consideration appears to have been given to the problem of thermal fatigue, a phenomenon being investigated by SHRP. Initially, SHRP will use the indirect tensile test in the Level I AAMAS to evaluate this phenomenon. However, the thermal stress restrained specimen test on beam specimens will be advanced simultaneously (Level II). If a demonstrated advantage or a significant technical advancement is clearly shown the thermal stress restrained specimen test may ultimately be included in SHRP's AAMAS.

The NCHRP AAMAS investigation did not include studies of the fatigue behavior of asphalt mixtures. Rather, a correlation procedure developed by Bauer (1988) using modulus measurements has been recommended as an initial way of characterizing fatigue response in the NCHRP AAMAS design methodology. At this time SHRP does not view characterization of fatigue solely by stiffness measurements as adequate. Of primary concern is the input to the performance models to evaluate the sensitivity of the test results to predicting fatigue. The repeated load indirect tensile test will be used as the SHRP Level I test with the flexural beam test employed to advance the understanding of the fundamental engineering properties affecting fatigue.

Both the British and French have identified problems with state-of-the-art beam fatigue testing, primarily associated with the time required to conduct the test and variability in the results. SHRP is presently evaluating an advancement of the beam test, testing protocol and method of analysis. As noted earlier, use of the dissipated energy concept may eliminate the need to consider mode of loading, temperature, stress or strain levels and loading frequency. This approach is being considered in the Level II AAMAS.

To define the permanent deformation potential of a mix, the NCHRP AAMAS study group performed a series of repeated-load indirect tensile tests. A few axial creep tests were performed. Values for $a$ and $\mu$ (required in the VESYS pavement analysis procedure) were obtained from these tests. Although the mixture comparisons and permanent deformation evaluations are based on diametral testing, the axial compression creep test is recommended for use in the resulting NCHRP AAMAS procedure. SHRP has developed a unique test for evaluating the repetition of high shear stresses at the pavement surface. The test being advanced by SHRP is the confined axial compression shear and is included in both Level I and Level II AAMAS.

4.4 AAMAS Procedures

SHRP's strategy relative to the AAMAS procedures will be to continue to advance the AAMAS framework as identified in Figure 1. Volumetric properties are key to the development of the initial mix design and are linked to the fundamental engineering properties. Therefore, the first stage in the AAMAS procedure will be evaluating air voids, VMA and density.

The second stage will include the evaluation of the mechanical or fundamental engineering properties and optimization of the asphalt-aggregate mixture design based on these properties. A key to the success of the performance-based accelerated laboratory test selection is the validation of these tests and predictive models.

The NCHRP AAMAS approach clearly indicated that one of its short comings was the use of mathematical models that were not truly performance based. As NCHRP # 338 report indicates, numerous models are available for predicting pavement performance in terms of various condition indicators. Most currently available models relate materials response such as resilient modulus, fatigue, or creep properties to predicted long-term performance, as measured by cracking or rutting. In addition, considerable laboratory research has been conducted to identify models that relate mix parameters (e.g., voids, asphalt content, aggregate absorption, etc) to material response such as resilient modulus or fatigue parameters. Unfortunately, there are few, if any, models that directly relate mix parameters to performance. Therefore, the relationship between mix parameters and pavement performance will have to be evaluated using a process such as the stepwise procedures illustrated in Figure 7. Even if the performance-based accelerated laboratory tests are validated, consideration must be given, to some extent, to practicality, cost and time. Also, the AAMAS must be applicable to those situations in which the mix design is the responsibility of the SHA's central or district laboratory, or the contractor. Shown in Table III is an estimate of the equipment costs and approximate time required to conduct the laboratory testing with the various AAMAS. The time identified for the mix design does not include specimen conditioning, mix curing period and compaction, which may add an additional 4-5 days to that identified.

The cost of the SHRP Level II AAMAS equipment is primarily associated with the fatigue, permanent deformation and low-temperature cracking modular equipment and the environmental chamber equipment, all of which are entirely computer driven. However, all test systems, particularly the SHRP Level II will provide a better estimate of in-place performance. This estimate, in turn, will provide a means of designing a mix that should significantly increase in service-life and decrease maintenance and rehabilitation costs. SHRP envisions this to be the case especially with its Level I and Level II AAMAS. Although initial costs may be higher, when amortized over the extended life of the pavement, this may result in a more economically attractive engineering investment strategy based on the performance-based laboratory tests.

5. FINAL SELECTION OF PERFORMANCE-BASED ACCELERATED LABORATORY TESTS

SHRP's research strategy concerning the development of performance-based accelerated laboratory tests is to provide a battery of tests based on fundamental engineering properties directly linked to field performance factors. From a technical point of view, test selection would be based on the following:

1) sensitivity to mix variables, particularly the asphalt binder;
2) simulation of field conditions;
3) applicability of results for use in design or performance prediction models (e.g: fundamental units);
4) ease of implementation; and
5) suitability for aging and water conditioning (e.g: fatigue, permanent deformation and thermal cracking).
Figure 7. Conceptual framework for relating asphalt properties to performance.
### AAMAS SYSTEMS

<table>
<thead>
<tr>
<th>TYPE</th>
<th>EQUIPMENT COST ($1000)</th>
<th>MIX DESIGN (DAYS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCHRP</td>
<td>100-150</td>
<td>3</td>
</tr>
<tr>
<td>SHRP I</td>
<td>150-200</td>
<td>3-4</td>
</tr>
<tr>
<td>SHRP II</td>
<td>220</td>
<td>4</td>
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</table>

Table III. Cost of AAMAS Systems
In terms of marketing these tests to the SHAs other factors may be considered. For example, State Highway Administrators or engineers may deem the following to take precedence over the technical aspects:

1) time involved for mix design and testing;
2) equipment cost;
3) technical expertise required of lab personnel;
4) simplicity and practicality of the tests;
5) perception of the tests as it relates to field equipment (eg: rolling wheel compaction);
6) communication in terms of ease of describing exactly what the test accomplishes;
7) "predictive" proof (eg: wheel tracking test provides "conclusive" evidence that a particular mix has the potential to rut or not rut); and
8) management's confidence - investment in the equipment will allow more accurate estimate of performance, extend service life and minimize maintenance and rehabilitation cost.

SHRP's strategy, or perhaps, challenge, is to develop an AAMAS that will accommodate the needs of the State Highway Agencies and paving contractors without compromising the technical advancements of the research.

References
Investigation of Asphalt–Aggregate Interactions and Their Sensitivity to Water

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An Investigation of Asphalt-Aggregate Interactions and Their Sensitivity to Water

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ABSTRACT

Asphalt-aggregate interactions are important factors in adhesion because bonding between the asphalt and aggregate at the interface is essential for good pavement performance. Water is an insidious intruding agent that disturbs or even destroys the interfacial bonding. An investigation into the type and water resistivity of asphalt-aggregate bonding has been undertaken in the SHRP A-003B contract. Specifically, the adsorption and desorption of asphalt and asphalt models, with functional groups similar to those present in asphalt, on siliceous and calcareous aggregates have been examined. These aggregates are representative of both good and poor performing aggregates in the United States.

The adsorption of asphalt models has shown that the amount adsorbed is dependent on the chemical properties of both the asphalt model and the aggregate. The more polar models, such as benzoic acid and phenylsulfoxide, adsorbed in larger quantities on all the aggregates than did the less polar models, such as indole and pyrene. The adsorption behavior is also strongly dependent on the chemistry of the aggregate surface. Desorption has been performed using both large and small quantities of water. The desorption behavior is dependent on the polarity of the asphalt models and on the quantity of water used. Modification of the aggregate surface can cause enhancement or reduction in the adsorption and desorption of the asphaltic model.

Asphalt adsorption and desorption on both siliceous and calcareous aggregates have shown that the asphalt chemical composition affects the amount of asphalt adsorbed and desorbed. Even more pronounced is the effect of aggregate chemistry on asphalt adsorption and desorption behavior. For any given asphalt, substantial differences are observed for the net adsorption, i.e., amount of asphalt remaining on the aggregate surface after desorption by water, on different aggregate types. Changes to the aggregate surface by modification can result in changing the net adsorption. The extent of change is dependent upon the chemistry and degree of coverage of the modifier. Addition of antistripping agents to the asphalt results in a lowering of both the initial amount of asphalt adsorbed as well as the amount desorbed. This methodology is being developed into a test method for selecting good performing asphalt-aggregate pairs.
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1. INTRODUCTION

The adhesion of asphalt to aggregate is requisite for maintaining the integrity of a pavement. Although there are many different causes and modes of failure, moisture intrusion in asphalt pavement can be insidious and pervasive causing asphalt to be stripped off of the aggregate surface. Asphalt stripped from aggregate can result in a loss of the structural integrity of a pavement itself and a reduction in pavement lifetime.

Asphalt pavement can be conceived as a continuum of variously sized aggregates ranging from large to extremely small that are coated with asphalt. It is conceptually possible that every asphalt molecule may either be in direct contact with an aggregate surface or in contact with other asphalt molecules that are in contact or influenced by aggregates. Asphalt that is not in contact with a larger aggregate may be strongly influenced by aggregate fines that per unit weight have higher surface areas and, hence, a larger sphere of influence than the larger aggregates.

Water affects asphalt bonding to aggregate by penetrating the asphalt film and intruding between the asphalt film and the aggregate surface. Water is a much smaller and more mobile molecule than an asphalt molecule and can compete for the active sites on the aggregate surface. The competitiveness for the surface relative to the asphalt molecules depends upon the surface chemistry of the aggregates, the polarity of the active sites and the chemistry of the asphalt molecules. If water has higher affinity for the aggregate surface than do the resident asphalt molecules, water can then compete successfully for the surface and displace asphalt from the surface, thereby yielding a chemical environment different from the surrounding asphalt-aggregate matrix. As more water occludes to the aggregate surfaces, asphalt is displaced from the surface, disrupting the adhesive bond between asphalt and aggregate that provides cohesion for the asphalt pavement. This phenomenon results in areas of pavement with less strength and internal integrity that culminates in inferior pavements.

Research in SHRP A-003B has focused upon describing and defining asphalt-aggregate interactions that are either resistive or sensitive to moisture. Three specific areas of research have been examined. The first area involved evaluating the specific chemistry of asphalt adsorption onto rock using model species that are representative of polar functional group types that are present in asphalt. The second aspect evaluated the compatibility of different asphalt-aggregate pairs and their respective sensitivity to water. The third aspect resulted from the observation that some aggregates are sensitive to water regardless of the asphalt type present. Hence, the aggregate surface was modified by silane treatment with silanes of different chemistries to determine the effect of these chemistries on asphalt-aggregate interactions and water sensitivity.
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2. DEFINITION OF ASPHALT-AGGREGATE INTERACTIONS BY ASPHALTIC MODEL COMPONENT ADSORPTION

Asphalt-aggregate interactions are complex because they involve the chemistries of asphalt and aggregate both of which are inherently heterogeneous and dependent on source materials. Using model organic species to represent some of the functional groups present in asphalt offers a simplification by which to gain understanding as to how individual asphaltic components may interact with aggregates.

Model components were selected based on the literature description of the types of components present in aged and unaged asphalts. The components chosen ranged from acidic to basic and some contained heteroatoms such as oxygen, nitrogen and sulfur. The functional group types selected and the specific compound chosen to represent that group are given in the following: sulfoxides by phenylsulfoxide; carboxylic acids by benzoic acid; nitrogen bases by phenanthridine; pyrroles by indole; ketones by fluorenone; phenols by l-naphthol; and aromatic hydrocarbons by pyrene and naphthalene.

A number of different aggregate types was used including granite, gravel, limestone and greywacke. All of these materials were obtained from the SHRP Materials Reference Library (MRL). The experiment that was performed included two parts: first, the adsorption of the model component from cyclohexane solution onto the aggregate at 25°C, and, second, the desorption of the model from the aggregate after introduction of an equal volume of water as cyclohexane into the solution at 25°C. The adsorption from solution provides a means of allowing close contact between the model species and the aggregate without raising the temperature.

Comparing the adsorption affinity for the models over the seven aggregates tested resulted in an affinity ranking of phenylsulfoxide > benzoic acid > phenanthridine > l-naphthol > 9-fluorenone > indole > pyrene > naphthalene. The more polar species, phenylsulfoxide, benzoic acid and phenanthridine, showed substantial adsorption onto aggregates while the less polar naphthalene and pyrene adsorbed a very small amount, if any. The higher surface area aggregates adsorbed substantially more model component than did the low surface area aggregates. Comparing the aggregates for their affinity to adsorb asphaltic model components resulted in the ranking of RH-greywacke > RL-gravel > RC-limestone > RJ-gravel > RB-granite > RD-limestone > RA-granite. Hence, the aggregate adsorption behavior varied over the different aggregates and seemed more dependent on the surface chemistry and morphology of the gravel, granite, greywacke or limestone than on the type of aggregate.
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The desorption by water experiments showed a different behavior than that observed in adsorption. The more polar substances, phenylsulfoxide and benzoic acid, that adsorbed the most, desorbed the most in the presence of water at 25°C. The desorption behavior of the model components over all of the aggregates ranked as phenylsulfoxide > benzoic acid > indole > 9-fluorenone > phenanthridine > 1-naphthol. The aggregates exhibited the following ranking as their propensity to desorb asphaltic models: RL-gravel > RJ-gravel > RH-greywacke > RC-limestone > RB-granite > RD-limestone.

These adsorption and desorption experiments showed that different asphaltic models had different affinity for adsorbing onto aggregates and that different aggregates had different propensities for being able to retain the different models under adverse water conditions. Although most of the model-aggregate pairs desorbed model component in the presence of water, increased adsorption was observed for all models with RA-granite, benzoic acid and RD-limestone and 1-naphthol in combination with several aggregates. The presence of water for these systems seemed to provide increased adsorption sites that promoted adsorption of the model through hydrogen bonding. But did this increased adsorption remain over a period of time and did displaced asphalt models readсорb onto the aggregate in the presence of water? To answer those questions, desorption experiments were performed after four and seven days for aggregates RA-granite, RD-limestone, RJ-gravel, and RL-gravel. The percent desorption or the percent increased adsorption indicated by a + sign is shown in Table 1. The differences in the desorption behavior from four to seven days were minimal, indicating that the desorption behavior once established remained as the operative behavior.

Table 1. Effect of Time on Desorption of Asphaltic Models

<table>
<thead>
<tr>
<th>Model</th>
<th>RA-granite</th>
<th>RD-limestone</th>
<th>RJ-gravel</th>
<th>RL-gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4</td>
<td>7</td>
<td>4</td>
<td>7</td>
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<tr>
<td>Benzoic Acid</td>
<td>+578</td>
<td>+576</td>
<td>+16.2</td>
<td>+14.4</td>
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<tr>
<td>Phenylsulfoxide</td>
<td>+139</td>
<td>+126</td>
<td>69.3</td>
<td>70.5</td>
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<tr>
<td>1-Naphthol</td>
<td>+445</td>
<td>+487</td>
<td>+113</td>
<td>+140</td>
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<tr>
<td>Indole</td>
<td>+100</td>
<td>+55.6</td>
<td>38.3</td>
<td>38.3</td>
</tr>
</tbody>
</table>

It is well-known that the initial contact between asphalt and aggregate in a hot-mix occurs at relatively high temperatures. However, all of the adsorption experiments discussed thus far were performed at 25°C. To evaluate the effect of heat on the bonding between asphalt and aggregate, a series of experiments was
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performed in which the cyclohexane solution was decanted from the aggregates after adsorption; the model coated aggregate was heated at 150°C for two hours; and then the model coated aggregate was desorbed at 25°C at pH 5.9 and pH 8.4. As a control, the model coated aggregates were also desorbed with water after no heating as shown in Table 2.

Table 2. Effect of Heat and pH on Percent Bulk Water Desorption Test of Selected Asphalt Models and MRL Aggregates

<table>
<thead>
<tr>
<th>Aggregate</th>
<th>Model</th>
<th>Percent Desorbed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>pH 5.9 no heat</td>
<td>pH 5.9 heat</td>
</tr>
<tr>
<td>RC</td>
<td>Benzoic Acid</td>
<td>81.4</td>
</tr>
<tr>
<td></td>
<td>Phenylsulfoxide</td>
<td>49.1</td>
</tr>
<tr>
<td></td>
<td>1-Naphthol</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>Phenanthridine</td>
<td>17.5</td>
</tr>
<tr>
<td></td>
<td>9-Fluorenone</td>
<td>15.9</td>
</tr>
<tr>
<td></td>
<td>Indole</td>
<td>27.3</td>
</tr>
<tr>
<td>RJ</td>
<td>Benzoic Acid</td>
<td>55.9</td>
</tr>
<tr>
<td></td>
<td>Phenylsulfoxide</td>
<td>63.2</td>
</tr>
<tr>
<td></td>
<td>1-Naphthol</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>Phenanthridine</td>
<td>49.3</td>
</tr>
<tr>
<td></td>
<td>9-Fluorenone</td>
<td>58.2</td>
</tr>
<tr>
<td></td>
<td>Indole</td>
<td>30.6</td>
</tr>
</tbody>
</table>

* Normal testing time period, 2 days.
* b Complex formed that interfered with the visible measurement.

Some of the model-aggregate systems were responsive to heat and change in pH. Notably, phenylsulfoxide showed increased desorption after heating with RC-limestone while benzoic acid and indole with RC-limestone gave unusual behavior. It is suspected that benzoic acid and indole formed a water soluble complex with some inorganic constituent from RC-limestone which led to changes in the UV visible spectrum. The background signal led to false readings that suggested increased adsorption of the model which was impossible under this experimental set-up. However, these unusual results point to the possibility of interactions between asphaltic components and water soluble inorganic species that may lead to loss of adhesion during water treatment. For RJ-gravel, benzoic acid, phenylsulfoxide and phenanthridine showed sensitivity to heat and pH. Phenylsulfoxide and phenanthridine, both basic molecules, showed increased desorption after heat at both pH 5.9 and 8.4 while benzoic acid showed...
substantially less desorption at pH 8.4. However, though these changes were real, the overall trend for both aggregate was that heating of the coated model did not cause any substantial improvement in the retention of the model species. Only with the more acidic or basic compounds did changing the pH to more basic result in differences in desorption behavior.

3. THE COMPATIBILITY AND WATER SENSITIVITY OF ASPHALT-AGGREGATE PAIRS

The sensitivity of asphalt pavements is of concern because damage caused by water can affect both performance and longevity. If the water sensitivity of a pavement can be predicted, then either modifications or additions to the asphalt-aggregate systems can be implemented to promote better adhesion and increased resistivity of water. It is desirable to develop a methodology by which to predict such sensitivity to water.

The evaluation of the compatibility and affinity of a given asphalt for a given aggregate was performed initially by determining isotherms and monolayer amounts of the asphalt adsorbed onto aggregate. Then analysis of the desorption behavior followed, also as an isotherm. This methodology provided information concerning both the adsorption and desorption behavior of asphalt from aggregate over a concentration range. Considerable amounts of time were required to attain the data. An alternative methodology was developed based on data obtained from the isotherms. A point at the higher concentration range of the isotherms was selected for adsorption. Hence, the following experimental procedure was employed. First, asphalt was adsorbed onto aggregate from a 0.6 g/L asphalt in toluene solution for eight hours using a recirculating column maintained at 25°C. The aggregate used was −40 to +80 mesh. The amount of asphalt adsorbed was determined by measuring the amount of asphalt remaining in solution by using visible spectroscopy and calculating the amount remaining in solution and the amount adsorbed. Second, water was introduced at ~280 mmolar into the toluene solution. The amount of asphalt desorbed from the aggregate was determined by visible spectroscopy and the amount remaining on the aggregate surface calculated. The amount remaining is termed net adsorption and gives a measure of the affinity of the asphalt-aggregate pair. The difference between the amount of asphalt adsorbed before and after desorption by water serves as an indicator of the water sensitivity of the pair.

3.1 Net Adsorption Behavior

For this study, the eleven MRL aggregates were used in net adsorption experiments with three asphalts, AAD-1, AAK-1, and AAM-1. The properties of the virgin asphalts are given in Table 3. For this study, the asphalts had been
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aged in a thin film oven for five hours at 163°C prior to the net adsorption test. The eleven aggregates ranged from siliceous to calcareous-based containing such materials as granite, gravel, basalt, greywacke, sandstone and limestone. The classification and properties of these aggregates are presented in Table 4.

Although the asphalts differ quite substantially in their composition, the differences in the asphalt adsorption behavior were relatively small for a given aggregate. The amount adsorbed ranked as AAD-1 > AAK-1 > AAM-1 for most aggregates with AAD-1 and AAK-1 exchanging position occasionally (Table 5). The amounts obtained for initial adsorption ranged from a high of 1.9 for AAD-1 on RC-limestone to a low of 0.18 mg/g on RA-granite; a high of 1.7 mg/g of AAK-1 on RC-limestone to a low of 0.25 mg/g on RA-granite; highs of 1.4 mg/g on RC-limestone and RK-basalt and a low of 0.2 mg/g on RA-granite. Hence, each asphalt exhibited high and low levels of adsorption on the same aggregates, but the magnitude of the differences among the aggregates for each given asphalt was quite large.

The adsorption behavior of the siliceous aggregates before and after water desorption varied considerably. Two aggregates, RA-granite and RJ-gravel, showed low adsorption and were quite sensitive to water regardless of the asphalt used. Aggregates, RB-granite, RE-gravel, and RG-sandstone, showed similar behavior in their initial asphalt adsorption for the three asphalts; however, RE-gravel tended to show a higher sensitivity to water and an increased amount of asphalt desorbed compared to the other aggregates. The two siliceous aggregates that gave the largest amounts of asphalt adsorption, regardless of asphalt, were RH-greywacke and RK-basalt. Both of these aggregates had low sensitivity to water.

The limestones used in this study included RC, a highly absorptive limestone, RD, a nonabsorptive limestone, and RF, a limestone with many other types of minerals present. The initial adsorption behavior of the three asphalts was similar on the three limestones which ranked as RC > RF > RD. However, the moisture sensitivities of these limestones seemed somewhat asphalt dependent with AAM-1 showing more sensitivity to water than either AAD-1 or AAK-1. RF-limestone yielded considerably more desorbed asphalt than did either RC or RD-limestones.

Even though the initial amount of adsorption and the amount of asphalt desorption that occurred varied somewhat from asphalt to asphalt, for a given aggregate the net adsorption, defined as the amount of asphalt remaining on the aggregate, was similar. The net adsorption ranking of the siliceous aggregates for all three asphalts was RK-basalt > RH-greywacke > RB-granite > RE-gravel > RG-sandstone > RJ-gravel > RA-granite. The net adsorption of the limestone aggregates ranked as RC > RD ≈ RF.
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Associate Professor  
Auburn University

## Table 3. Physical and Chemical Properties of Asphalts

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>Asphalt Properties</th>
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<tbody>
<tr>
<td></td>
<td>AAD-1</td>
<td>AAM-1</td>
<td>AAK-1</td>
<td></td>
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<tr>
<td>140°F, Viscosity Poise</td>
<td>1055</td>
<td>1992</td>
<td>3256</td>
<td></td>
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<tr>
<td>275°F, cst</td>
<td>309</td>
<td>569</td>
<td>562</td>
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<td><strong>Component Analysis</strong></td>
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<tr>
<td>Asphaltenes, %</td>
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<td>21.1</td>
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<td>Polar Aromatics, %</td>
<td>41.3</td>
<td>50.3</td>
<td>41.8</td>
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<td>Naphthene Aromatics, %</td>
<td>25.1</td>
<td>41.9</td>
<td>30.0</td>
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<td>Saturates, %</td>
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<td>5.1</td>
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<td><strong>Elemental Analysis</strong></td>
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<td>Carbon, %</td>
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<td>10.2</td>
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<td>Nickel, ppm</td>
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<td>C\text{aromatic}^%</td>
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<td>24.7</td>
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<td>H\text{aromatic}^%</td>
<td>6.8</td>
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<td><strong>Absorptivity (liters/ gm cm)</strong></td>
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<td>410 nm</td>
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<td>375 nm</td>
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<td>8.11</td>
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<td><strong>Functional Groups</strong></td>
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<td>Carboxylic Acids</td>
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<td>0.013</td>
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<td>Acid Anhydrides</td>
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<td>Quinolones</td>
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### Table 4. Properties of Aggregates

<table>
<thead>
<tr>
<th>Sample</th>
<th>RA Granite</th>
<th>RB Granite</th>
<th>RC Limestone</th>
<th>RD Limestone</th>
<th>RE Gravel</th>
<th>RE Glacial Gravel</th>
<th>RG Sandstone</th>
<th>RH Greywacke</th>
<th>RJ Gravel</th>
<th>RK Basalt</th>
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<tr>
<td>SiO₂</td>
<td>73.4</td>
<td>56.2</td>
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<td>Al₂O₃</td>
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<td>19.8</td>
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<td>1.86</td>
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<td>13.7</td>
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<td>Fe₂O₃</td>
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<td>0.99</td>
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</tr>
<tr>
<td>P₂O₅</td>
<td>&lt;0.05</td>
<td>0.06</td>
<td>&lt;0.05</td>
<td>&lt;0.05</td>
<td>&lt;0.05</td>
<td>&lt;0.05</td>
<td>&lt;0.05</td>
<td>0.13</td>
<td>0.09</td>
<td>0.22</td>
<td>0.05</td>
</tr>
<tr>
<td>MnO₂</td>
<td>0.05</td>
<td>0.12</td>
<td>&lt;0.02</td>
<td>&lt;0.02</td>
<td>&lt;0.02</td>
<td>0.03</td>
<td>0.04</td>
<td>0.20</td>
<td>&lt;0.02</td>
<td>0.21</td>
<td>&lt;0.02</td>
</tr>
<tr>
<td>LOV</td>
<td>0.22</td>
<td>1.98</td>
<td>40.3</td>
<td>35.0</td>
<td>0.43</td>
<td>37.4</td>
<td>18.7</td>
<td>0.96</td>
<td>0.59</td>
<td>-0.36</td>
<td>11.2</td>
</tr>
<tr>
<td>Surface Area m²/g</td>
<td>99.59</td>
<td>100.11</td>
<td>100.71</td>
<td>100.25</td>
<td>98.75</td>
<td>99.95</td>
<td>99.88</td>
<td>99.47</td>
<td>99.48</td>
<td>100.4</td>
<td>98.23</td>
</tr>
</tbody>
</table>

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AN INVESTIGATION OF ASPHALT-AGGREGATE INTERACTIONS AND THEIR SENSITIVITY TO WATER
Table 4. Properties of Aggregates (continued)

<table>
<thead>
<tr>
<th>Sample</th>
<th>RA Granite</th>
<th>RB Granite</th>
<th>RC Limestone</th>
<th>RD Limestone</th>
<th>RE Gravel</th>
<th>RF Glacial Gravel</th>
<th>RG Sandstone</th>
<th>RH Greywacke</th>
<th>RJ Gravel</th>
<th>RK Basalt</th>
<th>RL Gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acid Insolubles %</td>
<td>94.6</td>
<td>87.9</td>
<td>7.9</td>
<td>23.5</td>
<td>96.1</td>
<td>28.2</td>
<td>55.7</td>
<td>92.1</td>
<td>96.2</td>
<td>90.1</td>
<td>85.3</td>
</tr>
<tr>
<td>Water Solubles %</td>
<td>11.7</td>
<td>8.1</td>
<td>8.1</td>
<td>5.1</td>
<td>6.6</td>
<td>5.0</td>
<td>4.9</td>
<td>9.7</td>
<td>6.3</td>
<td>7.4</td>
<td>9.3</td>
</tr>
<tr>
<td>Lithology %</td>
<td>98.4 Granite 1.4 Basalt</td>
<td>100 Granite</td>
<td>100 Limestone</td>
<td>53.3 Shaly Limestone 26.8 Limestone 19.7 Arenaceous Limestone</td>
<td>Misc.</td>
<td>72.6 Limestone 10.8 Misc. 5.9 Greywacke 4.4 Chert 3.7 Grandiorite 2.6 Basalt</td>
<td>100 Calcareous Sandstone</td>
<td>71.3 Micaceous Sandstone 11.2 Misc. 10.9 Granite 6.6 Chert</td>
<td>47.4 Sandstone 28.4 Granite 23.7 Misc. 0.4 Sandstone</td>
<td>94.4 Basalt 4.5 Misc. 0.6 Sandstone</td>
<td>59.1 Chert 18.2 Arenaceous Limestone 11 Granite 5.8 Misc.</td>
</tr>
</tbody>
</table>
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Dr. Christine W. Curtis
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Table 5. Initial and Net Adsorption of Asphalt on Aggregate

<table>
<thead>
<tr>
<th>Asphalt/Aggregate</th>
<th>AAD-1 Initial</th>
<th>AAD-1 Net</th>
<th>AAK-1 Initial</th>
<th>AAK-1 Net</th>
<th>AAM-1 Initial</th>
<th>AAM-1 Net</th>
</tr>
</thead>
<tbody>
<tr>
<td>RA-granite</td>
<td>0.18 ± 0.03</td>
<td>0.07</td>
<td>0.25 ± 0.04</td>
<td>0.18</td>
<td>0.20 ± 0.18</td>
<td>0.001</td>
</tr>
<tr>
<td>RB-granite</td>
<td>0.85 ± 0.04</td>
<td>0.68</td>
<td>0.89 ± 0.11</td>
<td>0.73</td>
<td>0.77 ± 0.03</td>
<td>0.60</td>
</tr>
<tr>
<td>RC-limestone</td>
<td>1.9</td>
<td>1.5</td>
<td>1.7</td>
<td>1.3</td>
<td>1.4</td>
<td>1.2</td>
</tr>
<tr>
<td>RD-limestone</td>
<td>0.73 ± 0.06</td>
<td>0.60</td>
<td>0.73 ± 0.02</td>
<td>0.59</td>
<td>0.69 ± 0.09</td>
<td>0.50</td>
</tr>
<tr>
<td>RE-gravel</td>
<td>0.98 ± 0.05</td>
<td>0.69</td>
<td>1.01 ± 0.006</td>
<td>0.61</td>
<td>0.85 ± 0.02</td>
<td>0.45</td>
</tr>
<tr>
<td>RF-glacial gravel</td>
<td>0.90 ± 0.04</td>
<td>0.61</td>
<td>0.85 ± 0.06</td>
<td>0.52</td>
<td>0.83 ± 0.05</td>
<td>0.44</td>
</tr>
<tr>
<td>RG-sandstone</td>
<td>0.70 ± 0.02</td>
<td>0.58</td>
<td>0.60 ± 0.09</td>
<td>0.42</td>
<td>0.59 ± 0.02</td>
<td>0.34</td>
</tr>
<tr>
<td>RH-greywacke</td>
<td>1.3</td>
<td>1.0</td>
<td>1.22</td>
<td>0.94</td>
<td>1.2</td>
<td>0.91</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td>0.31 ± 0.003</td>
<td>0.12</td>
<td>0.34 ± 0.06</td>
<td>0.19</td>
<td>0.42 ± 0.63</td>
<td>0.21</td>
</tr>
<tr>
<td>RK-basalt</td>
<td>1.7</td>
<td>1.4</td>
<td>1.56</td>
<td>1.26</td>
<td>1.4</td>
<td>1.2</td>
</tr>
<tr>
<td>RL-gravel</td>
<td>1.4</td>
<td>1.0</td>
<td>1.4</td>
<td>0.99</td>
<td>1.2</td>
<td>0.83</td>
</tr>
</tbody>
</table>

4. EFFECT OF SILANE MODIFICATION ON ASPHALT-AGGREGATE INTERACTIONS

Since the strength and durability of the interfacial bond between asphalt and aggregate in a pavement are the direct consequence of the interfacial bond, modification of the aggregate surface may lead to improvements in adhesion and in the water resistivity of the bonding. The research performed involved using organosilane compounds to pretreat the aggregate surface prior to adhesion. The adsorption behavior for both model asphalt species and asphalts was determined for three MRL aggregates, RC-limestone, RJ-gravel, and RL-gravel. These three aggregates represented behaviors in the net adsorption study of high adsorption/low desorption; low adsorption/high desorption and intermediate adsorption/intermediate desorption, respectively.

Four model compounds, benzoic acid, phenylsulfoxide, 1-naphthol, and phenanthidine, and two asphalts, AAD-1 and AAK-1, were used in adsorption and desorption experiments with organosilane treated aggregate. Two organosilane reagents, n-octyltrichlorosilane and 3-mercaptopropyltrimethoxysilane, designated as hydrocarbon and thiol, respectively, were used to pretreat three aggregates. The experiment was performed by first pretreating the
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aggregates with silanes and then using these pretreated aggregates in adsorption and desorption experiments. In the adsorption experiments, the asphalt models were adsorbed onto the aggregate from cyclohexane while the asphalts were adsorbed from toluene. The amount of asphalt model that was adsorbed was determined by measuring the UV absorbance at a specific wavelength of high absorbance for each compound and then calculating the amount of model remaining in solution and the amount adsorbed on the aggregate surface. Likewise, the amount of asphalt adsorbed was determined in the same manner but with measurements of the amount remaining in solution being taken at three wavelengths: 283, 410, and 450 nm. The desorption behavior of the asphalt models and asphalts was determined by adding an equivalent amount of water as organic solvent present and determining the amount of adsorbed component that desorbed in both the aqueous and organic phases. All adsorption and desorption data using the organosilane treated aggregates were compared to the testing of the asphalt models and the asphalts or untreated aggregates.

Adsorption of the asphalt models from cyclohexane solution onto the silane treated aggregates was compared to the adsorption of these same models onto untreated aggregates under the same adsorption conditions. As shown in Table 6, thiol pretreatment decreased adsorption of most of the asphalt models and enhanced only the adsorption of benzoic acid on RC-limestone and RJ-gravel and phenylsulfoxide on RL-gravel. Likewise, the majority of the asphalt models showed less adsorption with the hydrocarbon pretreatment of the aggregates than the untreated. However, more asphalt models showed increased adsorption with hydrocarbon pretreatment compared to those with thiol pretreatment.

Comparisons of the percent of asphalt model desorbed among the untreated, hydrocarbon pretreated, and thiol pretreated aggregates are given in Table 7. The hydrocarbon pretreatment resulted in less desorption of the adsorbed model in almost every model-aggregate combination. In four cases, the hydrocarbon coating yielded higher levels of adsorption of the model in the desorption experiment than in the adsorption experiment compared to one case with the untreated aggregate. The thiol pretreatment appeared to enhance the ability of RJ-gravel to retain the asphalt model substantially more than the other aggregates.

The adsorption of asphalt onto untreated and silane treated aggregates was monitored at three different wavelengths on the premise that different types of components inherent in the asphalt may be involved and absorbing at different wavelengths. A comparison of the adsorption behavior of AAD-1 and AAK-1 asphalt onto the pretreated aggregates is given in Table 8 as the percent change in the amount of asphalt adsorbed on the pretreated aggregates compared to the untreated aggregates. The hydrocarbon pretreatment promoted increased adsorption in four out of nine cases for AAD-1 and six out of nine cases for AAK-1. The thiol pretreatment appeared to enhance RJ-gravel in conjunction.
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Dr. Christine W. Curtis
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Auburn University

Table 6. Percent Change* in Adsorption Amounts** of asphalt Models on MRL Aggregates Treated with Organosilanes

<table>
<thead>
<tr>
<th>Modifier/ Aggregate</th>
<th>Asphalt Models</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Benzoic Acid</td>
</tr>
<tr>
<td>Thiol</td>
<td></td>
</tr>
<tr>
<td>RC-limestone</td>
<td>+25</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td>+8</td>
</tr>
<tr>
<td>RL-gravel</td>
<td>-8</td>
</tr>
<tr>
<td>Hydrocarbon</td>
<td></td>
</tr>
<tr>
<td>RC-limestone</td>
<td>+20</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td>-26</td>
</tr>
<tr>
<td>RL-gravel</td>
<td>-14</td>
</tr>
</tbody>
</table>

* Percent change = 100 ((amount adsorbed on untreated aggregate – amount adsorbed on treated aggregate)/amount adsorbed on untreated aggregate) (mg/g)

** Based on the average of four experiments (max 2g aggregate)

Table 7. Comparison of Percent Asphalt Models Desorbed from Treated and Untreated Aggregates

<table>
<thead>
<tr>
<th>Aggregate/ Model</th>
<th>Aggregate Treatments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No Treatment</td>
</tr>
<tr>
<td>RC-limestone</td>
<td></td>
</tr>
<tr>
<td>Benzoic Acid</td>
<td>54.0</td>
</tr>
<tr>
<td>Phenylsulfoxide</td>
<td>71.5</td>
</tr>
<tr>
<td>1-Naphthol</td>
<td>+34.9</td>
</tr>
<tr>
<td>Phenanthridine</td>
<td>54.1</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td></td>
</tr>
<tr>
<td>Benzoic Acid</td>
<td>62.8</td>
</tr>
<tr>
<td>Phenylsulfoxide</td>
<td>63.4</td>
</tr>
<tr>
<td>1-Naphthol</td>
<td>12.5</td>
</tr>
<tr>
<td>Phenanthridine</td>
<td>29.4</td>
</tr>
<tr>
<td>RL-gravel</td>
<td></td>
</tr>
<tr>
<td>Benzoic Acid</td>
<td>53.4</td>
</tr>
<tr>
<td>Phenylsulfoxide</td>
<td>67.8</td>
</tr>
<tr>
<td>1-Naphthol</td>
<td>16.5</td>
</tr>
<tr>
<td>Phenanthridine</td>
<td>37.7</td>
</tr>
</tbody>
</table>

+ Increased adsorption of asphalt model was observed. No desorption occurred.
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Dr. Christine W. Curtis
Associate Professor
Auburn University

Table 8. Percent Change* in Amount of Asphalt Adsorbed on Organosilane Treated Aggregates

<table>
<thead>
<tr>
<th>Asphalt/Aggregate</th>
<th>Wavelengths (nm)</th>
<th>283</th>
<th>410</th>
<th>450</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hydrocarbon</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AAD-1</td>
<td>RC-limestone</td>
<td>-9.4</td>
<td>+0.8</td>
<td>+3.1</td>
</tr>
<tr>
<td></td>
<td>RJ-gravel</td>
<td>+40.8</td>
<td>+14.4</td>
<td>-17.1</td>
</tr>
<tr>
<td></td>
<td>RL-gravel</td>
<td>-18.1</td>
<td>-30.0</td>
<td>-12.8</td>
</tr>
<tr>
<td>AAK-1</td>
<td>RC-limestone</td>
<td>+17.2</td>
<td>-75.9</td>
<td>-0.3</td>
</tr>
<tr>
<td></td>
<td>RJ-gravel</td>
<td>+119</td>
<td>+114</td>
<td>+50.2</td>
</tr>
<tr>
<td></td>
<td>RL-gravel</td>
<td>+16.4</td>
<td>+40.5</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>Thiol</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AAD-1</td>
<td>RC-limestone</td>
<td>-18.2</td>
<td>-15.0</td>
<td>-21.1</td>
</tr>
<tr>
<td></td>
<td>RJ-gravel</td>
<td>+1.5</td>
<td>-9.5</td>
<td>-28.2</td>
</tr>
<tr>
<td></td>
<td>RL-gravel</td>
<td>-19.6</td>
<td>-35.9</td>
<td>-38.1</td>
</tr>
<tr>
<td>AAK-1</td>
<td>RC-limestone</td>
<td>-13.5</td>
<td>-50.9</td>
<td>-12.3</td>
</tr>
<tr>
<td></td>
<td>RJ-gravel</td>
<td>+172</td>
<td>+140</td>
<td>+186</td>
</tr>
<tr>
<td></td>
<td>RL-gravel</td>
<td>-64.1</td>
<td>-77.9</td>
<td>-17.8</td>
</tr>
</tbody>
</table>

* Percent change = 100 ((amount (mg/g) adsorbed on untreated aggregate - amount adsorbed on treated aggregate)/amount adsorbed on untreated aggregate).
+ Increase in amount asphalt adsorbed with organosilane treatment.
— Decrease in amount asphalt adsorbed with organosilane treatment.

with AAK-1 asphalt specifically. The other pretreated aggregates showed variable behavior with AAK-1.

The desorption behavior of the two asphalts at three different wavelengths compared to the untreated aggregate is given in Table 9. The hydrocarbon pretreatment decreased the amount of AAD-1 desorption occurring with all three aggregates at 283 nm. At 410 and 450 nm, the results were variable for the different aggregates. However, hydrocarbon pretreated RL-gravel showed consistently more AAD-1 retention in the presence of water than did untreated aggregate. The thiol pretreatment enhanced the retention of AAD-1 asphalt on RJ-gravel at all three wavelengths.
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The effect of the aggregate pretreatments on the adsorption behavior of AAK-1 was different than on AAD-1. The hydrocarbon pretreatment resulted in slightly more desorption of AAK-1 at all wavelengths than that which occurred with the untreated aggregate. The thiol pretreated aggregates resulted in variable desorption behavior. Slight enhancements of AAK-1 asphalt retention occurred with RC-limestone and RL-gravel but RJ-gravel always showed substantially increased desorption with thiol pretreatment.

Table 9. Evaluation of Organosilane Modified Aggregates for Hydrophobic Asphalt-Aggregate Bonding

<table>
<thead>
<tr>
<th>Asphalt/Aggregate</th>
<th>U Untreated Aggregate Percent Desorption, %</th>
<th>M Modified Aggregate Percent Desorption, %</th>
<th>Percent Bonding(^a) Enhancement with Modifier, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hydrocarbon</td>
<td>Thiol</td>
<td>Hydrocarbon</td>
</tr>
<tr>
<td>AAD-1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-limestone</td>
<td>-3.82(^b)</td>
<td>+0.28(^c)</td>
<td>-10.8</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td>-0.37</td>
<td>+8.23</td>
<td>+55.0</td>
</tr>
<tr>
<td>RL-gravel</td>
<td>-33.9</td>
<td>+8.58</td>
<td>-45.7</td>
</tr>
<tr>
<td>RC-limestone</td>
<td>-4.5</td>
<td>+5.7</td>
<td>+49.2</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td>-25.7</td>
<td>-6.0</td>
<td>-38.7</td>
</tr>
<tr>
<td>RL-gravel</td>
<td>-3.39</td>
<td>-8.87</td>
<td>+36.8</td>
</tr>
<tr>
<td>RC-limestone</td>
<td>+12.6</td>
<td>-6.86</td>
<td>-1.75</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td>-3.39</td>
<td>-8.87</td>
<td>+36.8</td>
</tr>
<tr>
<td>RL-gravel</td>
<td>-17.3</td>
<td>-7.47</td>
<td>-32.2</td>
</tr>
<tr>
<td>AAK-1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-limestone</td>
<td>-0.81</td>
<td>-7.66</td>
<td>+2.59</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td>+68.5</td>
<td>-13.0</td>
<td>-8.39</td>
</tr>
<tr>
<td>RL-gravel</td>
<td>-7.29</td>
<td>-12.0</td>
<td>+2.46</td>
</tr>
<tr>
<td>RC-limestone</td>
<td>-7.5</td>
<td>-6.9</td>
<td>-5.1</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td>+50.1</td>
<td>-7.2</td>
<td>-7.3</td>
</tr>
<tr>
<td>RL-gravel</td>
<td>-6.7</td>
<td>-8.3</td>
<td>-6.8</td>
</tr>
<tr>
<td>RC-limestone</td>
<td>-6.33</td>
<td>-12.2</td>
<td>-6.58</td>
</tr>
<tr>
<td>RJ-gravel</td>
<td>+27.1</td>
<td>-13.2</td>
<td>-9.35</td>
</tr>
<tr>
<td>RL-gravel</td>
<td>-10.1</td>
<td>-12.8</td>
<td>-7.34</td>
</tr>
</tbody>
</table>

\(^a\) (\text{M-U})100 = \% bonding enhancement
\(^b\) = decrease in amount asphalt desorbed
\(^c\) = increase in amount asphalt adsorbed
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5. SUMMARY AND CONCLUSIONS

Modification of the aggregate and the changes that occurred in the adsorption behavior point to the conclusion that the asphalt-aggregate interaction is highly specific. The amount of adsorption and desorption by water that occurs is dependent on the chemistry between the asphalt and the aggregate surface. Model component adsorption and desorption experiments conclusively showed that polar compounds adsorbed at different levels of affinity for different aggregates. The same was true for their desorption behavior. Some polar compounds were more easily removed from aggregates than others and the amount by which they were removed was dependent on the aggregate chemistry as well as the pH and heat history of that particular system. Net adsorption studies between a series of aggregates and a set of three asphalts clearly indicated that the aggregate chemistry controlled the adsorption behavior. Although small differences were observed among the adsorption and desorption behavior of the asphalts, the differences among the aggregates were substantial. Silane pretreatment of aggregates using specific chemical groups resulted in enhancement of adsorption and retention of asphalt models and asphalts in specific cases while in others no change or lesser adsorption and increased water sensitivity occurred. Once the aggregate surface was modified the chemistry of the asphalt appeared to be more specific in both its adsorption and desorption behavior.
Thermal Fatigue Cracking of Asphalt Concrete Pavements
An Experimental Approach

N Mike Jackson
Graduate Research Assistant
Oregon State University
U S A

T S Vinson
Professor of Civil Engineering
Oregon State University
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and

Vincent Janoo
Research Civil Engineer
USA CRREL
U S A
THERMAL FATIGUE CRACKING OF ASPHALT CONCRETE PAVEMENTS
AN EXPERIMENTAL APPROACH

by

N. M. JACKSON, T. S. VINSON, and V. JANOO

ABSTRACT

Asphaltic concrete pavements are believed to experience fatigue related failure due to internal stresses caused by cyclic fluctuations in ambient temperatures. Based on field observations, cracking often occurs at temperatures above the established restrained fracture temperature of the mixture. The primary mechanism generally attributed to this type of failure is thermal fatigue. Thermal fatigue may lead to considerably shortened service life of asphaltic concrete pavements and costly maintenance requirements if not accounted for in the initial pavement design.

Conventional mix design procedures such as the Marshall or Hveem methods do not adequately simulate or measure the thermal fatigue resistance of asphalt-aggregate mixtures. Thus, in regions susceptible to broad daily fluctuations in ambient temperature, it is desirable to incorporate into the mix design a test procedure which will simulate and measure the thermal fatigue response of the pavement.

As part of the Strategic Highway Research Program (SHRP), monotonic and cyclic thermal stress tests are performed on asphalt concrete beam specimens in an effort to simulate actual field conditions. The laboratory tests are conducted at Oregon State University and the USA CRREL facilities. In this study, restrained asphalt concrete beam specimens are subjected to thermally induced tensile stresses and fatigue cracking responses are monitored relative to different asphalt-aggregate mixtures. Tests are performed with state-of-the-art testing equipment and computer assisted, closed-loop data acquisition systems.

Various mixture and binder properties obtained from existing laboratory test procedures are evaluated with respect to the test results obtained in this study and a methodical approach to the prediction of thermal fatigue performance in areas of extreme thermal cycling is presented. Attempts are also made to validate the laboratory results with respect to performance of existing pavements and test strips documented within the United States.
1.0 INTRODUCTION

1.1 Problem Definition

Asphalt concrete pavements may be susceptible to fatigue failure when subjected to changes in ambient temperatures. Observations of pavement performance in the field indicate cracking often occurs at temperatures above the critical fracture temperature of the binder. The primary mechanism generally attributed to this type of failure is thermal fatigue. In moderate climates, thermal stresses are typically not great enough to cause immediate fracture, but it is conceivable that cyclic temperature variations above the fracture temperature may induce fatigue in the pavement and make it more susceptible to cracking under subsequent thermal and/or traffic induced stresses (Gerritsen, et al., 1988).

Thermal fatigue may lead to considerably shortened service life of asphalt concrete pavements and costly maintenance requirements if not accounted for in the initial pavement design.

Conventional mix design procedures such as the Marshall or Hveem methods do not address the thermal fatigue resistance of asphalt-aggregate mixtures. Thus, in regions susceptible to broad daily fluctuations in ambient temperature, such as the southwestern United States, it is desirable to incorporate a test procedure which will estimate thermal fatigue resistance of the pavement into the mix design. Several tests which are potential indicators of thermal fatigue response in pavements were identified through a national survey questionnaire conducted by the U.S. Army Cold Regions Research and Engineering Laboratory (USA CRREL) and an exhaustive literature review (Vinson, et al., 1989). The most promising tests are currently under evaluation at Oregon State University (OSU) and the USA CRREL. These tests include the Thermal Stress Restrained Specimen Test (TSRST), the Direct Tension Test under constant rate of extension (Haas, et al., 1973), the C*-Line Integral test (Abdulshafi, 1983), and direct and indirect tensile creep tests. The TSRST is presently considered to be the most promising test procedure to identify/validate the relationships between asphalt binder properties and pavement performance.

1.2 Purpose of Paper

The research described herein is part of the Strategic Highway Research Program (SHRP) Project A-003A "Performance-Related Testing and Measuring of Asphalt-
Aggregate Interactions and Mixtures.” The primary objective of A-003A is to validate the relationships between asphalt binder properties and pavement performance. A secondary objective is to develop accelerated mixture performance test procedures to be incorporated into standard design specifications. The goal of this study, a sub-task of A-003A, is to identify a suitable laboratory test program which will provide an estimate of the thermal fatigue fracture resistance of asphalt concrete mixtures.

2.0 BACKGROUND

2.1 Thermal Fatigue Cracking

Cracking that results from thermal cycling above the critical fracture temperature of the pavement binder is generally referred to as thermal fatigue cracking. Thermal cycling at relatively low temperatures produces cyclic tensile stresses within the pavement due to volumetric contraction of the material. The temperature range which thermal fatigue has been considered to be critical is estimated to be between about -7 C (20 F) and 21 C (70 F) (Carpenter, 1983). Above this range, thermal stresses cannot be sustained in the pavement due to visco-elastic behavior of the binder, and below this range, low temperature cracking (immediate failure) is believed to be the dominant mode of distress. The low end of this range is debatable, however, and may extend to temperatures as low as -25 C (-13 F) for mixtures containing soft grade asphalts or modifiers.

Load cycle fatigue tests performed in the laboratory have demonstrated that fatigue phenomena can occur in asphalt-aggregate mixtures under slow load frequencies (Gerritsen, et al., 1988). Thermal cyclic fatigue tests performed on asphalt-aggregate mixtures indicate binder properties tend to control thermal fatigue response in mixtures. Soft grade asphalt tends to resist thermal fatigue failure to a greater degree than harder grade asphalt (Sugawara and Moriyoshi, 1984). The failure mechanism associated with thermal cracking is considered to be predominantly tension related. Typical aggregates used in asphalt concrete pavements have minimal influence in tensile failure at low temperatures as long as the aggregate is sound and does not alter the properties of the asphalt cement through selective or excessive absorption (Carpenter, 1983).

Thermal fatigue fracture is generally evaluated from either a mechanistic or phenomenological approach (Vinson, et al., 1989). The phenomenological approach generally attempts to model actual loading conditions as closely as possible. With respect to fatigue failure, this approach typically consists of extrapolation of the number of cycles required to cause failure in laboratory fatigue tests to actual pavement distress. Miner’s hypothesis, (1945) is commonly applied in phenomenological fatigue analyses. In general, Miner’s hypothesis suggests that fatigue damage is cumulative and fatigue failure occurs when the sum of the ratios of fatigue life expended at incremental stress or strain levels are equal to or greater than unity. Miner’s hypothesis may be expressed in the following form:

\[
\sum_{i=1}^{n} \frac{n_i}{N_i} = 1
\]  

(2.1)
where,
\[ n_i = \text{number of load applications at level i} \]
\[ N_i = \text{number of load applications to failure at level i} \]

This relationship has historically been applied to relate pavement distress as determined in the laboratory to field performance. Due to effects which may not be accounted for in the laboratory, "shift" factors are typically required to predict field performance from laboratory test results.

Mechanistic models generally attempt to relate pavement distress to fundamental material properties such as mixture stiffness, strain energy, and fracture parameters such as the critical Stress Intensity Factor (\( K_{IC} \)), the J-Integral or the C*-Line Integral. Majidzadeh (1970) and Salam (1971) applied a quantitative relationship for crack growth in asphalt concrete with respect to the number of load applications with the following equation:

\[
\frac{da}{dN} = A K_I^n
\]  
(2.2)

where,
\[ a = \text{crack length} \]
\[ N = \text{fatigue load cycles} \]
\[ A, n = \text{material constants} \]
\[ K_I = \text{Stress Intensity Factor (mode 1 case only)} \]

The constants in equation 2.2 were estimated from regression analysis based on limited laboratory test data. Salam notes the merits of this approach include (1) it permits a quantitative measure of the fatigue process, and (2) it is flexible enough to be adopted on a rigorous microscopic level or a phenomenological basis to design. However, this approach requires a considerable amount of experimental data which is not available at present. A similar relationship utilizing the J-Integral and elastic strain energy was employed by Abdulshafi and Majidzadeh (1985). The form of this relationship is as follows:

\[
\frac{da}{dN} = A \left( \frac{J_I}{2U_e} \right)
\]  
(2.3)

where,
\[ a = \text{crack length} \]
\[ N = \text{fatigue load cycles} \]
\[ A = \text{material constant} \]
\[ U_e = \text{elastic strain energy} \]
\[ J_I = \text{J-Integral (mode 1 case only)} \]

Experimental results indicate this relationship may provide similar predictions for fatigue life as the Stress Intensity approach, however, additional laboratory testing is necessary to establish a relationship between the two methods. The critical Stress Intensity Factor and J-Integral are typically associated with elastic or elasto-plastic material behavior. Another fracture mechanics parameter, the C*-Line Integral has been related to fatigue fracture of asphalt concrete mixtures by Abdulshafi (1983). The relationship applied by
Abdulshafi to correlate fatigue crack growth to the C*-Integral is as follows:

\[ \frac{\ln da}{dN} = \ln R + q \ln C^* \]  

(2.4)

where

- \( a \) = crack length
- \( N \) = fatigue load cycles
- \( R, q \) = material constants
- \( C^* \) = C*-Line Integral

As with the previously mentioned fracture mechanics parameters, laboratory test data pertaining to the C*-Line Integral is limited at this time. However, the data presented by Abdulshafi suggests the C* parameter may be appropriate for evaluation of crack resistance of asphalt concrete mixtures.

In an attempt to account for crack propagation and ultimate fracture, Majidzadeh, et al., (1976) developed a mechanistic model based on linear elastic fracture mechanics. Lytton and Shanmugham, (1982) also developed a mechanistic fracture mechanics model to predict the number of thermal cycles required to produce fracture in a pavement. The Majidzadeh model was evaluated based on equations 2.2 and 2.3 and the Energy Release Rate parameter, \( J_{IC} \), was identified as the most appropriate parameter for characterization of elastic and elasto-plastic fracture (Abdulshafi and Majidzadeh, 1985). Crack initiation was not accounted for in these analyses but it was noted that crack initiation could be evaluated based on the number of cycles required to develop the initial crack independently of crack propagation. The Energy Release Rate associated with creep crack growth, the C*-Line Integral, was successfully applied to differentiate the performance of modified asphalt concrete mixtures with respect to load fatigue cracking distress at moderate temperatures (Abdulshafi and Kaloush, 1988). Abdulshafi and Kaloush suggest the C*-Line Integral may also be successfully applied to the study of thermal fatigue cracking.

The maximum strain energy may also be used to predict thermal fatigue fracture in asphalt concrete mixtures. The maximum strain energy may be obtained from the Indirect Tension Test or Direct Tension Test (Haas, et al., 1977). Maximum Strain Energy Theory may be applied to evaluate the energy or work required to induce fracture. Under uniaxial tension, the strain energy density is given by the following expression:

\[ U = \frac{1}{2} \sigma \varepsilon \]  

(2.5)

where,

- \( U \) = strain energy density
- \( \sigma \) = ultimate stress
- \( \varepsilon \) = ultimate strain

The strain energy density is the area below the stress versus strain curve. Mahboub (1985), and Little and Mahboub (1985) reported that results from the diametral Indirect Tensile Test did not satisfactorily differentiate the low temperature fracture potential of plasticized sulfur binders. In addition, Abdulshafi and Kaloush (1988) reported that Indirect
Tensile Test results could not be used to clearly differentiate the performance of modified mixtures. However, the conclusions drawn in these studies were based on the measured tensile strength of the mixes and the strain energy associated with fracture was not considered. Based on Maximum Strain Energy Theory, it is hypothesized that an analysis of the strain energy required to induce fracture under tensile loading may provide a suitable approach to ranking asphalt concrete mixtures with respect to thermal fatigue resistance. Possible correlations with ultimate tensile strength, ultimate tensile strain, and secant modulus may also be meaningful.

2.2 Factors Influencing Thermal Fatigue Response

The primary factors which influence thermal fatigue response may be divided into three categories: (1) material factors, (2) environmental factors, and (3) geometric factors (Vinson, et al., 1989). The material factors addressed in this study are (1) asphalt cement properties, and (2) voids content. The primary environmental factor of interest in this study is temperature. Other variables are assumed to remain constant. Geometric factors have been addressed in a parallel study at OSU of low temperature cracking of restrained specimens under conditions of monotonic cooling.

2.2.1 Asphalt Cement Properties

The asphalt cement is believed to be the most critical factor affecting thermal fatigue response in pavements. A lower viscosity grade of asphalt cement will produce a lower rate of increase in stiffness with decreasing temperature and consequently reduce the potential for thermal cracking. In other words, soft grade asphalt cements tend to resist thermal fatigue failure to a greater degree than harder grade asphalt cements (Sugawara and Moriyoshi, 1984). The temperature susceptibility of the asphalt cement is also critical when evaluating the response of mixtures over a range of temperatures. The relative ranking of the stiffness properties of asphalt cements will often change with temperature. Thus, it is important to possess rheological data over the range of temperatures of interest. Knowledge of the coefficient of thermal contraction of the mix is also of interest. Asphalt concrete mixtures with relatively large contraction/expansion coefficients will realize greater internal stresses than those with lower coefficients.

2.2.2 Voids Content

The voids content of an asphalt concrete pavement provides an indicator of the degree of compaction imparted to the pavement. In general, existing test data indicate that specimens containing fewer voids will exhibit lower fracture temperatures than specimens with greater void contents with all other factors held constant (Sugawara and Moriyoshi, 1984). The effects of voids content was found to be relatively minor with respect to thermally induced stresses at temperatures above the glass transition temperature. However, below this transition temperature, voids content was found to be significant.

2.2.3 Ambient Temperature

As the surface temperature decreases, the resulting stresses within a restrained asphalt concrete pavement tend to increase. In addition, asphalt-aggregate mixtures tend to exhibit more plastic behavior than elastic behavior as the temperature is lowered. Below the glass transition temperature, by definition, the mixture responds as a purely brittle
material. The zone of interest in thermal fatigue fracture may bracket the glass transition
temperature of the binder.

3.0 EXPERIMENTAL TEST PROGRAM

3.1 Selected Laboratory Tests

Researchers at USA CRREL and OSU are currently performing phenomenological
thermal fatigue tests on selected asphalt concrete mixtures obtained from the SHRP
Material Resource Library (MRL). This testing is being conducted with the TSRST
apparatus shown in Figure 1. The TSRST is typically performed under conditions of
monotonic cooling. However, only slight modifications to the test procedure and equipment
are necessary to perform cyclic testing. An example of typical thermal fatigue test data
from the USA CRREL is provided in Figure 2.

The data presented in Figure 2 exhibits a relatively rapid decrease in thermal
stresses in the initial stages of the test and relatively gradual decrease thereafter. This
behavior is characteristic of thermal fatigue test data and has been documented by other
researchers (Sugawara, et. al., 1984). The response shown is commonly referred to as
stress relaxation and results from the visco-elastic properties of the mixture as well as the
formation of micro-cracks in the test specimen. This complex behavior contributes to the
difficulty in developing a realistic model to characterize thermal fatigue.

As was stated previously, the phenomenological TSRST has been identified as the
most promising test method to validate the relationship between asphalt binder properties
and pavement performance with respect to thermal effects. However, the time required
to perform the fatigue test may be excessive for practical applications. Thus, research is
also being performed to identify a test of practical duration which may be used as an
indicator of thermal fatigue response. Tests which are currently being evaluated at OSU
include the TSRST under conditions of monotonic cooling with a stress relaxation phase,
direct and indirect tensile creep tests, direct tension under constant rate of extension, and
the C*-Line Integral test procedure.

3.2 Experiment Design

In order to evaluate the above-referenced test procedures with respect to thermal
fatigue, an experiment design was developed around two of the SHRP MRL asphalt
grades of significantly different temperature susceptibility and stiffness (AAG1 and AAK2).
A typical, non-moisture susceptible aggregate was used (RB). Test samples were
prepared with the California kneading compactor and two levels of voids (4% and 8%)
were considered. A full factorial experiment design was outlined for each candidate test.
In addition, all tests were replicated.
3.3 Materials

3.3.1 Mineral Aggregate

Mineral aggregate from a single source was used in this study. The aggregate utilized here is identified by the SHRP MRL as RB. The RB aggregate is described as a relatively non-stripping aggregate and predominantly consists of crushed granite. The physical properties of the RB aggregate, as provided by MRL, are summarized in Table 1.

3.3.2 Asphalt Cement

Two asphalt cements were used to evaluate the suitability of the test procedures applied here to differentiate mixture response. The asphalt cements considered are identified by MRL as AAG1 and AAK2. The AAG and AAK designations refer to crude sources with differing temperature susceptibility characteristics and the numeric designations refer to different asphalt grades from the given sources. Two additional asphalt grades (AAG2 and AAK1) were included in the evaluation of the C*-Line Integral test program for analytical purposes. The primary binder properties, as provided by MRL are summarized in Table 2.

4.0 TEST RESULTS

The full test program is not complete at this time. However, preliminary test results have revealed some interesting findings. The phenomenological TSRST procedure is still considered to be the best indicator of thermal fatigue response in asphalt concrete pavements. However, as stated previously, the time requirements are generally not acceptable (2 to 6 weeks). The TSRST will continue to be a useful tool for validation purposes in the research environment. The preliminary results of the alternative tests with respect to the evaluation of thermal fatigue are presented in the following paragraphs. The test programs for the C*-Line Integral test and the Direct Tension test with constant rate of extension have been completed at this time. The TSRST with relaxation and tensile creep test programs are still under evaluation.

In general, the C*-Line Integral test procedure, as performed in this study, appears to be inappropriate for the evaluation of low temperature distress. The direct tension test exhibits somewhat promising results when analyzed with respect to strain energy at fracture. The C*-Line Integral test procedure appears to result in a relative ordering of the mixtures with respect to resistance to fracture which is inconsistent with results obtained in previous studies. The C*-Line Integral test data obtained in this study is summarized in Figures 3 and 4. Based on data presented by other researchers, soft grade asphalt should generally resist thermal fatigue failure to a greater extent than harder grade asphalt (Sugawara and Moriyoshi, 1984). In other words, holding all other variables constant, the AAK2 mixture should consistently exhibit greater resistance to cracking than the AAG1 mixture. Since the C*-Line Integral can be interpreted as an energy release parameter, the slopes of the curves provided in Figures 3 and 4 exhibit the relative energy required to cause cracking in the respective test specimens. Analysis of the test data presented here indicates that the only statistically significant effect observed for the C*-Line Integral
versus crack growth is temperature. No significant effects were found for either asphalt source or voids content (Painter, 1990). In addition, several mixtures could not be evaluated with the C*-Line Integral test procedure as applied herein due to extremely rapid crack propagation. As shown in Figure 4, only three of the mixtures tested at 10°C could be evaluated with the C*-Line Integral procedure. At relatively low temperatures, generally in the temperature range of interest for thermal fatigue cracking, the controlling fracture parameter is most likely the J-Integral, Jc or more likely the Stress Intensity Factor, Kc. The C*-Line Integral Test procedure followed for this study does not readily provide sufficient small scale crack opening data for the determination of these parameters. The C*-Line Integral test does appear to be an appropriate test for investigating crack growth in asphalt concrete mixtures at moderate temperatures (Abdulshafi, et. al., 1990). However, at temperatures below about 20°C (88°F), the results appear to be inconsistent with established patterns. Significantly decreasing the test displacement rates may improve test response at lower temperatures. However, test procedures would still have to be refined to account for the extremely rapid crack growth rates which occur at lower temperatures (Abdulshafi, et. al., 1990).

The Direct Tension test data appears to provide a more reasonable indication of thermal fatigue response than the C*-Line Integral data. Based on the relative ranking of the test data with respect to direct tensile strength and strain energy of the beam specimens tested, it appears the strain energy required to induce fracture is the more reliable parameter for correlation with low temperature performance of the mixtures tested. The ultimate tensile strengths of the various mixtures tested in this study are presented in Figure 5. The strain energies required to induce fracture in the same mixtures are presented in Figure 6. Ranking of the mixtures based on strain energy tends to be generally consistent with the low temperature properties of the binder (critical temperature). In other words, the soft grade asphalt tends to require greater strain energy (more work) to produce fracture than the harder grade asphalt (AAG1). The test results indicate, at constant voids, the energy required to fracture the specimens decreases with decreasing temperature. Test results also indicate, at constant temperature, the energy required to fracture the specimens decreases to some extent with increasing voids. However, the effect of voids is not as significant as that of temperature. These findings are consistent with the results presented by other researchers.

Ranking of the mixtures based on direct tensile strength is generally not satisfactory as shown in Figure 5. This conclusion is not surprising, however, based on previous research efforts. Test data presented by Mahboub (1985), Little and Mahboub (1985), and Abdulshafi and Kaloush (1988) indicates tensile strength as determined from indirect tension testing does not clearly differentiate between mixtures. Thus, attempts to rank the mixtures based solely on tensile strength will most likely yield misleading results.

5.0 CONCLUSIONS

A number of candidate tests to identify the relative thermal fatigue resistance of an asphalt concrete mixture are currently being evaluated under the SHRP A-003A project. The most promising test procedure is a phenomenological fatigue test conducted with the TSRST apparatus. However, this test may require periods of 2 to 6 weeks to perform. Thus, several alternative tests have been identified and are currently being evaluated at
The thermal fatigue test with the TSRST apparatus nevertheless appears to be an excellent validation tool for research purposes.

The preliminary results from the OSU study suggest a possible relationship between the energy absorption capacity (strain energy required to induce fracture) of asphalt concrete specimens under direct tension and low temperature performance of the specimens. Attempts to correlate the energy rate integral (C*-Line Integral) and direct tensile strength to low temperature performance of the mixtures were unsuccessful. Correlations between tensile creep test results and the TSRST with relaxation are being developed but have not been completed. The results of these tests are anticipated in the coming months and the most suitable test method for the evaluation of thermal fatigue will be presented at that time.

REFERENCES


### Table 1  Physical Properties of RB Aggregate (from MRL).

<table>
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<tr>
<td>Apparent</td>
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### Table 2  Summary of Asphalt Cement Properties (from MRL).

<table>
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<th>AAK2</th>
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<tr>
<td>Asphalt Grade</td>
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<td>AC-10</td>
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**Original Asphalt Properties**

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<td>Penetration</td>
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<tr>
<td>R&amp;B Softening Point, F</td>
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**Residual Properties**

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<td>Viscosity (140 F, Poises)</td>
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<tr>
<td>Viscosity (275 F, Cst)</td>
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**Component Analysis**

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<td>Saturates %</td>
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Figure 1  Schematic of TSRST apparatus (after Duhwoe, 1990).

Figure 2  Typical thermal fatigue test data from USA CRREL.
Figure 3  Summary data showing C*-Line Integral versus crack growth rate for different asphalt concrete mixtures at 20 C.

Figure 4  Summary data showing C*-Line Integral versus crack growth rate for different asphalt concrete mixtures at 10 C.
Figure 5  Mean ultimate tensile strength for different asphalt concrete mixtures.

Figure 6  Mean strain energy at fracture for different asphalt concrete mixtures and temperatures.
Development of Test Methods for a Performance-Related Bitumen Specification

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DEVELOPMENT OF TEST METHODS FOR A PERFORMANCE-RELATED BITUMEN SPECIFICATION

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1. INTRODUCTION - CHAPTER 1

One of the primary objectives of the Strategic Highway Research Program (SHRP) is the development of a new specification for bitumen (1). In North America, both penetration-graded (ASTM D 946) and viscosity-graded (ASTM D 3381) specifications are used to specify paving-grade bitumen (2). These specifications, which are largely based on empirical properties such as penetration and ductility, grade the bitumen according to its consistency at 25 °C and 60 °C respectively. Because they are empirical, the criteria in the current penetration-graded or viscosity-graded specifications do not relate mechanistically to the distress mechanisms that are prevalent in highway pavements. As a consequence, it is impossible to use the current empirical specification criteria as a basis for the development of a performance-related specification. Therefore, in order to develop new performance-related specifications for bitumen new or improved specification criteria and associated test methods are required. The development of such criteria and test methods is one of the major tasks within the SHRP asphalt research program.

The purpose of this paper is to describe the specification criteria and test methods that have been developed for the new SHRP binder specification. The development of these specification criteria and test methods was conducted primarily at The Pennsylvania Transportation Institute of The Pennsylvania State University as part of the SHRP research contract, A-002A "Binder Characterization and Evaluation". Penn State University is serving as a subcontractor to Western Research Institute, the primary contractor for the A-002A research contract.

2. APPROACH TO THE PROBLEM - CHAPTER 2

2.1 Strategy Used in Developing Specification Tests

The A-002A Binder Characterization and Evaluation contract was the first SHRP asphalt program research contract to be awarded and therefore it was necessary for the A-002A contractor to establish a strategy with respect to the development of the specifications
and their relationship to pavement performance. Because the new specifications were to be performance-related, the A-002A research team concluded that the specifications should be driven by the critical distress mechanisms as reported in the field. Further, it was concluded that the specification criteria should be based upon fundamental, performance-related parameters to facilitate the development of the needed relationships between the specification criteria and pavement performance. The overall strategy used in the development of the specification tests is illustrated in figure 1.

A performance-related specification should assure the owner agency of a certain level of performance. The strategy illustrated in figure 1 was designed to address the critical failure mechanisms as reported in the field and to provide test methods and data so that the specification criteria can be related to performance.

2.2 Properties Selected for Use in the Specification

In the design of most civil engineering structures two basic engineering material properties are important: the stress-strain response of the material and the failure or fracture properties of the material. As an example, in the design of a reinforced Portland cement concrete beam, the design engineer uses the modulus of elasticity of the concrete to calculate the deflection of the beam. The design engineer must also know the design strength of the concrete to ensure that the beam will not crack under load. In a similar manner, both the stress-strain behavior and strength of bitumen must be considered.

The stress-strain response of bitumen is very complex because the effect of both time of loading and loading temperature must be considered. Empirical test methods, such as penetration and softening point, do not adequately separate or consider the effects of time and temperature. Therefore, a more fundamental method of characterizing the stress-strain-time-temperature behavior of bitumen is required for the new specification. Based upon data in the literature and the experience of the research team, a linear viscoelastic characterization (LVE) of bitumen was selected as the basis for the development of the new specification. Although a wide variety of models are available for characterizing the stress-strain response of time and temperature dependent materials, the LVE model was chosen as the best compromise between an accurate representation of the behavior of bitumen and the need for a model that can be readily implemented within a specification.

The characterization of the failure or fracture behavior of time-dependent materials is also very complex. As part of the A-002A research program, a variety of test methods are being used to characterize the fracture and failure properties of bitumen to include fatigue, fatigue crack propagation, and fracture mechanics testing. Although these test methods and their analytical techniques provide a fundamental representation of material
Step 1: Determine critical distress modes that control pavement performance

Step 2: Identify fundamental material properties that relate to the critical distress modes, thus assuring performance-related material properties

Step 3: Identify test methods that generate the fundamental performance-related material properties

Step 4: Seek surrogates where the fundamental performance-related properties are not suited for specification use

Figure 1. Flow diagram illustrating the strategy used in the development of the SHRP binder specifications.
behavior, they were considered by the A-002A team as too complex for general specification use. Consequently, the direct tension test was selected as a test method for the characterization of the failure properties of bitumen.

Thus, in the new SHRP binder specification two basic characterizations of the behavior of bitumen are used: an LVE characterization of the stress-strain-time-temperature response and a direct tension characterization of failure behavior.

2.3 Stress-Strain Behavior of Bitumen

For bitumen, the coefficient of proportionality, or modulus, that relates stress and strain must account for both time and temperature dependency. The response of bitumen to an applied load is illustrated in figure 2 where a shear load is applied to bitumen sandwiched between two parallel plates in a traditional creep test. Three distinct types of deformation result from the application of the step load. At the moment when the load is applied there is an instantaneous elastic deformation of the bitumen. This is followed by a delayed elastic deformation. Both the instantaneous elastic and the delayed elastic deformation are recoverable. The instantaneous elastic deformation is recovered immediately upon the removal of the load whereas the delayed elastic deformation requires some time before it is recovered. The viscous component of the deformation is the third component of the deformation. This component, which for a linear viscoelastic material is the result of simple Newtonian flow, is operable over the entire time of loading, \( t > t_o \), and represents the non-recoverable portion of the deformation.

Mathematically, the strain can be related to the applied stress in the following form (3):

\[
\frac{1}{S(t)} = \frac{1}{S_g} + \int_{-\infty}^{\infty} L(1-e^{-\tau/\tau}) d \ln \tau + t/\eta
\]  

(1)

where

- \( S_g \) = glassy stiffness
- \( L(1-e^{-t/\tau}) \) = relaxation spectrum
- \( t \) = real time
- \( \tau \) = relaxation time
- \( \eta \) = coefficient of Newtonian viscosity

Stiffness is expressed as a function of time, \( S(t) \), when quasi-static (creep) tests are conducted or in terms of angular frequency, \( S(\omega) \), when the applied load is oscillatory (sinusoidal). The time of loading, \( t \), in a creep test is approximately equal to the reciprocal of the angular velocity, \( \omega \), in an oscillatory test.
Figure 2. Typical creep test for paving grade bitumen: left - stress and strain as a function of time; right - sliding plate test geometry.
The shape of the deformation curve, shown in figure 2, is highly dependent upon the temperature at the time of loading. At very short loading times, or at low temperatures, the elastic and delayed elastic portion of the curve will dominate and there will be little viscous flow. In contrast, at high temperatures the elastic and delayed elastic component will contribute little to the deformation and the majority of the deformation will be the result of viscous flow. Indeed, at temperatures approaching 60 °C, most unaged bitumens used in pavement construction behave as simple Newtonian fluids and can be characterized by a single coefficient of viscosity, $\eta$, and any contribution from instantaneous or delayed elasticity can be neglected. Only when this condition of Newtonian flow is approached can the capillary-tube viscometer, as is used in the viscosity-graded specifications, be used to characterize the flow properties of bitumen.

2.4 Temperature Dependency of the Stress-Strain Behavior of Bitumen

The simple creep curve, as illustrated in figure 2, is acceptable for describing the flow behavior of bitumen over a limited range of times and temperatures. However, because bitumen is highly time and temperature dependent and the stiffness modulus varies over a very wide range of values, a logarithmic representation of the data, as shown in figure 3, is required.

![Figure 3. Typical creep response for asphalt cement (log-log scale), with important parameter shown diagrammatically.](image)
To further simplify the characterization of bitumen it is convenient to invoke the simple interrelationship between time and temperature as illustrated in figure 4. Using this principle of time-temperature superposition, the curves obtained at different temperatures may be adjusted along the time axes to produce a single curve (4). This simple shifting of the data produces a single master curve at a single reference temperature. In this manner, a series of shift factors, log aT, are developed. The shift factors are simply the distance along the time axes that the individual curves must be shifted in order for the individual curves to become coincident with the master curve.

By invoking the time-temperature superposition principle it is possible to characterize the stiffness modulus for the binder at any time and temperature. It is important to note that this characterization requires two curves: the master curve at some reference temperature, generally 25 °C, and a shift function; both of these curves have been included in figure 4.

2.5 Representation of LVE Master Curve for Bitumen

Referring to figure 3, some key characteristics of the master curve can be observed:

- The general shape of the curve is that of a hyperbola.
- All bitumens obtain a common glassy or elastic stiffness as shown by the asymptote (glassy modulus).
- At long loading times (or high temperatures) the master curve reaches an asymptote which is described by a Newtonian coefficient of viscosity, η.
- At intermediate loading times the glassy asymptote and the viscous asymptote intersect at t₀, the crossover time; the slope of the master curve at this point, on a log-log scale, is equal to 1/2.

The location of the viscous asymptote of the master curve is defined by the steady-state coefficient of viscosity, as described above. The position of the master curve along the time axis is characterized by the point of intersection of the glassy and viscous asymptotes. This point on the time axis is called the crossover time t₀, and can be thought of as the location parameter for the master curve. The shape of the master curve is determined by the rheological index, R. This parameter is defined as the distance between the glassy modulus and the modulus at the crossover time, on a log-log scale. The R value is related to the distribution of the relaxation times and therefore to the distribution of the polar bonding forces within the binder (5).
2.6 Characterization of Failure Behavior of Bitumen

At very low temperatures bitumens behave in a brittle manner with a linear stress-strain relationship to failure. In this region the strain at failure is relatively small, typically less than 1 or 2%. As the test temperature is increased, or as the rate of the elongation is decreased, the bitumen fails in a ductile manner and undergoes a transition to brittle-ductile failure in which the stress-strain curve is no longer linear but for which a definite rupture is observed. At higher temperatures, the bitumen fails in a ductile manner, typical of the traditional ductility tests. The first two regions of flow, the brittle region and the brittle-ductile transition region, are illustrated in figure 5.

As noted above for the stress-strain behavior in shear, all bitumens exhibit a common set of temperature shift factors at low temperatures. When these shift factors are applied to the direct...
tension test results, common master curves for strain to failure and energy to failure are generated as shown in figure 6. The application of the common shift factors to direct tension test results allows the development of master curves with a limited number of test results. Master curves such as those shown in figure 6 follow a Weibull distribution. The location of the failure master curve on the time axes is asphalt-specific. In addition, the strain or energy to failure in the brittle region is also asphalt specific.

3. DISTRESS MECHANISMS ADDRESSED - CHAPTER 3

As noted in figure 1, the first step in the development of the specification was to identify the distress mechanisms. These distress mechanisms were then used to select the material properties upon which the new specification is based. Three primary distress mechanisms were identified: rutting in the upper layer of the asphalt concrete, thermal shrinkage cracking, and fatigue cracking. Because these distress mechanisms also relate to the climatic conditions and traffic levels, provisions for these factors are also included in the specification. Each of these mechanisms are addressed separately in the new SHRP binder specification.

Figure 5. Direct tension test data for typical bitumen.
3.1 Rutting

Rutting in the upper pavement layers is caused by the accumulated plastic deformation in the mixture that results from the repeated application of traffic loading. Although the rutting tendencies of a pavement are primarily influenced by aggregate and mixture properties, the properties of the binder are also important. This is particularly true for polymer-modified bitumens which are claimed to enhance the rutting resistance of pavements. Finally, rutting is more prevalent at the upper range of service temperatures than at intermediate or low temperatures.

Based on these observations, some measurement of the nonrecoverable deformation of the bitumen at the upper service temperatures and for loading rates commensurate with traffic loading were established as critical to specifying the bitumen with respect to rutting resistance. This led the researchers to recommend the viscous component of the stiffness at 0.1 sec loading time as the critical specification criteria for rutting resistance. Because the viscous component is temperature dependent, the mean of the highest daily pavement temperatures for the hottest month of the year was selected as the reporting temperature. In this manner, the specification criterion for rutting is dependent upon the climatic region.
The viscous component of the stiffness is simply the coefficient of viscosity, $\eta$, divided by the time of loading, $t$, which, in this case, is 0.1 sec. The specification criterion for rutting is included in table 1, which is a summary of the proposed SHRP bitumen specifications. It should be noted that the criteria listed in table 1 should be considered as tentative and subject to change pending the results of validation testing underway in other SHRP projects.

3.2 Thermal Shrinkage Cracking

Thermal shrinkage cracking is a serious problem in much of the northern United States and in most of Canada. Thermal shrinkage cracking occurs when the pavement shrinks as the temperature is lowered. As the temperature of the pavement is lowered tensile stresses are created within the pavement and the when the tensile stresses exceed the tensile strength of the pavement thermal shrinkage cracking is the result (6,7). Therefore, the binder specification must address both the stress-strain-time-temperature response of the binder as well as the fracture properties of the binder.

A fundamental evaluation of the thermal cracking problem might include a consideration of the crack-propagation characteristics of the binder and a fracture mechanics analysis. Although such characterizations are being pursued as part of the A-002A research contract, these characterizations were not considered appropriate for specification use.

For specification use, the strain failure at the minimum pavement service temperature was selected as one of the specification criteria, table 1. This ensures that the pavement will not transcend into the brittle region within its service temperature regime. Other researchers have demonstrated the validity of a limiting stiffness temperature in predicting low-temperature thermal shrinkage cracking (8). The limiting stiffness temperature is merely the temperature at which a specific value of stiffness is obtained after a given loading time. For the purposes of the current version of the specification, table 1, the limiting stiffness temperature was chosen as the temperature at which a stiffness of 200 MPa is obtained after a loading time of 2 hours (8). The thermal cracking criteria are subject to change pending the results of the work currently under way in the A-005 contract in which new and improved models that relate binder and mixture properties to pavement performance are being developed.

3.3 Fatigue Cracking

The selection of specification criteria to assure resistance to fatigue cracking is perhaps the most difficult challenge presented by the new binder specification. First, fatigue cracking generally occurs late in the life of a pavement, requiring the testing of bitumen that is appropriately aged to simulate the long-term in-situ properties of the binder. The selection of
Table 1. SHRP Specifications for Paving Grade Bitumens.

<table>
<thead>
<tr>
<th></th>
<th>AGED ASPHALT BINDER GRADES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PG 1-</td>
</tr>
<tr>
<td></td>
<td>1  2  3  4</td>
</tr>
<tr>
<td><strong>MEAN MAXIMUM PAVEMENT TEMPERATURE</strong>, °C</td>
<td>21 - 27</td>
</tr>
<tr>
<td><strong>LOWEST ANTICIPATED PAVEMENT TEMPERATURE</strong>, °C</td>
<td>&gt; -23 -29 to -23 -34 to -28 -40 to -34</td>
</tr>
<tr>
<td><strong>LOW TEMPERATURE CRACKING (TFOT residue)</strong></td>
<td></td>
</tr>
<tr>
<td>Maximum allowable stiffness at 2 hours at designated temperature, Pa</td>
<td>200 200 200 200</td>
</tr>
<tr>
<td>Minimum failure strain at designated temperature, percent</td>
<td>5 5 5 5</td>
</tr>
<tr>
<td>Designated temperature, °C</td>
<td>-23 -29 -34 -40</td>
</tr>
<tr>
<td><strong>PERMANENT DEFORMATION (TFOT residue)</strong></td>
<td></td>
</tr>
<tr>
<td>Minimum viscous component of stiffness at 0.1 second, at designated temperature, Pa</td>
<td>1400 1400 1400 1400</td>
</tr>
<tr>
<td>Designated temperature, °C</td>
<td>27 27 27 27</td>
</tr>
<tr>
<td><strong>FATIGUE CRACKING (PAV residue)</strong></td>
<td></td>
</tr>
<tr>
<td>Minimum allowable m-value at 0.1 second, at designated temperature</td>
<td>0.6 0.6 0.6 0.6</td>
</tr>
<tr>
<td>Designated temperature, °C</td>
<td>18 16 13 10</td>
</tr>
<tr>
<td><strong>CONSTRUCTABILITY</strong></td>
<td></td>
</tr>
<tr>
<td>Mixing equi-viscous temperature, °C, for a viscosity of 170 ± 20 mPa•s</td>
<td>120 - 176</td>
</tr>
<tr>
<td>Compaction equi-viscous temperature, °C, for a viscosity of 280 ± 30 mPa•s</td>
<td>90 - 160</td>
</tr>
<tr>
<td><strong>SAFETY</strong></td>
<td></td>
</tr>
<tr>
<td>Flash point (COC Flash Point, ASTM D 92), °C</td>
<td>176</td>
</tr>
</tbody>
</table>

*Calculated as the average of the maximum daily pavement temperature for the hottest month of the year.

**NOTE:** The criteria and specification values included in this table are subject to change and are for presentation purposes only. At this time they should not be used for the specification of bitumen.
appropriate specification criteria is further complicated by conflicting evidence regarding the effect of bitumen properties on fatigue performance. The results of laboratory stress-controlled fatigue tests imply that stiffer binders are more resistant to fatigue cracking (9). Conversely, laboratory strain-controlled fatigue testing implies that softer binders are more resistant to fatigue cracking (10).

Ideally, the fatigue properties of the bitumen and the crack propagation properties should be included in the binder specification. However, as noted above, these properties are considered too complex for use in a specification. Therefore, surrogate properties must be selected as the specification criteria. Research conducted by others for polymer-type materials has shown that the slope of the log stiffness versus log time curve, \( m = \frac{d \log S(t)}{d \log t} \), can be correlated with the rate at which cracks are propagated during fatigue (11). The slope, \( m \), is also related to the shape of the master curve, in particular, the R value as illustrated in figure 5. The R value is also related to the distribution of relaxation times which has been shown by others to be related to the toughness of polymeric materials.

Based on the above information, the tentative recommended specification criterion for fatigue is based on the \( m \) value at a loading time of 0.1 seconds and a temperature equal to the estimated mean annual pavement temperature. For these conditions, \( m \) must be less than or equal to 0.5. Including the pavement temperature as part of the specification allows the fatigue criterion to be adjusted according to the climatic region. In a further development of the specification, the \( m \) value may also be adjusted according to the traffic loading.

3.4 Aging

The specification criteria in a true performance-related specification must be representative of the material in the pavement. The binder can "age" during the mixing and compaction process as well as during long-term service. The Thin-Film Oven Test (ASTM D 1754) and the Rolling Thin-Film Oven Test (ASTM D 2872) have been used extensively in the United States and Canada to estimate the "aging" that occurs during mixing and compaction. These tests were continued unchanged into the new SHRP binder specification. Because thermal cracking and rutting occur early in the life of a pavement, the specification criteria for rutting and thermal cracking are based upon TFOT (ASTM D 1754) or RTFOT (ASTM D 2872) residue.

To simulate long-term exposure in the field the pressure aging test was adopted. This test has been used by other researchers and has been modified for the new SHRP binder specification (12, 13). In its current form, standard thin-film oven tests pans are placed in a pressure vessel which contains air at 2 MPa. The pans are held in the pressure vessel for 6 days at 60 °C to 80 °C in order to simulate long-term field exposure. Both long-term field
exposure and the PAV exposure cause complex changes in the rheology of the bitumen. The temperature dependency, shape of the master curve, and the hardness of the bitumen are all affected by long-term aging. Therefore, a simple shift factor such as an aging index is insufficient to determine the rheological properties of aged bitumen. Instead, it is necessary to measure the specification criteria for the aged material, table 1.

4. DETERMINATION OF MATERIAL PROPERTIES - CHAPTER 4

The material properties required for the new SHRP Binder Specification are summarized in table 1. These properties cannot be obtained from the traditional specification criteria such as penetration, softening point, and 60 °C capillary viscosity and therefore new test methods are required (14). The test methods include both the rheology of the bitumen as well as the failure properties of the bitumen.

In the case of the viscous component of the stiffness, it is not possible to obtain this parameter by direct measurement: it must be calculated from the master curve. The remaining rheological and failure parameters can be obtained by direct measurement, however, this would require measurements at a series of different test temperatures and loading conditions. One of the advantages of the new SHRP binder specification is that can accommodate different levels of traffic loading and climatic regions. This is accomplished by adjusting the values of the specification criteria for the climatic region or traffic level.

4.1 Development of a LVE Master Curve for Specification Purposes

Conducting creep tests or dynamic shear tests at a series of test temperatures and generating the master curve as per the method illustrated in figure 4 is too complicated and time consuming for a specification. Instead, an abbreviated method was developed for calculating the master curve (14). Two test methods have been selected for this abbreviated procedure. A simple creep test using a simply supported beam of bitumen and a dynamic shear test are used to characterize the master curve. The essentials of the low-temperature bending beam rheometer are shown schematically in figure 7.

In this test, a simply supported beam of bitumen is loaded at its mid point for 240 s and the deflection of the bean is measured as a function of time. By limiting the strains to less than 1/2 percent and invoking the correspondence principle it is possible to determine the stiffness modulus as a function of time (15). This test is conducted at -15 °C, and from the test data, the slope of the stiffness curve and the stiffness at 240 s is obtained.

In order to obtain the temperature dependency of the bitumen it is necessary to conduct the rheological testing at two test
conducting creep tests, such as the bending beam test just described or a sliding plate shear test, at the upper range of service temperatures in difficult and therefore dynamic measurements have been selected is lieu of static creep measurements.

The dynamic shear test rheometer is also illustrated schematically in figure 7. In this apparatus, a dynamic (oscillatory) shear strain (or stress) is applied to the test specimen at a frequency of 10 rad/s. By monitoring the torque and angular displacement of the specimen it is possible to determine the complex shear modulus, \( G^* \), and the phase angle, \( \delta \). The complex modulus is merely the peak-to-peak stress, \( \tau(\omega) \) divided by the peak-to-peak strain, \( \gamma(\omega) \), figure 8. When a linear viscoelastic material is loaded in an oscillatory or sinusoidal mode, the resulting strain will also be of a sinusoidal function of angular velocity. However, the strain will lag the applied load by some phase angle, delta. If the response of the bitumen is primarily elastic, the phase angle will approach zero. If, on the other hand, the response nearly viscous, the phase angle will approach 90°. Thus, dynamic shear testing at 40 °C yields a complex modulus, \( G^*(\omega) \), and a phase angle, \( \delta \), that is intermediate between 0 and 90.
In summary, the specification-type rheological testing results in a stiffness modulus and the slope of the stiffness modulus-time curve at -15 °C and the complex shear modulus and phase angle at 40 °C. These four parameters are then used through the appropriate modeling to obtain the master curve and the temperature shift function for the binder. The details of this mathematical analysis are beyond the scope of this paper and are presented elsewhere (5,14).

4.2 Development of a Direct Tension Master Curve for Specification Purposes

In the direct tension test, a simple "dog-bone" specimen is tested to failure as shown in figure 7 by pulling the specimen until a peak load is obtained or until the specimen ruptures. From this test the strain to failure, stress to failure, or energy to failure can be determined. Applying time-temperature superposition in the same manner as for the rheological data and using the temperature shift factors obtained from the rheological characterization, a master curve for the strain to failure may be obtained.
In the current specification, the strain to failure value must be less than 5% at the minimum service temperature of the pavement. Final details of the specification test procedure and the specification criteria will depend upon the results of the A-005 contract. In the current scheme, direct tension tests will be conducted at two different temperatures and, using the temperature shift factors from the rheological measurements, a master curve for the strain to failure will be developed. The strain to failure at the minimum pavement temperature can then be determined from the master curve.

4.3 Specification Framework

The new SHRP Binder Specification is based upon both rheological and failure properties of the bitumen. A number of researchers have attempted to simplify the rheological characterization of bitumen through a variety of mathematical models (15,16,17,18). The Penn State research team has extended the work of these earlier researchers to produce a unique LVE characterization for bitumen. This characterization separates the time and temperature dependency of the stress-strain response and accurately models the hyperbolic-like shape of the master curve. The mathematics of this representation are beyond the scope of this paper and are reported elsewhere (5,14).

The rheological properties for the specification are determined using two test procedures; a bending beam rheometer at -15 °C and a dynamic shear rheometer at 40 °C. Using the results of these two tests, a master curve is generated in the form illustrated in figure 3. The generation of the master curve requires several mathematical equations, the details of which are beyond the scope of this paper (14). The equations may be used with a programmable hand-held calculator or may be used with a personal computer. Perhaps the most efficient use of the equations for generating the specification criteria is through the use of a simple PC-based spread sheet program. A flow diagram summarizing the manner in which the test data are used to generate the specification criteria is illustrated in figure 9. It is important to note that the development of the master curve and the details of the required mathematics are transparent to the user. Four measured parameters are used as input to the computer program and the program prints the specification criteria as program outputs.

In its current form, the specification tests must be conducted on both TFOT or RTFOT residue as well as residue from the PAV test. Further development of the specification and a consideration of the results from the other SHRP contractors may allow the use of TFOT or RTFOT residue for the specification of fatigue cracking.

The simple hyperbolic-like representation of the master curve shown in figure 5 is adequate for both unaged and aged non-modified binders. The simple representation is inadequate for some polymer-modified binders which do not obtain a viscous asymptote as per conventional binders. For these materials, the
Figure 9. Flow diagram illustrating process of constructing master curve from specification test data.
As the work is completed on the other SHRP asphalt projects and as work continues on the validation of the numerical values in the proposed specification changes will undoubtedly be necessary. However, a framework for a new specification based on fundamental, performance-related material properties has been successfully developed.

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Characterization of Asphalt by NMR Spectroscopy
and High Performance Gel Permeation Chromatography

P Wyn Jennings
Professor

and

J A S Pribanic
M A Desando
M F Raub
R Moats
T M Mendes
J O Hoberg
J A Smith
F F Stewart

Department of Chemistry
Montana State University
U S A
CHARACTERIZATION OF ASPHALT BY NMR SPECTROSCOPY AND HIGH PERFORMANCE GEL PERMEATION CHROMATOGRAPHY

Department of Chemistry
Montana State University

INTRODUCTION

It is clear that scientists will eventually find physical tests for asphalt which directly relate to asphalt pavement performance. Likewise, scientists will find chemical relationships to both physical tests and road performance. The latter is particularly significant since it would allow for the manipulation of chemical entities to optimize the material's performance for a variety of uses. It is important to add that these tasks are difficult, costly and will take considerable time. There are several different chemical approaches in attempting to solve this problem. One may attempt characterization at either the "macro" level or the "micro" level. The former requires measurements dealing with assemblies of asphaltic chemical components and the latter requires detailed study of its molecular components. Both are necessary and neither one shall be definitive without the other.

In our laboratory, the "macro" level is being probed using High Performance - Gel Permeation Chromatography (HP-GPC) and the "micro" level is investigated by Nuclear Magnetic Resonance (NMR) Spectroscopy. Recent results from investigations in these two areas will be given following a brief discussion of background.

NMR SPECTROSCOPY

Despite its broad use throughout the world for a variety of purposes, asphalt is not very well characterized with regard to its chemical constituents. There are three good reasons: (a) it is a very complex mixture of organic compounds, inorganic compounds, and complexes thereof; (b) for each type of organic functionality, there are several variations in the substituents which complicate the analysis; and, (c) the techniques for investigating such a complex mixture are inadequate. Among modern tools for organic chemistry, NMR spectroscopy represents a nondestructive probe which can provide some details regarding the structure and functionality of asphalts. This is particularly true if it is combined with chromatography and/or chemical modifications. (1-11)

For this project, eight asphalts were selected by an expert task group. These samples represent a broad spectrum of crude sources and are part of a nationwide project involving a large number of chemists and engineers collected under the umbrella called Strategic Highway Research Program (SHRP). In the work at Montana State University, $^1$H and $^{13}$C nuclei and a variety of NMR techniques (1D, 2D, DEPT) have been used to describe a few chemical characteristics of the asphalts.

Proton Results

A typical proton spectrum shown in Figure 1 reveals both simplicity in the types of groups present and the complexity of the same groups. The most prominent features include methyl (0.9 ppm), methylene (1.2 ppm) and aromatic (6.6-8.5 ppm) resonances. In
addition, some resonance intensity is found downfield of the methylene groups (2.0-5.0 ppm) derived from protons on the carbons attached to heteroatoms and aromatic systems. The spike at 5.3 ppm is due to d₂-methylene chloride which was used as the solvent. Lack of substantial olefinic residues is evidenced by a paucity of significant intensity in the 4.5 to 0.5 ppm range.

Figure 1. Example of a proton spectrum for an asphalt in methylene chloride (d₂).

The complexity of asphalt alluded to earlier is obvious from the lack of fine structure. This arises because the wide variation in the structure of asphalt components results in a multitude of chemical shifts for any given resonance. This feature must be recognized in any effort to collect data on asphalt.

As a result, our effort was to integrate the proton spectra of the eight asphalts and determine the relative number of protons present in the aliphatic, aromatic and "deshielded" regions. Data in Figure 2 show the percentage of aromatic protons in all eight asphalts. The error in this measurement is ± 0.25% on average, which is fairly typical of proton NMR spectroscopy. Nevertheless, these asphalts can be classified into three statistically unique groups: I: F; II: B>A=G>D=K; III: M>C.

Figure 2. Percent aromatic protons in the eight selected asphalts.
Integration of the spectra for deshielded protons (2-5 ppm), which include protons near heteroatoms and aromatic nuclei, yields the data shown in Table I. Unfortunately, there is not much difference among these asphalts. However, it does place an upper limit of 15-20% for these types of protons. Efforts to elaborate the details of this portion of the spectrum are needed.

TABLE I

Ratio of protons in the 2-5 ppm region relative to the total aliphatic protons

<table>
<thead>
<tr>
<th>Asphalt</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>AAA</td>
<td>0.19</td>
</tr>
<tr>
<td>AAB</td>
<td>0.20</td>
</tr>
<tr>
<td>AAC</td>
<td>0.16</td>
</tr>
<tr>
<td>AAD</td>
<td>0.21</td>
</tr>
<tr>
<td>AAF</td>
<td>0.19</td>
</tr>
<tr>
<td>AAG</td>
<td>0.19</td>
</tr>
<tr>
<td>AAK</td>
<td>0.20</td>
</tr>
<tr>
<td>AAM</td>
<td>0.15</td>
</tr>
</tbody>
</table>

---

High and low values

Carbon Results

A typical spectrum from $^{13}$C analysis is shown in Figure 3. As with the proton data, the carbon resonances may be categorized as aromatic (110-160 ppm) and aliphatic

(10-70 ppm). Further differentiation of the aliphatic portion might be: deshielded systems (35-70 ppm), methylenes (22-40 ppm) and methyl groups (10-22 ppm). A more detailed
assignment for each of the major resonances in the aliphatic region is shown in Figures 4 and 5. These assignments were based on information in the literature (1,5) and DEPT spectra (90° and 135°) (11). A typical DEPT 135 spectrum is shown in Figure 6.

Figure 4. Expanded portion of the aliphatic region of the 13C spectrum of a typical asphalt. See Figure 5 for suggested assignments.

<table>
<thead>
<tr>
<th>Resonance (ppm)</th>
<th>Peak number</th>
<th>Resonance (ppm)</th>
<th>Peak number</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.4</td>
<td>1</td>
<td>26.9</td>
<td>8</td>
</tr>
<tr>
<td>17.1</td>
<td>2</td>
<td>25.7</td>
<td>8</td>
</tr>
<tr>
<td>17.1</td>
<td>2</td>
<td>26.3</td>
<td>8</td>
</tr>
<tr>
<td>11.8</td>
<td>3</td>
<td>22.7</td>
<td>9</td>
</tr>
<tr>
<td>12.8</td>
<td>4</td>
<td>19.7</td>
<td>10</td>
</tr>
<tr>
<td>31.9</td>
<td>5</td>
<td>18.0</td>
<td>11</td>
</tr>
<tr>
<td>50.9</td>
<td>6</td>
<td>11.4</td>
<td>12</td>
</tr>
<tr>
<td>29.7</td>
<td>6</td>
<td>10 - 24</td>
<td></td>
</tr>
<tr>
<td>27.9</td>
<td>7</td>
<td>CH₄</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5. Assignments for the resonances shown in Figure 4.
From these data, the major resonances can be assigned to the respective carbons dealing with aliphatic chains and branches. A broad resonance near 37 ppm is assigned to both a CH₂ next to a branch in a chain and to the carbon in a benzylic position. Efforts to correlate benzylic carbon with benzylic protons are currently being made. The broad resonance 35-60 ppm is not readily assigned to any specific functionalities. However, there are some conclusions that can be made. The DEPT 135 experiment shows that the major contribution to this resonance is from CH groups which are deshielded by heteroatoms. A minor contribution may also arise from unique CH₃'s on heteroatoms. Thus, this resonance is a measure of heteroatom contribution and may reflect sites of chemical vulnerability.

Integration of the aromatic carbon peak area and total aliphatic carbon peak area allows for the calculation of percent aromatic carbon content (Figure 7). Differences are apparent within a range of 24 to 33 percent aromatic carbon and statistically distinct groups may be found, albeit one less than with the proton data. In this case, the groups...
are: I: F\(\geq\)K\(\geq\)B\(\geq\)G\(\geq\)A; II: M\(\geq\)C\(\geq\)D.

If one compares the \(^1\)H and \(^{13}\)C aromatic percentages by rank there are some changes which reflect a difference in substitution:

\(^1\)H: F\(\geq\)B\(\geq\)A\(=\)G\(\geq\)D\(\geq\)K\(\geq\)M\(=\)C,

\(^{13}\)C: F\(\geq\)K\(=\)B\(\geq\)G\(\geq\)A\(\geq\)M\(=\)C\(\geq\)D

It is clear that the correlation between percent aromatic carbon and aromatic hydrogen, shown in these rankings, is altered for asphalts K, A and D. Asphalt K is more substituted relative to the norm, while A and D are less substituted than this simple correlation would predict. This idea is further elaborated by the Aromatic Substitution Index (ASI) which is determined by dividing the number of aromatic carbon atoms in an average molecule by the number of aromatic protons. [The number of aromatic carbon atoms is calculated by multiplying the percent aromatic carbon (from NMR analysis) by the total number of carbon atoms in an average molecule (from elemental analysis and average molecular weight). The number of aromatic hydrogen atoms is derived similarly.] An ASI value of 2 means that 50% of the aromatic carbons are substituted with groups other than hydrogen. A value of 3 means that 66% of the carbons are substituted with groups other than hydrogen. Thus asphalt K is very highly substituted (68%) and asphalt D is the least substituted (55%). The other asphalts cluster near an index of 2.5 or 60% nonhydrogen substitution. These nonhydrogen substituents can be other aromatic or alicyclic rings, hydrocarbon chains and other carbon or heteroatom groups such as -CO-, OH and NR.

HIGH PERFORMANCE GEL PERMEATION CHROMATOGRAPHY

Asphalt research has shown that the properties of the binder strongly influence the performance of the asphalt-aggregate mixture. Binder properties result from the chemical constituents and, more important, from the interactions of these constituents to reduce their total energy by self-assembly. The aggregate surface chemistry also participates by influencing self-assembly in the adjacent molecular layers (the cybotactic region).

In this research, high performance gel permeation chromatography (HP-GPC) has been used to characterize asphalt cements and to probe their relative abilities to self-assemble (12-15).

HP-GPC, which is commonly used in the polymer industry, is particularly useful in the study of complex mixtures. The constituents are separated by apparent molecular size (hydrodynamic volume). Subsequently, depending on the detector used, other chemical information may also be obtained. Detectors have included those using refractive index differences, ultraviolet-visible (uv-vis) absorption spectroscopy, fluorescence, etc.

Here, uv-vis spectroscopy with a diode array detector has been used to provide information not only about the apparent molecular size distribution (MSD) of an asphalt but also about the uv-vis absorption of components across the MSD. The data may be presented as a set of chromatograms at seven different wavelengths (Figure 8) showing the

Figure 8. Example of 7-chromatogram HP-GPC plot of an asphalt.
relative absorption intensity across the MSD. Alternatively (Figure 9), a three-dimensional plot may be constructed connecting the chromatograms at wavelengths between 210 and 600 nm with uv-vis spectra of the components eluting at various times (apparent molecular sizes).

In this paper, use of HP-GPC with a diode array detector to differentiate among asphalts and to probe their potential for self-assembly will be discussed. The samples include asphalts and fractions from the Strategic Highway Research Program.

Experimental Procedures

All HP-GPC analyses were carried out with sample concentrations of 0.5 percent (weight/volume) in tetrahydrofuran on a 103A gel permeation chromatography column (Jordi GPC-GEL). Flow rate was 0.9 ml/min; system temperature was 24° C. The diode array detector (DAD) (Hewlett-Packard 1040A HPLC detection system) was set to acquire chromatograms at seven wavelengths: 230, 254, 280, 340, 380, 410 and 440 nm, all with 4 nm bandwidth. Sampling interval was 3.52 seconds.

Numerous samples have been analyzed but the key features may be illustrated by just a few examples. These include three whole asphalts and two of their fractions: strong acids and neutrals derived from Ion Exchange Chromatography (IEC). (Samples provided by Western Research Institute, Laramie, WY).

Differences among the samples may be observed visually by comparison of the HP-GPC chromatograms. These data may also be quantitated by integration of the areas under each of the chromatograms and between selected elution times (Figure 8). By summing the areas under all seven chromatograms (to obtain the conjugated volume, CV), the relative content of conjugated molecules and portions of molecules can be assessed.

Since, in general, more extensive conjugated systems (e.g., large aromatic rings) absorb at longer wavelengths than do smaller systems, the ratio of the absorption at 340 nm to that at 230 nm (the conjugation index, CI) provides a relative assessment of the size of the absorbing conjugated units, the chromophores. The values of similar parameters for individual time segments may also be calculated. The percentage of the total conjugated volume accounted for in these segments is also a useful parameter. These are not absolute measures of the quantity of asphalt of a given molecular size because of the variation of extinction coefficients among molecules and because some asphalt molecules and portions of molecules do not absorb in the uv-vis region of the spectrum. However, because conjugated systems are capable of entering into intermolecular interactions (pi-pi interactions) and the polar functional groups these systems often include can form polar bonds, this uv-vis analysis provides information about asphalt components which contribute significantly to asphalt performance.

HP-GPC Results

In this paper, three asphalts and two of their fractions will be used to illustrate the principles involved. Figure 9 contains three-dimensional plots of three asphalts, labeled D, F and K. Visual comparisons indicate that the molecular size distribution of asphalt F is much narrower than that of D or K and that there are few if any large entities in F (apparent molecular size decreases from left to right in the chromatogram). That is, there is just one distinct population of molecular sizes and those are relatively small.

Asphalt K contains a much broader range of molecular sizes with two distinct populations indicated. The second population in the left-hand shoulder probably represents both very large molecules and large assemblies of smaller molecules. An absorption about 410 nm across much of the MSD probably results from a series of vanadyl porphyrins and their derivatives (16). A trimodal peak shape (i.e. three distinct populations) characterizes asphalt D which contains more very large molecules/assemblies.
Figure 9. Three-dimensional plots of HP-GPC chromatograms for SHRP asphalts D, F, and K.
than does K. However, it is dominated by strong absorption at shorter wavelengths among smaller molecular sizes. There is also an absorption at 410 nm for vanadyl porphyrins but it is less pronounced than in asphalt K.

Values for some HP-GPC parameters for these asphalts are presented in Table II.

### Table II

<table>
<thead>
<tr>
<th>Asphalt</th>
<th>CV&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>CI&lt;sup&gt;(2)&lt;/sup&gt;</th>
<th>%CV&lt;sub&gt;11-17&lt;/sub&gt;</th>
<th>%CV&lt;sub&gt;17-21&lt;/sub&gt;</th>
<th>%CV&lt;sub&gt;21-25&lt;/sub&gt;</th>
<th>%CV&lt;sub&gt;25-34&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>20.6</td>
<td>0.245</td>
<td>24.0</td>
<td>30.5</td>
<td>34.1</td>
<td>11.4</td>
</tr>
<tr>
<td>F</td>
<td>27.6</td>
<td>0.299</td>
<td>8.3</td>
<td>39.0</td>
<td>44.2</td>
<td>8.5</td>
</tr>
<tr>
<td>K</td>
<td>24.6</td>
<td>0.274</td>
<td>19.5</td>
<td>35.3</td>
<td>33.8</td>
<td>11.4</td>
</tr>
</tbody>
</table>

<sup>(1)</sup> conjugated volume, total area in mAU x10<sup>5</sup>

<sup>(2)</sup> conjugation index, total, ratio of area at 340 nm to that at 230 nm

These data show that asphalt F is much more aromatic (conjugated) than K and that D has the least evidence for conjugation (from CV<sub>i</sub>). Furthermore, the average chromophore is larger in F than in K but smallest in asphalt D (from CI<sub>i</sub>). However, the percentage of absorption between 11 and 17 minutes in the large molecular size region (LMS) is least in asphalt F, greatest in asphalt D. That is, evidence for large molecules and/or large assemblies of smaller molecules is least in asphalt F, greatest in asphalt D.

These asphalts were separated by ion exchange chromatography into five fractions including strong acids and neutrals. It may be seen easily in Figure 10 that the neutrals fraction coincides with the major peak of the whole asphalt whereas the strong acids (Figure 11) contain much larger molecules and assemblies. Further, the neutrals fractions are not highly aromatic and their chromophores are small (Table III) whereas the strong acids fractions are quite highly aromatic with larger chromophores.

### Table III

<table>
<thead>
<tr>
<th>Asphalt</th>
<th>CV&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>CI&lt;sub&gt;i&lt;/sub&gt;</th>
<th>%CV&lt;sub&gt;11-17&lt;/sub&gt;</th>
<th>%CV&lt;sub&gt;17-21&lt;/sub&gt;</th>
<th>%CV&lt;sub&gt;21-25&lt;/sub&gt;</th>
<th>%CV&lt;sub&gt;25-34&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neutrals</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>11.8</td>
<td>0.083</td>
<td>0.4</td>
<td>19.4</td>
<td>56.5</td>
<td>23.7</td>
</tr>
<tr>
<td>F</td>
<td>16.9</td>
<td>0.194</td>
<td>0.2</td>
<td>27.8</td>
<td>60.3</td>
<td>11.7</td>
</tr>
<tr>
<td>K</td>
<td>14.9</td>
<td>0.121</td>
<td>0.6</td>
<td>27.5</td>
<td>55.0</td>
<td>16.9</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Strong acids</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
</tr>
<tr>
<td>F</td>
</tr>
<tr>
<td>K</td>
</tr>
</tbody>
</table>

<sup>(1)</sup> x10<sup>5</sup> mAU
The conjugated volumes of both neutrals and strong acids follow the same order as the whole asphalts; that is, F is most aromatic, D least so. A similar pattern is found among the conjugation indices of the neutrals fractions. However, such comparisons do not hold true among the conjugation indices of the strong acids. The size of the average chromophore in the whole asphalts decreases in the order F>K>D but in the strong acids the order is K>D>F.

There is also a decided difference in the chromatogram shapes of the strong acid fractions. Absorption in the large molecular size region is very strong for K and D indicating the domination of large molecules and assemblies in these samples. Although there is some indication of bimodality in the chromatograms of strong acids for F, there is certainly not the strong evidence for large molecules/aggregations seen in K and D.
CONCLUSIONS

Nuclear Magnetic Resonance Spectroscopy and High Performance Gel Permeation Chromatography have both been used to characterize asphalts from the Strategic Highway Research Program. These techniques provide different but complimentary information: NMR spectroscopy at the level of molecular structure and HP-GPC at the level of intermolecular interactions.
Both techniques show asphalt D to be unique among the SHRP asphalts. It is the least aromatic and the aromatic units are least substituted. Its combination of broad molecular size distribution, strong absorption in the LMS region indicative of intermolecular assemblies and unusual absorption among small molecular sizes is unmatched among the SHRP asphalts (although it resembles an asphalt found during another study (16)).

On the other hand, asphalt F is the most highly aromatic of the eight SHRP core asphalts by both NMR and HP-GPC analysis. Its aromatic units are about average in terms of substitution. Nevertheless, it is representative of a small group of asphalts with narrow molecular size distributions and little evidence for large molecules and assemblies.

Asphalt K contains a slightly lower percentage of aromatic carbon than asphalt F, but the units are highly substituted. From HP-GPC data, asphalt K is representative of a group of "ordinary" asphalts with rather broad molecular size distributions and bi- or trimodal chromatograms indicating the presence of large molecules and/or assemblies. No other SHRP asphalt displays so intense an absorption at 410 nm indicative of vanadyl porphyrins, however.

This and much more information may be obtained from NMR and HP-GPC analysis. Although correlations of such traits with asphalt performance characteristics have not yet been made, the authors anticipate that both techniques will contribute to understanding the chemical basis of asphalt behavior.

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LITERATURE CITED

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Asphalt Research in the Netherlands

The paper will give a state of the art and the latest results of the asphalt research performed within the so called TWAO-research group as sponsored by the Dutch Roads and Hydraulic Department of Rijkswaterstaat. The final goal of the research program is to develop an easy-to-handle pavement design method based on a visco-elastic material model concerning both fatigue and rutting.

This research program started in 1987 and is closely related to the SHRP-NL program that started in 1990.

The fatigue phenomena in asphalt concrete is described in terms of dissipated energy. One part of the research is devoted to calculating the dissipated energy in road structures during the passing of lorry axles.

Another part of the research covers the determination of fatigue properties, including healing, for a number of different mixes and the optimization of the mix design for fatigue properties.

The material model used is Burgers' model. Assuming that each individual loading pulse "damages" the elements in Burgers' model, it is shown that the fatigue behaviour is related to these elements. As from experimental studies appears that the Burgers' models, elements are directly related to the mix composition, the change in fatigue properties due to a charge in mix composition can be predicted.

The developed fatigue model in terms of dissipated energy will be validated by the LiNear TRACKing device "Lintrack" of the Delft University of Technology.

In the Netherlands rutting resistance of asphalt concrete is tested in, among other tests, the wheel tracking test.

The first part of the research was devoted to qualifying the effect of the boundary conditions of the test which have a significant influence on the test results. In the second part four different calculation models were used to simulate the wheel tracking test and to compare the results among each other.

One of the calculation models, a finite element program with a visco elastic material model is the most promising. This one is used to simulate rutting in a semi full scale test, the so called Hextrack, to hopefully validate the chosen approach.
Asphalt Research in the Netherlands.

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1. INTRODUCTION - CHAPTER 1

The ongoing sharp increase in both traffic volume and traffic intensity has put such a strong pressure on the capacity of the existing infrastructural system, that there is a strong need for maintenance free pavements and for widening the existing facilities. Furthermore, there is a strong need to conserve the environment by recycling reclaimed asphalt and by replacing primary by secondary raw materials, like slags and masonry waste. Both aspects have triggered an extensive research into the characteristics of asphaltic materials and the failure mechanisms of asphaltic pavements. In the Netherlands the politicians as well as the road authorities state that fundamental knowledge is needed to be able to maintain in the future the high standards of the road infra-structure: providing a safe, quiet, comfortable and effective means to transport persons and goods. The importance of the transport system for goods can be read from the fact that the transport industry accounts for about 7% of the national income.

In the Netherlands three public institutes are involved into research and contract standardization concerning road building: the Delft University of Technology (TUD), the Department for Road and Hydraulic Engineering of Rijkswaterstaat (DWW) and the Bureau of Research and Contract Standardization in Civil and Traffic Engineering (CROW). The activities of these institutes include the whole scope of road engineering: design, construction and maintenance of asphaltic, concrete and small element paved roads. The roads are of all categories: From the primary state motorways down to the local rural roads.

Given the factors mentioned above DWW decided to start a large research program called TWAO, which is basically concentrated on the fundamental characterization of the behaviour of the currently used asphaltic mixes. Based on this fundamental work practical guidelines for mix improvement will be developed. This research program is devoted to permanent deformation, fatigue and ductility
of asphalt mixes. Much of the actual work is done through contract research, performed by external research institutes.

For the same reasons the CROW decided to concentrate on more practical related research leading to a renewed asphalt mix design procedure. Also the durability aspects of drain asphalt is covered by the CROW.

The Delft University is involved in the asphalt research program through the projects on fundamental characterization of crack growth in asphalt mixes and the reinforced asphalt layers. Furthermore full scale testing by means of the Delft University LIN-TRACK device enables calibration and improvement of the mechanistic design and performance models. It is quite clear that such programs necessitate a close cooperation between the various parties. Besides these three public institutes also the industry does research. As is known, Royal Shell has important laboratories in Amsterdam. So the informal scheme of figure 1 can be drawn up.

The research activities can be divided in fundamental and practical research. The practical problems of today must be solved in any case. The approach is direct: dollar driven and problem solving. Fundamental research is needed to develop new and enhanced knowledge, in order to be able to solve the practical problems of the future in time. Finally the investigations must result in practical recommendations which find their places in the contracts between authorities and contractors.

The relations and synergetic effects between the most important Dutch research projects are shown in figure 2. If this figure is read from left to right one sees the contributions of one project to another: The TWAO-project concerning fatigue and healing will yield results that are useful for the CROW-project into the mix design, and the TUD-projects into crack growth and

Figure 1. Informal scheme of the organization of asphalt research in the Netherlands.

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reinforced asphalt layers. The mix-design project itself will not directly produce knowledge to be used in one of the other studies. This particular project can be called an ‘end-project’, see below. The project ‘durability drain asphalt’ has a practical objective, which does not interact with the other projects.

Figure 2. Interaction between the various asphalt research projects in the Netherlands. This figure is not limitative. See text.

All the investigations must add to the knowledge to design high quality mixes from all kinds of building materials. Here ‘high quality’ means that the mix should ‘fit the purpose’. In the end the investigations are related to the mix design procedure: Once the building materials are mixed, errors can only be repaired at large costs: the design procedure should prevent this. In figure 3 a scheme is shown presenting the basic organization of the mix design.
procedure which will be introduced in the Netherlands. The similarity with the AAMAS procedure [1] is striking. Starting point is that mixes have to be made out of locally available materials, whether these are raw or waste materials. The road designer has to know (or to assume) some characteristic values for the mixes he wants to apply in the road to design a road which can withstand the traffic loadings, climatic influences, etc. The mix design procedure has, among other things, to ensure that the characteristic values are attained for the applied mixes. For Dutch conditions in principle all the mixes have to withstand fatigue and permanent deformation, and must have a good workability and an acceptable dynamical stiffness. Wearing courses should moreover withstand stripping, cracking and ageing, whereas water tightness and skid resistance must be guaranteed. If all these requirements are fulfilled the mix may be produced. If not, other choices of building materials or of mix compositions are needed.

When the Strategic Highway Research Program (SHRP) was announced, it was decided in the Netherlands to start an intensive cooperation with the Long Term Pavement Performance program and SHRP-NL was formed. Given the extensive asphalt program that was already going on in the Netherlands, a SHRP-NL-Asphalt program was considered not to be necessary. All asphalt information needed in the SHRP-NL-LTPP program should be available from the on going research. It was however concluded that a good cooperation between the SHRP-USA and the Dutch asphalt research program is beneficial for both and so contacts are developed between the asphalt researchers of both countries.

From the overview given above it is clear that the TWAO research program is a basic block within the asphalt research program in the Netherlands, since it is dealing with the fundamentals and principles of asphalt mix characterization and behaviour. Three items, all coming from the TWAO-project, are discussed in this paper: the visco-elastic response model of asphalt pavements to dynamic loading of a wheel, the characterization of fatigue by means of the concept of dissipated energy and the measurement of rutting (permanent deformation) in a wheel tracking apparatus.

These three items are interrelated because they all have to do with the visco-elasticity of asphaltic concrete. In earlier studies [2,3] it has been shown that fatigue is governed very likely by the amount of dissipated energy. In order to have energy dissipated, one needs to leave pure elastic models and switch to visco-elastic ones. Naturally, one must provide for a method to calculate this energy in actual roads, if a load passes. Therefore the visco-elastic response model is in development. Concerning rutting it is noted that conventionally permanent deformation is calculated using so-called deformation laws. These laws relate
phenomenologically elastic strain and permanent deformation. It seemed worthwhile, from both a theoretical and a practical point of view, to explore a visco-elastic approach.

In this article an overview is presented over the ongoing research on these items and recommendations for further research are done. The main objective is to exchange views and to generate discussion.

2. A VISCO-ELASTIC MODEL OF A ROAD - CHAPTER 2

2.1 The states of stress in the road

The stresses in the road are important because rutting and fatigue are directly related to them. Unfortunately, the state of stress is very complex, as Verstraeten has shown [4]. In figure 4 Verstraeten shows the direction of the principal stresses (P₁, P₂ en P₃) in the wearing course of a three layer construction. For details of the construction the reader is referred to the original paper.

It appears that the momentarily principal stresses vary between compressive and tensile and in direction, depending on the stiffness and thickness of the construction, on the mechanical position of the wheel and on the depth. As the wheel moves over the surface, the stresses will go from one state into another. This implies a time-dependence.

Failures in the mix are not necessarily due to the highest instantaneous loads. They may be caused by a subsequence of states, stemming from the movement of the load. Due to rotation of the principal axis the mixes are loaded in various directions. Failure occurs if a mix cannot withstand a load. It is not necessary that this occurs in the direction of the largest load, nor in the direction in which the mix is weakest (anisotropy). It is the combination of the load and the strength that will initiate failure.

2.3 A visco-elastic model for the road.

In literature numerous indications [5,6,7] have been given showing the necessities and possibilities to calculate the visco-elastic response of an asphaltic road. This is done by introducing visco-elasticity firstly. Eventually the plasticity can be accounted for subsequently.
Based on linear elastic material model for the various layers, a number of computer programs are available to calculate stresses and displacements in a trafficked road. However, a literature search has not revealed any computer program which enables direct calculation of these parameters in a multi-layer visco-elastic system, subjected to a moving load. A few programs [8,9] comply with the visco-elasticity of asphalt mixes using phenomenological relations. They do not use formulas derived from first principles. Perloff [5] proposed a method which enables direct visco-elastic calculation of the displacements, strains and stresses in a halfspace, subjected to a moving pointload. It appears rather easy to extend this method to a moving circular load [10]. However extension to a multi-layer system seems not possible.

In order to study qualitatively the feasibility of the visco-elastic approach we used this method to calculate analytically the vertical stress $\sigma(t)$ and strain $\varepsilon(t)$ in a visco-elastic Kelvin halfspace, due to a moving pointload. The formulas can be found in [7]. If, for a volume-element, $\sigma(t)$ and $\varepsilon(t)$ are known the dissipated energy $W_{\text{dis}}$, can be calculated according to

$$W_{\text{dis}} = \int_0^\infty \sigma(t) \cdot \frac{\delta \varepsilon(t)}{\delta t} \, dt$$  \hspace{1cm} (1)

In figures 6a-6d the vertical stress and strain and the dissipated energy are given as a function of time, for a point with coordinates $(x, y, z)$. The load is at the origin $(0,0,0)$ at $t = 0$ s. The point moves in the x-direction. The calculated maximum values of both stress and strain are infinite if the load is right on top of the point under consideration. This is because the area of the load is zero. Therefore this maximum is fixed in the figure to an arbitrary (high) constant. The vertical stress is symmetrical with respect to the moment that the load is just above the point under consideration (indicated by the dashed vertical line). From the shape of the vertical strain the visco-elastic nature of the asphaltic concrete can be seen. The rather steep rise reflects the velocity of the load and the visco-elasticity of the material. The maximum strain is reached after the load is passed. The decay of the strain follows an exponential if the load moves fast compared to the visco-elastic retardation in the material.

From the presented figures the following conclusions can be drawn:

--the velocity of the load influences strongly the maximum strain.
--a part of the energy put into the road is elastic; this part is released if the load is passed. The dissipated energy is higher with lower velocity of the load. This notation is important with respect to fatigue.
--at higher temperatures the asphalt is more viscous. So at these temperatures more energy is dissipated. This also has large consequences for fatigue.

\[ E: \text{modulus of the spring in Kelvin-model (MPa)} \]
\[ \eta: \text{modulus of the dashpot in Kelvin-model (MPa.s)} \]
\[ v: \text{velocity of the load (m/s)} \]
\[ x,y,z: \text{coordinates of point under consideration.} \]

The drawbacks and shortcomings of the program used in this analysis will be overcome in the near future, when a computer code will be developed using Burgers' model for the asphalt mixes and a moving circular load. This code will be based on the work reported by Huang [6] and will allow a full visco-elastic treatment. Unfortunately, it has the drawback that it takes quite a lot computing time on a personal computer.

3. RESEARCH INTO RUTTING - CHAPTER 3

Despite numerous studies prediction of rutting is still a major problem. Therefore the need was felt to check
rut-prediction methods. At the same time it was considered worthwhile to check various types of mixes on their susceptibility for rutting. For that latter purpose a wheel rutting apparatus (see figure 7) is used for more than a decade now. In this apparatus a loaded wheel rolls over the sample which is fixed rigidly in a trough of steel, not allowing horizontal displacement. The sample is placed directly on the bottom of the trough. The dimensions of the beam are 350 mm x 110 mm and height equal to the layer thickness (normally 60 mm). The test is run at 40 °C. The sample is immersed in water and loaded by a rubber wheel (⌀= 150 mm) with a width of 50 mm and an elliptical contact area of 900 mm². The contact pressure is 0.1 MPa. The rutting value is the rut depth after 100,000 passes divided by the thickness of the layer. Another parameter is the slope of the straight line through the latter part of the measurements (figure 7c).

Figure 7. Set-up of the wheel track test in front view (a) and in cross section (b). Also the rutting is given as a function of the number of wheel passes (c).

Another method to assess the rutting susceptibility is the uni-axial static creep test. Here a cylindrical core is loaded during one hour by a vertical static load and the elastic and permanent strains are measured to calculate the (static) mix stiffness. This test is needed to calculate the rutting according to the method proposed by Shell [11], which uses the so-called Sfr-Smix relation.

3.1 Wheel tracking experiments

As it was a goal of this project to validate two algorithms to predict the actual rutting in the road (the methods proposed by Shell [11] and Esso [12]) experiments have been carried out in the laboratorium. In these experiments both the temperature and the loading are known very precisely, which should add to the accuracy of the prediction. It was felt that the prediction should be quite accurate under these well defined circumstances. To account for the assumption that the layers are infinitely in the horizontal directions, the following experiments were done. In the wheel tracking apparatus, the rutting of samples with various widths has been measured for two thicknesses of the samples. Above a given width the influence of the wall will be negligible. Then the experimental set-up can be considered to be infinite in
the horizontal plane and the two models to be verified can be applied. It was foreseen that extrapolation of the experimentally determined rutting to infinite width might be necessary.

For the experiments a gravel asphaltic concrete mix, standardly used as a base layer, was taken. This mix was applied in an industrial manner using spreading machines and rollers. The results of the experiments, carried out at 40 °C, are given in figure 8.

![Figure 8. Rut depths (after 100,000 passes) and the slope of the linear increase of the rutting versus the width of the sample, for two thicknesses of the sample.](image)

The lines have been drawn to guide the eye while considering that the rut depths has to level off at larger widths, that for small samples the width of the wheel will be limitative and that the rut depths for the thicker sample will be higher than (or equal to) the value for the thinner one. It occurs that:
- the walls influence the rutting if the sample is smaller than some 60 cm,
- there is an increase in rutting on increasing width.
- the rut depths seem to depend on the thickness, certainly at intermediate widths of the sample,
- the slope of the linear part of the rutting seems to converge to one value (for this mix $2.1 \times 10^{-5} \text{ mm/pass}$),
- for the thicker sample the slope is lower for the smaller samples.

3.2 Preliminary interpretations

An extensive interpretation of these data and of other measurements is still in progress. However, from the analysis carried out up to now it seems that a direct application of the methods proposed by Shell [11] and Esso [12] does not predict the absolute values measured in the wheel tracking test for samples with infinite width [13]. This concerns the onset of the rutting curves and the value after 100,000 passes. As the experimental
conditions in this test are well defined (much better than in practice), it is concluded that the practical usefulness of both methods is quite limited.

Some preliminary calculations have been carried out with the computer code FLAC [14], which is a two dimensional finite difference method. In these calculations the bottom of the sample is allowed to move freely in the horizontal plane, while it is fixed in the vertical directions. Movement of the sides was prohibited in the horizontal and free in the vertical plane. The stress distribution has been calculated and used to apply the permanent deformation law according to Francken [15]. It appeared that the calculated relative rutting value for the thicker samples is 1.3 times the value for thinner samples, whereas experimentally a factor of 1 to 1.1 was found (figure 8). This difference may be due to the modelling of the interactions between the sample and the trough.

Programs using layers with infinite widths (ECDR and ELSYM), calculate a rut depth for the 12 cm thick sample which is about 1.1 times the value for the 6 cm sample. In these calculations also the permanent deformation law [15] was used.

From figure 8 it appears that the slope of the rutting curve is more independent from the width for the thinner than for the thicker sample. This might indicate that this slope is a better measure for the rutting susceptibility of the mix than the rut depth, presumably because the latter depends strongly on the onset of the rutting.

3.3 Some static creep test results

In figure 9 static mix stiffnesses (determined after application of a static load for 1 hour) have been plotted versus the rut depth measured in the standard wheel tracking test on the same material and at the same temperature (40 °C). The mixes are conventional Dutch mixes, which have been applied between 1980 and 1990. It is evident that there is no direct correlation. From this figure it is concluded that in the wheel tracking test another property is measured than in the static creep test. This implies that it is sensible to measure both properties.

Figure 9. Static creep stiffness versus rutting values for gravel, open and dense asphaltic concrete.
Inspection of the presented data reveal that the mixes tend to cluster. One might suppose that in the wheel tracking test the quality of the aggregates and of the aggregate skeleton is measured, whereas in the static creep test the properties of the binder are tested. Moreover, the size of the aggregate may be reflected in the results: maximum size of the aggregate of the gravel asphalt concrete is 32 mm, of the dense and open asphalt concrete it is 16 mm. Anyway, one has to be very careful as investigations have been published where the dynamic creep test was used because the static creep test does not reveal the enhanced properties of the modified binder.

One might conclude that the rutting values obtained in the wheel tracking test will be small if the static creep stiffness is higher than 13. Unfortunately, such a conclusion would exclude the use of dense and open asphalt concrete. As those mixes act quite satisfactorily in practice this conclusion must be discarded. A better suggestion might be to suppose that the wheel tracking test does not simulate practice satisfactorily. Then one must look to this test again: The reader is referred to paragraph 3.1.

4. RESEARCH INTO FATIGUE - CHAPTER 4

Recent experiments [2,3] have given evidence that the dissipated energy per cycle determines the fatigue life of asphaltic mixtures better than the maximum tensile strain per cycle. As this finding strongly influences the algorithm to design roads on fatigue, more experiments have been carried out to validate this finding even more stringently. In this paragraph fatigue experiments will be described. Then they will be interpreted: firstly in the conventional way, secondly using the concept of dissipated energy.

4.1 Fatigue experiments

The fatigue experiments have been carried out in a four point bending system. The applied signal was either a continuous sinus (cs-test) with period $T_0$, or a periodical non-sinusoidal signal (pns-test) with period $T_1$ (see figure 10). The experiments were done at 20 °C and 0 °C, at 29.3Hz and 56.7 Hz, under controlled strain conditions. In all the experiments $T_1=3.7T_0$. It is remarked that the pns-loading becomes a cs-test
if $\alpha \rightarrow 1$. The parameter $\alpha$ is a measure for the deterioration of the sinussoidal signal.

4.2 Conventional interpretation.

In figure 11 some of the results are presented. The measured fatigue life ($N_t$), determined in the pns-test, has been plotted versus the factor $\alpha$.

Principally, the pns-tests cannot be interpreted conventionally, as there is no sinussoidal loading. Nevertheless, in practice such an interpretation is done by counting the number of times the strain reaches a maximum. For $\alpha=1$, as the load in sinussoidal, the interpretation according to Wöhler can be done directly; naturally, the life determined in the cs-test must be divided by 3 to become comparable with the one determined in the pns-test. One might assume that the fatigue life is three times as high for $\alpha=0$ as for $\alpha=1$ because the number of peaks with equal height per period is 2 for $\alpha=0$ and 6 for $\alpha=1$. Now it is quite reasonable to connect the two points by a straight line to assess the dependence of the fatigue lifes on $\alpha$.

Conventionally, the noted differences between the measured and predicted fatigue lifes may be interpreted as healing ($H'$):

$$H' = \frac{N_{\text{dis}c}}{N_{\text{con}}}$$

where $N_{\text{dis}c}$ is the fatigue life for a discontinuous signal and $N_{\text{con}}$ for a continuous sinussoidal loading. Following this reasoning $H'$ can be obtained for various values of $\alpha$ from figure 11 (see figure 12). From figure 12 it appears that:

**Figure 11. Measured fatigue lifes versus $\alpha$. The straight lines are drawn according to a Wöhler-interpretation.**

**Figure 12. Values for $H'$ versus $\alpha$.**
--H' increases with decreasing α, which is in line with the findings in literature [14].
--there is no increase of the parameter H' with increasing temperature. This certainly raises question marks with respect to the interpretation as healing.
--H' is about 20 or more for α=0, even at 0 °C. This is unacceptable high for an interpretation as healing [16 and paragraph 4.3]. Therefore, this interpretation must be rejected. And by consequence the interpretation based on the Wöhler-curve must be discarded.

4.3 Conventional healing experiments

Three and four point bending experiments with controlled strain [17] have been performed on the same mixes to establish the healing values to be used in road design. In these experiments, done at 15 °C, a full period (T) of a single sinus was followed by a restperiod of 9.T. The results were striking as in these experiments no healing (H') was found. The results were the more puzzling as identical measurements reported by Bonnaure [18] did show healing. Moreover, controlled stress (two, three and four point bending) tests on the same mixes had a healing of 5-7. These latter values are quite reasonable with respect to literature [16]. Our results cannot be explained using conventional theories and certainly do not allow values for H' of 20 at low temperatures and for short restperiods, as mentioned in paragraph 4.2.

4.4 Interpretation based on dissipated energy

The fatigue life Nc for a periodical non-sinussoidal can be calculated [3] by:

$$N_c = \frac{T_0}{T_1} \cdot N_0 \cdot \left(\frac{T_1 \cdot \Delta W_o}{T_0 \cdot \Delta W_1}\right)^{b/2}$$

where $T_0$ is the period of one sinus in the cs-test
$\Delta W_o$ is the energy which is dissipated initially in one period of the same cs-test
$N_0$ is the fatigue life determined in the same cs-test at the appropriate strain level [3].
$T_1$ is the period of the loading with the pns-test.
$\Delta W_1$ is the energy which is dissipated initially in one period of the pns-test.
$b$ is the power in the Wöhler-curve: $N = a \cdot \varepsilon^{-b}$

Relation (2) enables calculation of the fatigue life for all types of loads: the assumption that the loads must be sinussoidal in time, as needed in the Wöhler-approximation, can be set aside and the actual time dependences of the stresses and strains occuring in the road can be used. In figure 13 the energy $\Delta W_1$ has been plotted versus $\alpha$ (0.2≤$\alpha$≤0.8) and extrapolated to $\alpha$=1. Thus the parameter is obtained which can be entered in
It is noted that the extrapolated values coincide very nicely with the dissipated energy \(3\Delta W_0\) calculated from the cs-tests (at \(\alpha=1\)). Due to experimental difficulties the cs-test at 20 °C and 29.3 Hz is not available. One may compare \(N_t\) from relation (2) with the fatigue life \(N_t\) determined in the pns-test (see figure 14). Here \(N_t\) is the number of periods the material can stand before a sudden decrease in the energy dissipated per cycle \(\Delta W(n)\) occurs [3]. This number has earlier been identified as the point where fatigue micro-cracks start to develop.

The prediction is quite good, although the calculated fatigue lives seem to be systematically low by a factor 2-4. To the authors' opinion this cannot be due to healing, as there should be some influence of temperature at least.

From equation (2) it is obvious that it may be due to either an overestimation of \(\Delta W_1\) or an underestimation of \(\Delta W_0\). Also, it should be recalled that (2) is derived assuming two important conditions:

1. The Wöhler-parameter \(b\) should be independent from the frequency in the range given by the Fourier-components of the pns-signal. In our case this range is from 10 to 60 Hz. As shown in [3], for practical application and for \(\alpha>0.5\) the most important frequency is \(1/(3T_1)=1/T_0\). As mentioned, the cs-tests were carried out at that frequency.

2. The parameters \(W_{cs}\) and \(W_{pns}\) should be equal, as otherwise their quotient appears between the brackets in relation (2). These parameters have been introduced by van Dijk and are given by:

\[
\begin{align*}
L_0 & = 0.6, \\
L_2 & = 0.4, \\
L_6 & = 0.4, \\
L_8 & = 0.4,
\end{align*}
\]
\[ \psi = \frac{\Delta W_1 \cdot N_1}{W_{tot}} \]  
where \( N_1 \) is the fatigue life  
\( \Delta W_1 \) is the initially dissipated energy in one cycle  
\( W_{tot} \) is the accumulated energy, dissipated during the whole experiment.  
If the ratio \( \psi_{ps} / \psi_{cs} \approx 1.5 \) the predicted fatigue lifes will coincide with the measured ones. This is under further investigation.

5 RECOMMENDATIONS AND FURTHER RESEARCH - CHAPTER 5

Although at first glance asphalt mixes are used mainly in only one application, they can fail in a large number of different ways. To understand and to prevent these failures scientist and civil engineers have to cooperate. Not only within one country, but preferably over a larger part of the world.  
The developed knowledge should preferably result in a mix design procedure, as in the stage of the mix design costly mistakes can be prevented. However, it is unrealistic to assume that the whole range of possible failure mechanisms can be controlled by only one or two mechanical tests. The more so as these test must be suitable for use in the daily practice of mix design. Therefore it is urged that the designer of the road construction, who uses in his design characteristics of the mixes, describes explicitly the mechanical loads (including special climatic conditions) the mix has to withstand. If possible he should also describe the mechanical properties he desires. Then the designer of the mix should assure that the wanted mechanical properties are attained, without loss of the general requirements like workability and resistance to ageing. So, for the daily practice a close cooperation between the road and the mix designer has to be pursued.

5.1 Visco-elastic responsmodel

In the near future a computer program will be developed to calculate the respons (stresses, displacements, strains and dissipated energies) in a three layered, linear visco-elastic system, subjected to a moving circular load. The material will be described by the Burgers' model.

5.2 Rutting

Rutting is not yet understood fundamentally, although it is described phenomenologically. Experimentally well defined measurements do not confirm the calculations based on models used in literature. Wheel tracking tests for rutting must be done in a set up which accounts for the limited size of the sample. The slope of the rutting curve might be a good parameter to express the
rutting susceptibility.

If the linear visco-elastic multi-layer program will become available, the rutting experiments will be analyzed using this program. To characterize the asphaltic mixes the four elements in Burgers’ model are needed. They will be determined using dynamic creep tests (with or without confining stresses) on samples taken from the mix on which also the rutting experiments have been carried out. These elements might depend on the number of applied loads. If it appears impossible to obtain a reasonable fit between theory and experiments the introduction of plasticity in the Burgers’ model will be considered.

5.3 Fatigue and healing

The assumption that the dissipated energy is a fundamental parameter to describe the damage caused by repeated loading appears very benificial. The hughes differences between measurements and expectations based on the Wöhler-curves are replaced by a systematic error of only a factor of 3. The energy concept seems to enable a characterization of healing independent from the method of measurement. Further research is initiated to prove more broadly the use of the relation between energy and fatigue. Also measurements have been started concerning:
--the amount of energy that can be absorbed by the material in dependence of temperature,
--determination of the values of the elements in Burgers’ model for application in the visco-elastic model for the road, both as a function of temperature and repeated loading,
--dynamic bending tests in controlled stress mode at several temperatures and frequencies.

5.4 General

The three main items described here must be assembled in one system in order to understand the mechanical behaviour of the roads more deeply and to weigh the various damage processes in the road. As the mechanical properties of the road and of the mixes are described by Burgers’ model, a direct possibility to investigate the influences of the mix composition on the damages may become available.

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Closed Track Testing of
Maintenance Work Zone Safety Devices

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Abstract

CLOSED TRACK TESTING OF INNOVATIVE MAINTENANCE WORK ZONE SAFETY DEVICES (SHRP Project H-109)

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Twelve innovative work-zone traffic control devices were evaluated in closed track field testing. "Standard" devices, consisting of those currently applied in operational practice, were also included in the testing to provide an experimental control baseline. Subjects drove an instrumented vehicle through simulated highway work zones in an industrial park which included a non-active airport runway. Applied study measures consisted of driver cognitive responses and vehicular behaviors.

Tested innovative traffic control devices consisted of flagging procedures (mechanical gate, hand-held paddle with flashing lights), maintenance vehicle-mounted flashing lights (providing an illusion vehicle deceleration), and barricades (incorporating arrow designs) indicating lane closures and two-way traffic flow.

Ninety-six driving subjects were utilized, equally divided between daytime and nighttime. The age distribution was determined to approximate the general driving public (i.e., an even male/female split, with age groupings as follows; 25 percent 16-24 years, 50 percent 25-54 years, and 25 percent 55-plus years.)

Designated traffic control device M.O.E.s (measures of effectiveness) were derived from desired device effects and consisted of appropriate vehicle performance and in-depth driver interview responses. Vehicle instrumentation gathered the following performance data: speed profiles approaching test devices, longitudinal and lateral acceleration, approach lane change point, and brake pedal activations. An interior-mounted video camera was also used to record the driver's field of view and structured commentary. Obtained driver intrusive measures were: (1) device detection, e.g., time required to discern that a target device would affect driving, following its appearance in the driver's view; (2) device interpretation, e.g., time required to assess appropriate vehicle action; (3) driver subjective ratings of device helpfulness and safety value; and (4) higher order perception, e.g. driver estimate of closure speed when following a slow maintenance vehicle.

Closed track testing was completed in December. Study results, contained in the submitted paper, will compare the effects of innovative devices with baseline devices. Those devices showing positive results will be subsequently evaluated in open highway testing.

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CLOSED TRACK TESTING OF
MAINTENANCE WORK ZONE SAFETY DEVICES

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1. INTRODUCTION

Innovative work-zone traffic control devices were evaluated in closed track field testing. Tested innovative procedures consisted of flagging devices (flagger gate, flashing STOP/SLOW paddle), travelled-way speed bump, diverging lights, direction indicator barricades, and opposing lane dividers. These innovative procedures were developed in a previous research project (1).

"Baseline" devices, consisting of standard devices currently applied in operational practice, were also included in the testing to provide an experimental control baseline. Subjects drove an instrumented vehicle through simulated highway work zones in an industrial park which included a non-active airport runway. Applied study measures consisted of driver cognitive responses and vehicular behaviors. Closed track findings produced recommendations for the full highway test.

This paper addresses the study procedure, driver characteristics affecting driver speed, applied measures of effectiveness, results, and field study recommendations.

2. STUDY PROCEDURE

The utilized test site was comprised of an inactive airport runway, taxiway, and approach roadway network. The closed-field test track layout enabled routing test subjects in a manner to achieve the following experimental objectives:

1. Subjects drove through a series of simulated work zones in roadway settings arranged to ensure processing of only one device at a time.

2. Subjects were exposed to "baseline" conditions (i.e., standard devices currently in use) as a reference in order to evaluate effects of the tested devices.

3. The order of devices was rearranged during testing so as to achieve "counterbalancing", i.e., to reduce response contamination due to learning, fatigue, expectancy.

Test subjects were recruited and randomly selected from a mass mailing, conducted in cooperation with a civic organization in Georgetown, Delaware, USA. A total of 96 subjects were tested, i.e., 48 each for daytime and night testing. A division of subject by gender (50% male, 50% female) and age (25% 16-24 years old, 50% 25-54 years old, 25% 55-plus years) was sought in order to approximately replicate the general driving public.

Subjects drove a medium-sized American sedan (with automatic transmission) equipped with Datron Systems, Inc. Correvit instrumentation consisting of a speed recorder, lateral/longitudinal...
accelerometer, and brake pedal activation recorder. Vehicle performance data were recorded in ASCII format at .5 second or 20-foot intervals. The vehicle was also equipped with a videotape camera recording the driver's field of view and structured commentary.

Prior to testing, subjects were given a brief introduction advising that they were to drive in a normal fashion. During a preliminary 3-mile open road drive, undertaken to familiarize subjects the instrumented vehicle, "baseline" data (e.g., speed, lateral deceleration variance) was unobtrusively measured to determine subjects' normal driving behaviors. A risk-taking profile was also obtained for each subject. An experimenter rode in the passenger seat, giving route directions and administering a structured questionnaire.

3. DRIVER CHARACTERISTIC EFFECTS ON BASELINE SPEED

Observed drivers' open highway speeds, as affected by driver characteristics in absence of any work zone devices is first discussed in order to illustrate the sensitivity of the applied study procedure. A baseline set of data was included in a questionnaire 1x3 measure driver characteristics known to affect speed (e.g., age, sex, driving risk-taking profile).

The introductory drive included a 3-mile, two-lane roadway highway section devoid of any work-zone activity. Subjects had no idea during this portion of the drive that the study involved work zone activity. Speeds were unobtrusively measured with the in-vehicle instrumentation.

Analyses of these baseline speeds with age, sex, and risk-taking attitude data revealed that baseline speeds were predictable on the basis of these observed variables. While seven attitudinal questions were included in the survey, responses to only one question appeared in the multiple regression equation which demonstrated the best ability to predict baseline speed. Surprisingly, baseline speeds were not shown to be affected by gender.

The single attitudinal question shown to be the best predictor of speed was the following.

"I like to pass cars when driving at relatively high speeds on two lane roads.

1. Strongly Agree
2. Agree
3. Undecided
4. Disagree
5. Strongly Disagree"

Multiple regression analysis yielded the following equation to predict drivers' baseline speed.

\[
\text{Speed} = 55.128 - (0.099)(\text{Age}) - (1.266)(\text{Response to question}).
\]

Very high statistical reliability (p<.001) was associated with the above finding. It is evident from inspection of this function that lower speeds were exhibited by older drivers and drivers who do not like to pass on two-lane roads.

The above example demonstrates the ability of the applied variable set to quantitatively determine which factors affected driver responses to tested experimental work zone devices. While speeds in response to work-zone traffic control devices were also noted to be affected by diver characteristics, their impact was not as strong. Therefore, it was found that tested work-zone devices were influential in overriding driver characteristic effects on speed.

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4. APPLIED MEASURES OF EFFECTIVENESS (M.O.E.s)

Two measures categories were as follows:

1. **Driver-intrusive Measures** - Driver response data obtained via the in-vehicle questionnaire, videotaped driver commentary, and post-drive interview.


The following driver-intrusive measures were utilized, on the basis of in-vehicle questionnaire responses, driver commentary as recorded on in-vehicle videotape, and the structured post-drive interview.

- **Device Recognition Time** - The time after which a test device comes into the subject's field of view until the subject recognized it as a device which may affect his/her driving.

- **Device Interpretation Time** - The amount of time required after, the device came into view, for the subject to interpret what action response was intended by the device.

- **Interpretation Correctness** - Whether or not the subject correctly reported the appropriate action message intended by the device.

- **Interpretation Problems** - Whether or not the subject reported problems in his interpretation of the device.

- **Helpfulness Rating** - Categorical response to the following question:
  
  "How helpful was this device to your driving?"

  Very helpful; Helpful; Not very helpful; Not at all helpful

- **Safety Rating** - Categorical response to the following question:

  "In your opinion, use of this device would make highways... Much safer; Somewhat safer; A little safer; No safer at all."

The following vehicle behavioral measures were determined from vehicle instrumentation installed in the car driven by test subjects.

- **Approach Speed** - Speed of the instrumented vehicle as the tested device first came into the driver's view.

- **Arrival Speed** - Speed of the instrumented vehicle as the vehicle arrived at the device location.

- **Approach Speed Profile** - Difference between the above two speeds.

- **Speed Variance** - The mathematical variance function based upon a set of speed measurements taken between the approach and arrival speed points. Speeds were recorded at intervals according to the type of device being evaluated as follows: (1) Delineation/divider devices -20 feet; (2) Lighting/-flagging device - .5 second

- **Lateral Acceleration** - The lateral g-force recorded by an in-vehicle accelerometer. The measurement interval was the same as for speed variance data above. The maximum value obtained
over the series of measurements was applied in the data analysis.

**Longitudinal Acceleration** — Based on measured speed at 20 feet or .5 second intervals, given the recording scheme noted above for speed variance. The maximum value obtained over the series of measurements was applied in the data analysis.

**Lateral and Longitudinal Acceleration Variance** — The mathematical variance function based on the acceleration measures noted above. These variance functions have been cited in the literature as characterizing driver confusion or discomfort in the context of evaluated traffic situations.

**Brake Light Advance Activation Time** — As determined from the vehicle's instrumented brake pedal, the first activation on the approach to a tested device.

**Brake Light Activation Frequency** — The number of times that the brake pedal was activated on the approach to a tested device.

**Advance Lane Change Distance** — The distance in advance of a lane closure that the driver maneuvered into the proper lane.

5. **RESULTS**

Findings address specific device categories, i.e., lighting, flagging, delineation, and lane divider devices.

5.1 **Lighting Devices**

Two lighting configurations were applied which warn of maintenance vehicles slowing in the traffic stream. Lighting devices were mounted on trucks being followed by the vehicle driven by the test subject. These devices were standard four-way flashers (the baseline condition) and the diverging lights (See Figure 1). Findings based on driver-intrusive measures are discussed first; these are followed by a discussion of vehicle behavioral measures results.

![Figure 1. Diverging Lights](image)
Driver responses. Improved device recognition was observed with use of the diverging lights; however, longer information processing times were noted. That is, this device more quickly realized that the diverging lights would affect their driving, yet more time was required to determine the appropriate action response than with the standard four-way flashers. The message most often conveyed by the diverging lights was to advise of caution and/or to slow down.

Drivers were more likely to report problems with the diverging lights. Typical problems reported with the diverging lights were that the lights were too dim, the flash rate was too slow, the lights were confusing, or that no directional message was provided. This last reported problem gives rise to a potential concern that some directionality message (e.g., some association with the arrow board) may be expected from the diverging lights and, therefore, the lack of a directionality cue may be a problem. However, a review of driver interpretations of the device did not confirm that the lack of a directional message was a problem. That is, no driver-reported interpretations were found to indicate the need for a directional message from the diverging lights.

Test subjects were asked to rate each device as being helpful or improving safety; no overall differences in ratings were observed between the four-way flashers and diverging lights. However, diverging lights were better accepted by drivers who were less predisposed to high speed driving.

Two "higher order perception" driver response measures were applied to determine whether drivers more accurately estimated the speed and closure rate with respect to the truck they were following. First, subject drivers were asked to estimate the speed of the truck they were following. In each case the truck was being driven at 5 mph. Average estimated speeds with use of the standard four-way flashers and diverging lights were 8.0 and 8.9 mph, respectively. Although, no statistical difference exists between these estimates for the total sample, older drivers were more likely to estimate faster closure speeds in presence of the diverging lights. Second, drivers were asked to subjectively estimate the closing rate between their vehicle and the truck they were following. Drivers' subjective estimates of closure between their vehicles and the leading truck identified as closing (1) very fast, (2) fast, (3) slowly, and (4) not at all. The average response for each lighting device was between "fast" and "slowly"; no statistical difference was found between device responses.

The extent to which lighting device responses were affected by driver characteristics (e.g., age, gender, risk-taking profile) was examined in a correlation analysis. Driver detection of both four-way flashers and the diverging lights was affected by age. Older drivers detected the lighting devices earlier than did younger drivers. Drivers approaching both the four-way flashers and the diverging lights at higher speeds were more likely to give higher estimates of travel speeds for the respective maintenance vehicles.

Two observed differences between the standard four-way flashers and the diverging lights are as follows. The four-way flashers were recognized further in advance during nighttime conditions. No day-night advance detection difference was found to be associated with the diverging lights. Higher estimates of the maintenance vehicle's speed were noted for driver's approaching at a higher rate of speed when four-way flashers were applied; however, this trend was not evident when the diverging lights were in use.

The above analysis indicated that the diverging were more effective in warning of a slow-moving maintenance vehicle when perceived by older and more cautious drivers.
Vehicle behaviors. No differences were found between the four-way flashers and the diverging lights in terms of either approach/arrival speeds or first brake light application time with respect to the leading maintenance truck.

A number of differences were found in terms of deceleration smoothness as the subject’s vehicle approached the leading maintenance truck. A strong tendency was observed for lower deceleration rates following the diverging lights, but the difference was not statistically significant. However, significantly reduced values of both speed and deceleration variance were observed in the presence of the diverging lights. Similarly, drivers approaching the four-way flashers at higher speeds demonstrated higher rates of deceleration and speed variance; yet this finding was not observed in the presence of the diverging lights. These results imply less indecision or uncertainty on the part of motorists as they approached a maintenance vehicle displaying the diverging lights. A confirmation that less driver uncertainty was associated with the diverging lights was the fact that, as previously reported, shorter driver recognition times were associated with the diverging lights.

5.2 Flagging Devices

The baseline device in this case was the standard STOP/SLOW paddle; tested devices were: (1) the travelled-way speed bump, a portable rumble bump placed in advance of the flagging operation, (2) the flashing STOP/SLOW paddle (See Figure 2), and (3) the flagger gate (See Figure 3). Driver responses are discussed first, and these are followed by vehicle responses.

![Figure 2. Flashing STOP/SLOW Paddle](image)

Driver responses. No recognition time differences were noted between devices. However, degraded driver information processing and device interpretation times (i.e., longer times required) were noted for both the flashing STOP/SLOW paddle and flagger gate. The travelled-way speed bump also produced an increase in information processing time, by comparison with the standard flagger. These results indicate that drivers consumed more time in their determination of the required action in response to both the flashing STOP/SLOW paddle and the flagger gate. Long information processing time associated with the flashing STOP/SLOW paddle can most likely be attributed to its novelty effect. Moreover, addition of the travelled-way speed bump did not significantly affect overall recognition time associated with the standard flagger paddle;
However, improved recognition times were shown for daytime as opposed to night conditions.

The research interpretation of longer information processing time must take into account the novelty effect of the tested innovative devices. That test subjects immediately recognized standard devices but required additional time to interpret the intended action message of the innovative devices is not surprising. While statistical differences between devices were observed, it must be noted that the magnitude of the response differences is not great. For example, average interpretation times between the best and worst flagger devices (standard flagger and flashing STOP/SLOW paddle) were 11.8 and 10.0 seconds. The additional 1.8 second task load imposed by experimental flagging devices is not viewed as consequential due to the fact that driver attention to the overall driving task was not diverted during this time.

Fewer subject drivers correctly interpreted the flagger gate and flashing STOP/SLOW paddle. Numerous test subjects confused the flagger gate with the railroad crossing protection gate. This driver interpretation was no doubt due to red cross-hatching on the gate arm. The color red was required by the U.S. Federal Highway Administration due to their interpretation of uniform traffic control device specifications. However, results point out that the color orange would be more effective in conveying the work zone activity message.

The predominate flagger gate interpretation may have contaminated responses to the flashing STOP/SLOW paddle. That is, an unexpectedly large proportion of subjects also associated the flashing STOP/SLOW paddle with a railroad crossing. However, as noted in one response, the alternating red flashing light was the cause for confusion of the device with a railroad crossing signal. Consequently, the device will be redesigned so as to avoid confusion with a rail-crossing signal.

Driver subjective ratings of device helpfulness and the extent to which test devices can make highways safer demonstrated that no improvement was associated with either the traveled-way speed bump or the flashing STOP/SLOW paddle. However, the flagger gate was rated as being more helpful and more conducive to increased highway
safety than the tested standard flagger device. In general, faster drivers rated the flagger devices as more helpful. This tendency was noted for both the standard flagger and experimental devices.

An observed difference between the standard flagger and experimental devices was that recognition time for the standard flagger was degraded under nighttime conditions. However, this effect was not found for either the flagger gate or STOP/SLOW paddle, indicating their superiority under nighttime conditions.

One noted benefit associated with the flagger gate was noted in that drivers who recognized the device earlier exhibited fewer brake light applications on their approach to the device. This result is interpreted to indicate less driver uncertainty regarding the required response to the device.

Vehicle behaviors. Approach speeds (e.g., point at which devices came into view) did not vary between tested flagging devices. Therefore, measured arrival speeds at device locations validly distinguished between devices. Arrival speeds at the tested flagging devices did not statistically differ from that observed with the standard flagger. However, average arrival speeds approaching the flashing STOP/SLOW paddle were statistically slower those approaching the flagger gate (in the arm down position). Speed profiles and travel times between the approach and arrival positions did not vary between tested devices.

Advance brake peddle activation time was utilized as a valid measure of driver preparatory response to flagging devices. This measure did not discriminate between the tested flagging devices. However, the longest advance brake activations were observed with the travelled-way speed bump. Obviously, drivers were braking for the device rather than the flagman or the paddle. A relatively weak correlative effect (i.e., .05 confidence level) was shown to associate advance braking time in response to the travelled-way speed bump with certain risk-taking questionnaire responses. More cautious drivers were more likely to apply their brakes further in advance of the speed bump; yet, these were the slower drivers in advance of the bump. This noted risk-taking trait did not, however, override the braking tendency elicited by the speed bump.

Vehicle deceleration differences were observed between flagging devices. Smallest deceleration values (e.g., indicative of the smoothest slowing) were associated with the flashing STOP/SLOW paddle. The maximum spot deceleration measurement, averaged across subject drivers, on the approach to this device was lower than that recorded for either the standard STOP/SLOW paddle or the travelled-way speed bump. Moreover, an examination of correlations with driver characteristic variables demonstrated no external influences on driver deceleration behavior in response to the flashing STOP/SLOW paddle.

Observed speed variances on the approach to flagging devices did not differ between the tested alternatives. However, due to the accentuated slowing response to the speed bump, higher deceleration variance was observed for this device.

In summary, the smoothest slowing was found to be associated with the flashing STOP/SLOW paddle, and the earliest braking response was caused by the travelled-way speed bump. The validity of lower speed variance as a benefit associated with the flashing STOP/SLOW paddle was confirmed by an examination of driver questionnaire responses. Drivers exhibiting lower speed variance were more likely to rate the flashing STOP/SLOW paddle as being more helpful. An added benefit of the STOP/SLOW paddle was seen for nighttime conditions in that speed variance was lower than that observed during daytime conditions.
5.3 Delineation Devices

Standard barricades were applied as the baseline condition in lane closure device arrays for both right- and left-lane closures. Test conditions consisted of two Direction Indicator Barricade (DIB) designs (See Figure 4) for each, left and right lane closure application. Tested DIB designs were the "pop" (direction arrow was raised from the top of the device exposing two striped panels) and the "flip" (direction arrow is hinged at top of device; exposing a single striped panel). Thus, the applied experimental scheme enabled comparisons of both the "flip" and "pop" designs in each, the right and the left lane closure application.

!["POP" DESIGN](image1)  
!["FLIP" DESIGN](image2)

Figure 4. Direction Indicator Barricades

Driver responses. Longer driver recognition and interpretation times were found for the direction indicator barricades by comparison with the standard barricades. In particular, this condition was noted for all DIB treatments, with the exception of interpretation time when the "pop" device applied in the left lane, in which case no differences was found. Longer information processing time was noted for the "flip" device applied in the right lane; otherwise no differences were found.

Three explanations likely underlie the longer response times noted above for the DIB devices. First is the fact that the DIBs are more visually complex than the baseline standard barricade. Because of their larger size and dual design (i.e., arrow plus stripe panel) there is simply more information to be processed. Second, the arrow
provides a additional message than merely a striped panel. Therefore, interpretation of the arrow likely involves deriving an additional message, i.e., distinctly separate from the panel. Finally, the novelty of the tested devices likely caused drivers to fixate on the devices for an extended duration prior to making their action decisions.

Confirmation of the above explanation of the longer recognition time associated with DIB devices was found by associating recognition time with driver subjective ratings of device helpfulness. In the presence of the standard right lane closure, drivers recognizing the devices earlier in their approach (i.e., longer recognition times) rated the standard devices as less helpful. Also, male drivers were less likely to rate the standard lane closure as helpful. However, the DIB devices consistently elicited high helpfulness ratings regardless of recognition time or gender. Thus, the higher helpfulness ratings of the DIB devices, independent of other factors, demonstrates a clear benefit associated with DIB devices.

Driver recognition of the delineation devices was also affected by a variety of other factors. In general, delineation devices were recognized earlier by older drivers and those less prone to high risk-taking attitudes (e.g., expressing enjoyment of high-speed driving). No recognition time differences were observed between male and female drivers. Also, certain behaviors were found to be associated with recognition responses to the delineation devices. Drivers who recognized the devices earlier were more likely to exhibit speed variances or sudden lane change behaviors (e.g., higher lateral accelerations), possibly indicating some element of confusion in their response.

Drivers approaching the DIB devices at higher speeds more often recognized and interpreted the devices later, however the lane change maneuver distance in advance of the devices did not differ from slower drivers who recognized the devices earlier. This result attests to the fact that adequate information assimilation and processing to the complex device design was achieved.

The two designs of DIB devices were compared to each other within the context of both right- and left-lane closures. The "pop" device demonstrated an improvement over the "flip" device on the basis of two measures comparisons. It produced better interpretation times in the left lane comparison and improved information processing times in the right lane comparison.

Benefits of the DIB devices were seen in that they were more likely to be correctly interpreted by subject drivers. These arrow barricade designs more frequently drew explicit responses (e.g., "right lane closed", or "merge left") rather than general responses (e.g., "lane closed"). Additionally, the DIB devices were more likely to receive higher subjective ratings, indicating that drivers viewed the devices as being more helpful and possessing greater potential to make highways safer, by comparison to the baseline standard barricades.

In order to distinguish between the two direction indicator barricade designs, subjects were shown pictures after the drive-through and asked to indicate a preference between the "pop" and "flip" designs. The pop was the predominately favored device, by a wide margin of 74 to 26 percent. Subjects' stated reasons for their selection are listed in Table 9. Typical reasons for selecting the "pop" device were: (1) greater target value is afforded due to the larger size, (2) more reflective area would provide improved nighttime delineation, and (3) the arrow is more visible due to the increased height. Typical reasons for selecting the "flip" device were: (1) the simple design makes the device easier to understand, (2) the arrow is at driver eye level, and (3) higher white-to-orange ratio is desirable.
An attempt was made to confirm subject preference ratings favoring the "pop" device by examining driving response differences between "pop" and "flip" devices within right and left lane closure approaches. For the left lane closure situation, improved driver interpretation times were seen for the "pop" device. In advance of the right lane closure, less information processing time was required (at the .05 statistical confidence level) in presence of the "pop" device. In all other comparisons, no statistical differences existed to indicate superior performance of either the pop or flip designs.

**Vehicle behaviors.** Lane change distances (i.e., the advance distance at which test subjects made their preparatory lane change maneuvers) demonstrated no differences between standard and DIB designs when the devices were deployed for left lane closures. However, in the case of right lane closures, test subjects changed lanes in closer proximity to the devices. This behavior difference can not be explained in terms of the device design. An examination of driver response variables (e.g., recognition times) provided no explanation. Furthermore, an examination of day-versus-nighttime responses revealed no differences. While lane change distances were statistically shorter for right lane closures, the differences did not appear to be of sufficient magnitude to warrant safety concerns. This right- versus left-lane change issue will be further examined during highway testing.

No differences were observed between tested delineation devices in terms of brake pedal activations, approach/arrival speeds, speed profiles, or speed variance.

The summary result is that DIB treatments produced an improvement over standard lane closure treatments in terms of driver-reported improvements in lane closure guidance information. Test subjects rated DIB devices as vastly superior in terms of helpfulness and safety potential. While the devices did require longer driver information processing times (likely due to their novelty effect), this response did not produce a significant degradation in vehicle performance.

5.4 **Lane Dividers**

Two device treatments were tested which advise of two-way traffic for application in work zone situations in which both lanes are normally used for one direction of traffic flow. Standard 36-inch high road tubes were applied as the standard, or "baseline", treatment. Opposing lane dividers (See Figure 5) were applied as the comparable experimental device.

**Driver responses.** The opposing lane dividers generally elicited favorable driver response results. Improved recognition and interpretation times were associated with their use. That is, drivers more quickly recognized that the opposing lane dividers would affect their driving and made earlier decisions regarding the appropriate action response. Once the device was recognized, the time required to determine the appropriate action did not significantly differ between the standard tubes and the opposing lane dividers.

Driver recognition of the standard tubes differed between day and night conditions. Drivers recognized the tubes further in advance during conditions of darkness due to the reflectivity of the devices. No significant day-night difference was observed for the opposing lane dividers due to their superior daytime conspicuity.

The opposing lane dividers were correctly interpreted more often than were the standard road tubes. A significant proportion of test subjects did not associate any message (e.g., "no idea regarding
device intent") with the standard tubes. Many test subjects associated general messages (e.g., "work zone") with tubes. However, correct interpretations of the tubes (e.g., "stay in lane") were not as explicit as interpretations of the opposing lane dividers (e.g., "two-way traffic"). This result substantiates the utility of the two directional arrow face design of the device.

Not surprising in view of the above, the opposing lane dividers received higher subjective ratings in terms of being more helpful and potentially making highways safer. The standard tubes were more likely to be rated as helpful and improving safety by drivers who detected the devices further in advance. However, the opposing lane dividers consistently received higher ratings regardless of driver's advance detection distance.

Vehicle responses. Higher approach speeds were observed for standard tubes than for opposing lane dividers. (Approach speeds were those recorded 400 feet in advance of lane divider devices). However, vehicle speeds in the vicinity of the devices did not vary between the standard tubes and opposing lane dividers. Geometric and pavement conditions associated with the test site may have accounted for some of the observed speed profile and speed acceleration variance difference. The speed profile issue will be further pursued in the highway field study.
Correlations between driver response and vehicle behavior variables tended to elucidate effects of the devices, thus explaining possible causes of speed variation. Drivers who recognized and interpreted the devices earlier were more likely to approach the opposing lane dividers at higher speeds. However, drivers who were late in their device recognition more often failed to correctly interpret the device. Also, these drivers reduced their speeds more suddenly, although their speeds passing the device were not necessarily slower than drivers exhibiting earlier device recognition.

Varied reactions to the opposing lane dividers were associated with certain driver characteristics. Older drivers approached the devices more slowly, rated the device higher in terms of safety potential, and then proceeded to pass the device more slowly than younger drivers. Therefore, it is seen that speed variation in response to lane dividers is attributable to both device recognition and driver age.

6. SUMMARY

The test track evaluation has produced results based on driver and vehicle responses to tested devices. Findings have provided insights regarding effectiveness (or ineffectiveness) of tested devices and do provide direction for further study in open highway testing.

While the diverging lights did experience some light brightness shortcomings, this problem is expected to be corrected during a design modification. The diverging lights produced improved driver recognition times and smoother decelerations on the part of following vehicles, by comparison with the standard four-way flashers.

The flashing STOP/SLOW paddle, flagger gate, and the travelled-way speed bump will be tested further. The flashing STOP/SLOW paddle produced the smoothest vehicle deceleration responses by comparison with other tested devices and was more effective than the flagger gates in slowing approaching motorists. The flagger gate was rated more helpful than the standard flagger device and produced a reduction in brake activations (indicating greater driver certainty regarding action response). The travelled-way speed bump was associated with more advanced braking behaviors than any of the other tested flagging devices and produced improved nighttime recognition of the standard flagger paddle.

The Direction Indicator Barricade (DIB) is recommended for field testing. The "pop" design, incorporating two orange/white panels, is considered superior to the "flip" design due to its increased attention-gaining utility. This effect was confirmed in test track results. In addition, the DIB conveyed more specific guidance information to test subjects and commanded vastly higher safety and helpfulness ratings by comparison to standard barricades.

The opposing lane dividers are recommended for highway evaluation. These devices elicited more favorable driver subjective ratings than any other tested device. Use of the opposing lane dividers produced improved driver recognition and detection (by comparison with standard tubes), were more often correctly interpreted, and conveyed a more explicit message.

7. REFERENCE

Innovative Materials for Pavement Surface Repairs

Shashikant Shah
Sr Staff Engineer
Strategic Highway Research Program
USA
ABSTRACT

INNOVATIVE MATERIALS FOR PAVEMENT SURFACE REPAIRS

By

Shashikant Shah
Sr. Staff Engineer, SHRP

Pavement maintenance activities comprise a large proportion of the work effort of the highway agencies. As the system ages and deteriorates, this proportion will increase. Highway agencies will need to devote more time and resources to such maintenance activities in the next decade. Improved maintenance materials and methods can reduce resource requirements by increasing productivity and minimizing frequency of repairs and roadway occupancy requirements.

SHRP's two projects, H-105 and H-106, deals with identification of promising materials for repairs of potholes, spalls, joints and crack, and field installation and short-term evaluation of these materials, respectively. The paper discusses the findings of these two studies.

The first part of the paper discusses the results of the questionnaire, the performance of these materials as reported in the questionnaire, and the materials selected for further testing in the field.

In the second part, the paper further discusses the experimental design and research plans for field installation of the materials. The plan includes factors of environment, traffic, pavement condition, method of repair, etc., under which the materials are to be evaluated. The laboratory testing plan, to investigate the association of field performance with these laboratory test results, is also discussed. Limited short-term field performance data are presented.
1. INTRODUCTION - CHAPTER 1

Increase in traffic loads and user demands, coupled with the aging of the highway system, has inevitably increased pavement maintenance requirements. Furthermore, these increased requirements have outdistanced the available resources necessary to accomplish the demands to provide a higher level of service. Two of the most common maintenance activities are pothole repairs and crack and joint sealing. Both of these activities require personnel and equipment to occupy zones of temporarily closed roadways. This results in traffic delays and exposes both maintenance personnel and motorists to increased safety risks, particularly in urban areas. These delays and risks would be reduced if operations could be made more rapid and/or less labor-intensive, and if the repair themselves could be made more durable. This might be done using materials that set up or cure rapidly, that can be applied under a broad range of temperatures, that have good bonding and friction characteristics and can be applied quickly.

One of the major goals of the Strategic Highway Research Program (SHRP) is to further the state of knowledge in the pavement maintenance area, specifically, in the area of surface repairs of potholes, spalls, cracks and joints. To accomplish this goal, SHRP initiated two projects, H-105 and H-106. The former, completed in April, 1990 (1), dealt with identification of promising materials for above repairs, and the latter which was initiated in October, 1990, deals with field installations and evaluation of the materials identified as promising in H-105. This paper briefly discusses the relevant findings of these two studies.

1.1. Study Objectives

The overall objectives of the two projects can be stated thus:

- To identify promising materials, procedures, and equipment for patching potholes and sealing/filling cracks in asphalt concrete, and repairing spalls and sealing/filling cracks and joints in portland cement concrete
that are more effective and efficient in preventing pavement deterioration than existing methods.

- To develop a set of experimental plans for each of the above repairs that would guide the performance of a field experiment.

- To conduct laboratory and field testing of the above identified promising materials for short and long-term field evaluation.

- To prepare manuals and training courses on the use and application of the materials/procedures for adoption by State Highway Agencies (SHA).

The first two objectives have been accomplished through Project H-105. The last two objectives are to be accomplished through the current on-going project H-106.

2. **RESEARCH APPROACH - CHAPTER 2**

Data collection was accomplished through the following sources:

- Literature search
- Questionnaire response from SHAs, cities, counties and Canadian Highway Agencies
- International highway agencies
- Material producers
- Trade journals and associations
- Personal visits to field sites based on questionnaire response

2.1. **Types of Information Collected**

Information on materials included various test specification data, performance data, and other pertinent data of interest. Specifically, the following was collected for patching and sealing materials:

- performance data in terms of traffic and years
- Adhesiveness
- allowable temperature ranges for application
- durability
- stiffness
- allowable moisture conditions
- hole or joint/crack preparation requirements
- special handling/mixing requirements
- cure time for PCC
- environmental effects
2.2. **Data Base**

All information collected for the project was systematically stored in the computer using relational data base manager ORACLE. The contents are based entirely on the information received in the returned questionnaire. A comprehensive user's manual was prepared to describe the data use and manipulation(2).

2.3. **Presentation of Data**

Only summary results from the questionnaire are presented in the following chapter for each of the four repair materials. Comprehensive results are presented and discussed in reference 1. It should also be pointed out that the results that will be presented in the next chapter are broad interpretations averaged across different climates, design procedures and installation procedures. However, the presentation will provide an insight into the relative performance of the various repair materials discussed in this paper.

3. **TEST RESULTS - CHAPTER 3**

3.1. **Portland Cement Concrete (PCC) Joint Resealing**

The purpose of joint resealing is to reduce the amount of water entering the pavement structure through the joints and also prevent intrusion of incompressibles into the joint. The historical performance of joint resealing of pavements has not been very satisfactory. Several studies have indicated that only a small portion of existing concrete pavements are effectively sealed. This points to the need to improve the performance and effectiveness of joint resealing.

3.1.1 **Types of Joint Sealants**

According to the American Concrete Institute (ACI), most of the sealants fall into three general categories.

- Thermoplastic materials
  - cold-applied
  - hot-applied
- Thermosetting materials
  - chemically-curing
  - solvent release
- Preformed compression seals

The thermoplastic materials are bitumen based materials grouped into hot-applied and cold-applied materials. The thermosetting materials are one- or two-component materials. Silicones, polysulphides and polyurethanes...
are the sealant materials in this category. Preformed compression seals are premolded strips of styrene, urethane, neoprene or other such synthetic materials.

3.1.2 Joint Sealant Performance

Whereas it is difficult to pinpoint the factors that define performance characteristics of joint sealants, there are, however, certain factors on which the performance can be evaluated. These include:

- adhesion
- cohesion
- resiliency
- weathering
- resistance to infiltration of incompressibles

One must be aware, however, that the failure in any of these modes may or may not be directly related to the properties of the sealant. Pavement joint design, installation procedures, environmental conditions, condition of the joint, etc., all have some effect on their performance.

Table 3-1 is a generalized information extracted from the questionnaires. As mentioned before, the table represent broad interpretations across different climates, pavement designs and installation procedures. However the review does provide some insight into the relative performance of the sealants in use today.

3.1.3 Summary of Findings

From the questionnaire response and from review of available literature, several trends regarding the performance of concrete joint sealants have become apparent. Based on these trends, the following materials are considered worthy of field installation for further evaluation of their performance.

- Silicone
- Polymerized asphalt rubber
- Low-modulus polymerized asphalt rubber

The next chapter discusses the experimental design, research and evaluation plans for each of the repair materials.

3.2 Crack Sealing/Filling in Asphalt Pavements

One of the primary forms of distress in asphalt pavements is cracking. Cracks, if ignored, will contribute to accelerated deterioration of the pavement which ultimately leads to development of potholes. Crack sealing/filling, if done properly and with high
quality materials, can be a cost-effective operation to extend the life of the pavement. This section discusses the selection of crack sealants and fillers to be evaluated in the SHRP H-106 experiment.

3.2.1 Types of Crack Sealants and Fillers

The crack sealants and fillers used in asphalt pavements are generally those used in PCC joints, except that some of the thermosetting solvent release materials are not used to seal these cracks. Furthermore, these sealants must also be compatible with asphalt concrete:

- Thermoplastic materials
  - hot-applied
  - cold-applied
- Thermosetting chemically-cured materials

3.2.2 Crack Sealant/Filler Performance

Factors on which performance of crack sealers/fillers can be evaluated are: adhesion, cohesion, resistance to abrasion/wear, resistance to weathering, flexibility and resiliency, etc., basically the same as those for joint sealing.

Table 3-2 summarizes the experience of the respondents with various sealant materials. The table shows the average experience with the material. As can be seen, virgin asphalt material is the most widely used material for this surface repair. Notice the range of life expectancy reported by the respondents, any where from one month to a maximum of 12 years for asphalt rubber material. Generally, the performance of sealants placed in dry conditions was better than the performance placed in wet conditions. Furthermore, the average reported life expectancy of sealants placed in temperatures above 40F was usually greater than the average life of the sealants placed in temperatures below 40F.

3.2.3 Summary of Findings

Based on the overall response on the performance of the crack sealing/filling materials, the following materials are identified as worthy of field installation for further evaluation of their performance:

For crack sealants
- Polymerized asphalt rubber
- Low-modulus polymerized asphalt rubber
- Fiberized asphalt
- Asphalt compatible silicone

For Crack Fillers
- Asphalt rubber
- Fiberized asphalt
- Asphalt cement
- Proprietary emulsion
### Table 3-1: General Summary of Questionnaire Responses for PCC Joint Sealing

<table>
<thead>
<tr>
<th>Sealant Type</th>
<th>Total No. of Responses</th>
<th>Experience</th>
<th>Range of Life Expectancy</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Average</td>
<td>Range</td>
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<tr>
<td>Asphalt Cement</td>
<td>8</td>
<td>23.4</td>
<td>2 - 50</td>
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<td>Asphalt Emulsion</td>
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<td>10 - 35</td>
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<td>Modified Emulsion</td>
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<td>6.7</td>
<td>5 - 10</td>
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<tr>
<td>Asphalt Cutback</td>
<td>9</td>
<td>25.3</td>
<td>7 - 50</td>
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<tr>
<td>Asphalt Rubber</td>
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<td>11.5</td>
<td>3 - 40</td>
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<td>32</td>
<td>7.8</td>
<td>0.5 - 15</td>
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<td>Fiberized Asphalt</td>
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<td>7.1</td>
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<td>30</td>
<td>5.8</td>
<td>1 - 12</td>
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<td>PVC Coal Tar</td>
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<td>10.0</td>
<td>10</td>
</tr>
<tr>
<td>Polysulfide</td>
<td>1</td>
<td>15.0</td>
<td>10</td>
</tr>
<tr>
<td>Compression Seal</td>
<td>17</td>
<td>16.3</td>
<td>2 - 25</td>
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</table>

### Table 3-2: General Summary of Questionnaire Responses for AC Crack Sealing

<table>
<thead>
<tr>
<th>Sealant Type</th>
<th>Total No. of Responses</th>
<th>Experience</th>
<th>Range of Life Expectancy</th>
</tr>
</thead>
<tbody>
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<td></td>
<td></td>
<td>Average</td>
<td>Range</td>
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<tr>
<td>Asphalt Cement</td>
<td>21</td>
<td>16.6</td>
<td>1 - 25</td>
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<tr>
<td>Asphalt Emulsion</td>
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<td>14.8</td>
<td>1 - 35</td>
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<tr>
<td>Modified Emulsion</td>
<td>4</td>
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<td>2 - 5</td>
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<td>Asphalt Cutback</td>
<td>24</td>
<td>19.4</td>
<td>1 - 12</td>
</tr>
<tr>
<td>Asphalt Rubber</td>
<td>40</td>
<td>6.9</td>
<td>1 - 15</td>
</tr>
<tr>
<td>Polymerized Asphalt Rubber</td>
<td>29</td>
<td>6.9</td>
<td>1 - 15</td>
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<td>Fiberized Asphalt</td>
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<td>4.4</td>
<td>0 - 12</td>
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<tr>
<td>Silicone</td>
<td>2</td>
<td>6.5</td>
<td>1 - 12</td>
</tr>
<tr>
<td>PVC/Coal Tar</td>
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<td>15</td>
<td>15</td>
</tr>
</tbody>
</table>

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The placement and evaluation plan of the above materials is discussed in the next chapter.

3.3. Portland Cement Concrete (PCC) Spall Repair

Concrete pavement deterioration at the joints and cracks is generally characterized as spalls. This type of deterioration can be attributed to construction, design, materials or combination of these. Such repair is generally accomplished by either full-depth or partial-depth repair procedures. In this study only partial depth repair materials are investigated.

3.3.1 Types of Spall Repair Materials

Although cementitious materials are predominantly used for spall repairs, there are several other categories that include materials that have been used for patching deteriorated concrete. Following are these categories:

- Inorganic materials (cementitious)
- Organic Materials (polymeric)
- Conventional bituminous mixes
  - hot mix
  - cold mix
- Modified/proprietary bituminous materials

Most of the cementitious materials such as the portland cement-based products, gypsum-based products, magnesium phosphates and high alumina cements fall in the inorganic material category. Epoxies and urethane are in the organic material category.

3.3.2 Performance of Spall Repair Materials

Short cure time, excellent bonding capabilities, ease of placement, strength and durability, resistance to adverse condition of placement, etc., are some of the characteristics desired in spall repair materials.

Table 3-3 provides overall summary of the PCC spall repair questionnaire response. As in previous tables, this table also gives a general indication of highway agency usage and experience with the various generic material types and estimated performance ranges. At the lower end of performance scale, only one material shows life of one year or more.

Bituminous cold mix shows the largest use as temporary fix by 51 percent of the respondents, followed by portland cement concrete and bituminous hot mix. High alumina cement, polyester styrene and methyl methacrylate had less than five percent response and as such reliable performance history could not be determined.
3.3.3 **Summary of Findings**

From the questionnaire response, it was evident that very few materials could be identified as having promising performance capabilities. These few were selected for further field evaluation, as were also those for which scarce information was available. The final list includes:

- Portland cement concrete
- Gypsum-based concrete
- Epoxy
- Urethane concrete
- Magnesium phosphate concrete
- High alumina cement
- Hydraulic cement concrete
- Modified/proprietary bituminous cold mix

3.4. **Asphalt concrete Pothole Repair**

No pavement surface deterioration receives as much public scrutiny and generates aggravation as the pothole. Potholes not only cause vehicle damage but also present safety hazard to the user and the workers involved in the pothole patching operation. Because of the hazards placed on the patching crew, majority of the patching operation employ "throw and go" or "dump and run" technique. In most cases, the patch life using such techniques is less than a few hours or days or, in rare instances, weeks.

The greatest need and potential benefit for improved repair of potholes is for rapid repair under adverse environment of wet and cold. Likewise, materials that can provide longer life under the current "throw and go" operation would be the most desireable from cost effective point of view.

3.4.1 **Types of Pothole Repair Materials**

The materials for pothole repair fall in the following categories:

- Conventional bituminous mixes
  - hot mix
  - cold mix
- Modified/proprietary bituminous materials
- Proprietary concrete mixes

3.4.2 **Performance of Pothole Repair Materials**

Based on questionnaire responses, table 3-4 was prepared. The table summarizes the usage and experience with the various generic material types and the estimated performance ranges. Permanent hot mix and temporary cold mixes make up majority of the usage for patching. Less
<table>
<thead>
<tr>
<th>Repair Material Type</th>
<th>Total No. of Responses</th>
<th>Experience (Years of Use)</th>
<th>Range of Life Expectancy (Years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland Cement Concrete</td>
<td>47</td>
<td>13.2</td>
<td>1 - 50</td>
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<tr>
<td>Magnesium Phosphate Concrete</td>
<td>12</td>
<td>8.3</td>
<td>3 - 15</td>
</tr>
<tr>
<td>Epoxy</td>
<td>21</td>
<td>14.9</td>
<td>1 - 35</td>
</tr>
<tr>
<td>Polymer</td>
<td>6</td>
<td>6.0</td>
<td>2 - 10</td>
</tr>
<tr>
<td>Bituminous Hot Mix (Temp)</td>
<td>36</td>
<td>18.2</td>
<td>1 - 40</td>
</tr>
<tr>
<td>Bituminous Hot Mix (Perm)</td>
<td>37</td>
<td>20.6</td>
<td>3 - 60</td>
</tr>
<tr>
<td>Bituminous Cold Mix (Temp)</td>
<td>51</td>
<td>20.8</td>
<td>1 - 60</td>
</tr>
<tr>
<td>Bituminous Cold Mix (Perm)</td>
<td>23</td>
<td>15.3</td>
<td>1 - 50</td>
</tr>
<tr>
<td>Fiberized Asphalt Mix</td>
<td>7</td>
<td>6.2</td>
<td>5 - 12</td>
</tr>
<tr>
<td>Proprietary Cold Mix</td>
<td>27</td>
<td>5.6</td>
<td>0.4 - 10</td>
</tr>
</tbody>
</table>

Table 3-3: General Summary of Questionnaire Responses for PCC Spall Repair

<table>
<thead>
<tr>
<th>Repair Material Type</th>
<th>Total Number of Responses</th>
<th>Experience (Years of Use)</th>
<th>Range of Life Expectancy (Years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bituminous Hot Mix (temporary)</td>
<td>40</td>
<td>21</td>
<td>2.5 - 45</td>
</tr>
<tr>
<td>Bituminous Hot Mix (permanent)</td>
<td>70</td>
<td>20.3</td>
<td>3 - 45</td>
</tr>
<tr>
<td>Bituminous Cold Mix (temporary)</td>
<td>65</td>
<td>19.4</td>
<td>1 - 50</td>
</tr>
<tr>
<td>Bituminous Cold Mix (permanent)</td>
<td>45</td>
<td>13.4</td>
<td>1 - 45</td>
</tr>
<tr>
<td>Sylvax UPM (proprietary)</td>
<td>27</td>
<td>6.4</td>
<td>1 - 12</td>
</tr>
<tr>
<td>Perma-Patch (Proprietary)</td>
<td>5</td>
<td>3.3</td>
<td>1 - 5</td>
</tr>
<tr>
<td>Fiberized Mix</td>
<td>11</td>
<td>6.9</td>
<td>4 - 12</td>
</tr>
<tr>
<td>Other Proprietary Mixes</td>
<td>20</td>
<td>7.7</td>
<td>1 - 15</td>
</tr>
<tr>
<td>Proprietary PCC</td>
<td>3</td>
<td>6.3</td>
<td>3 - 8</td>
</tr>
</tbody>
</table>

Table 3-4: General Summary of Questionnaire Responses for AC Pothole Repair
than five responses on some proprietary materials have been grouped together under Other Proprietary Bituminous Mixes. One proprietary material had 27 responses indicating its widespread use. In general, the response showed that the average proprietary materials greatly out performed conventional cold mixes placed under adverse conditions (cold temperatures and wet holes). This, however, is not noticeable when placed under warm conditions (>32F). Some of these findings are graphically presented in Figures 3-1 through 3-4.

3.4.3 Summary of Findings

Based on the evaluation of questionnaire response and published literature, the following materials were recommended for further evaluation under contract H-106 which is discussed in the next chapter.

- Conventional cold mixes
- Four proprietary materials
- Fiberized cold mix
- Mixes with modified emulsions
- Penn Dot 485 (special graded mix)
- Injection spray material (to be placed with automated equipment)

4. FIELD TESTING AND EVALUATION - CHAPTER 4

This chapter discusses the experimental plan for field evaluation of the materials identified as better performing materials in project H-105 (discussed above). The plan provides for a nationwide testing of the most promising materials and procedures for all four repair operations. It also describes the field layout to fulfill the last two objectives defined in chapter 1.

4.1. Experimental Design

The experimental factors selected represent those variables that are believed to have a strong effect on the performance of the repair materials. The following factors are considered for field evaluation:

- Materials
- Climate
- Procedures
- Traffic

Each of the factors is discussed briefly below

Materials: The materials selected for field installation and evaluation are those discussed in chapter 3. Details can be found in reference 1 and 3.
COLD VS WARM TEMPERATURES
Wet Hole

PATCH LIFE, MONTHS

<table>
<thead>
<tr>
<th>Material</th>
<th>Cold</th>
<th>Warm</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC HOT MIX</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>COLD MIX</td>
<td>8</td>
<td>15</td>
</tr>
<tr>
<td>PROPRIETARY</td>
<td>15</td>
<td>22</td>
</tr>
</tbody>
</table>

Figure 3-2: Life Expectancy of Patches Placed in Cold and Warm Temperatures

PROPRIETARY MIXTURES
Temporary: Wet Hole - Cold Temperature

PATCH LIFE, MONTHS

<table>
<thead>
<tr>
<th>Material</th>
<th>0</th>
<th>10</th>
<th>20</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>QPR 2000 (1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WESPRO (1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>COLD MIX W/FIBERS(4)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ALAPATCH (1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WOLFE (1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UPM (2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>INSTA PATCH (1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PERMA PATCH (2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CONV AC HOT MIX</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CONV COLD MIX</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3-4: Life Expectancy of Various Proprietary Materials

PERMANENT VS TEMPORARY PROCEDURES
Permanent(dry hole) Temporary(wet hole)

PATCH LIFE, MONTHS

<table>
<thead>
<tr>
<th>Material</th>
<th>Temporary</th>
<th>Permanent</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC HOT MIX</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td>COLD MIX</td>
<td>15</td>
<td>10</td>
</tr>
</tbody>
</table>

Figure 3-1: Life Expectancy of Permanent vs Temporary Patches

WET VS DRY HOLE
Cold Temperatures

PATCH LIFE, MONTHS

<table>
<thead>
<tr>
<th>Material</th>
<th>Wet Hole</th>
<th>Dry Hole</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC HOT MIX</td>
<td>20</td>
<td>15</td>
</tr>
<tr>
<td>COLD MIX</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>PROPRIETARY</td>
<td>22</td>
<td>17</td>
</tr>
</tbody>
</table>

Figure 3-3: Life Expectancy of Patches Placed in Wet and Dry Potholes
Climate: Sites are located in each of four major SHRP environment zones to represent temperature and moisture conditions as follows: wet-freeze, dry-freeze, wet-nofreeze and dry-nofreeze. Figure 4-1 is a map of the four zones. The map also shows distribution of the sites in each zone for each repair.

Procedures: For each repair, several procedures are included to determine the material/procedure system effect on the performance. Following is a brief description of the procedures considered in the installation of each repair material. Details can be found in reference 3.

Pothole: two repair procedures: rapid emergency and semi-permanent.


Cracks: five configurations: route and flush, route and band aid with 0.75"x0.75" reservoir, route and band aid with 1.5"x0.2" reservoir, simple band aid with no routing, and rout/saw and recess.

Joints: three configurations: recessed with backer rod, overband with extensive joint preparation, and overband without extensive preparation.

Traffic: medium to high traffic.

4.2. Site Selection and Installation

Sites were solicited from states (USA) and Canadian provinces. Of the 114 sites offered, 21 sites meeting the experimental design and other selection criteria were selected for the experiment. Figure 4-1 shows the distribution of the final selected sites.

Installation of repair materials was performed by the state maintenance personnel with assistance from the material suppliers and the contractor responsible for project H-106. By June, 1991, the installations were completed. All data collected during installation were recorded on appropriate forms specifically developed for each repair(4).
Figure 4-1: SHRP Climate Zones and H-106 Site Locations
4.3. Evaluation

4.3.1 Field

A total of five evaluations are planned for each repair site. The first three to be done at three month interval and the remaining at six month interval. Post repair evaluation data is recorded on forms specifically developed for each repair (4). As of this writing, two evaluations have been completed at some sites. This will continue until the end of the project in December, 1992. If the repairs are still performing at the end of this period, they will be monitored by post SHRP agency.

4.3.2 Laboratory

One of the objectives of this study is to identify laboratory tests whose results might be good indicator of field performance of these same materials. The existence of such performance-related specifications would greatly enhance maintenance departments' abilities to identify which new or untried materials showed the greatest promise and, therefore, warrant field testing.

To realize the above objective, laboratory tests are conducted on all materials included in the repair experiment. The tests are standard ASTM and, in some cases, specialized tests (3).

4.4. Performance Results

As mentioned above, post repair evaluation has been limited as of this writing. Some of the pothole patch materials at three sites have failed within one month after installation. All the failures have been with the local materials. At these sites two of the proprietary materials have also failed at this early stage after installation. These are preliminary findings only and the causes of the failures have not been determined at this point in the evaluation.

5. SUMMARY - CHAPTER 5

The preceding sections presented an overview of what can be considered to be the most comprehensive field experiment relative to pavement surface repairs. The data from this carefully designed experiment is anticipated to provide the highway agencies the needed information, not only with regard to the effectiveness of materials and procedures, but with respect to the cost-effectiveness of the specific repair operation.
6. **ACKNOWLEDGEMENTS - CHAPTER 6**

The author wishes to recognize the contractor, ERES Consultants, Inc., for contract H-105 and H-106. The contents of this paper reflect the views of the author and does not indicate approval or endorsement of these views by SHRP or its sponsors.

7. **REFERENCES - CHAPTER 7**


MINSA LT — A 5-Year Study to Minimize the Negative Effects of Salt

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and

Gudrun Öberg
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Sweden
MINSALT - a 5-year study to minimize the negative effects of salt.

Kent Gustafson and Gudrun Öberg, VTI

Abstract

In 1985 the Swedish National Road Administration (SNRA), the Swedish Association of Local Authorities and the Swedish Road and Traffic Research Institute were commissioned to initiate a research programme with the objective to reduce the negative effects of winter salting without detracting from traffic safety and trafficability. The programme titled MINSALT has been running for 5 years and will be completed with a final report in June 1991.

"Minsalt" was divided into three parts.
(a) Extension of salt-free regions and roads.
(b) New deicing methods.
(c) New deicing strategies.

Extension of unsalted regions included major experiments in three different parts of Sweden. On Gotland, an island in the Baltic Sea, both rural and urban roads were left unsalted from winter 1986/87 and onwards. In another county, Dalarna, the road network where salt was used were reduced during two test winters and the third test were performed on 50 km of road E4 in the northern part of Sweden. The aim of the three tests was to make a comparison of accidents, road conditions, corrosion, environment, road user opinion, road maintenance costs etc. when going from chemical deicing to winter maintenance without salt.

New deicing methods included studies of more effective salt spreading. The methods of pre-wetted salt and spreading of salt solution were tested with mostly positive results. The methods have been used primarily for preventive actions and in situations with hoar frost and thin ice. Due to mild winters there are still some questions about the applicability in more severe weather situations like heavy snowfall. New methods also included tests with alternative chemicals. Among others calciummagnesiumacetate (CMA) was tested as regards spreading, deicing, corrosion to metals and effect on cement concrete.

New deicing strategies included the development of road weather information system and the use of de-icing or anti-icing measures at critical locations. An experiment with salt spreading only in junctions and just before these has been carried out in a town. So called "Ice-retardant overlays", rubber asphalt RUBIT and Verglimit, with salt, have been tested at a test road in the south of Sweden. The comparison was made to conventional asphalt concrete as regards skid resistance (esp in wintertime), wear, economy etc.
MINSALT - A 5-YEAR STUDY TO MINIMIZE THE NEGATIVE EFFECTS OF SALT

Kent Gustafson and Gudrun Öberg
Chief Researchers
Swedish Road and Traffic Research Institute

PURPOSE AND ORGANIZATION OF TESTS

The purpose of the MINSALT project was to find out whether and how the harmful effects of salt in winter road maintenance could be reduced without a deterioration in traffic safety.

The Ministry of Transport and Communications decided on three different ways of minimizing the harmful effects of salt:

A Extension of the regions where salt is not used.
B New methods for snow and ice control.
C New strategy for snow and ice control.

The research programme was structured to conform with the proposal of the Ministry of Transport and Communications.

In regard to point A, tests with unsalted roads have been carried out in the counties of Gotland, Kopparberg and Västerbotten. Under point B, new mechanical and chemical de-icing methods and agents have been tested and evaluated. Experiments have also been carried out with the aim of developing more efficient methods of snow clearance and ice scraping. New de-icing strategies which have been tested under point C are spot measures with de-icing and anti-skid pavements (RUBIT rubberized asphalt and Verglmit), weather forecasts, information systems for road conditions, and training.

EXTENSION OF THE REGIONS WHERE SALT IS NOT USED

In the MINSALT project, the effects of winter road maintenance without using salt were studied in the counties of Gotland, Kopparberg and Västerbotten.

Extent

Before the test in Gotland, 215 km of the rural road network were salted all through the winter (A-salted roads) and 177 km were salted chiefly in the autumn (B-salted roads). In the town of Visby, 50 km of streets were salted. No salt has been used on any of these roads and streets since the winter of 1986/87.

In the winters of 1987/88 and 1988/89, salting was discontinued on 1,000 km of rural roads in the county of Kopparberg. Barely half of these were A-salted roads and the remainder consisted of B-salted roads. Not all the road network was included in the tests. About 500 km of the biggest roads continued to be salted during the two test winters.

In the county of Västerbotten, a 50 km stretch of the E4 trunk road was unsalted during the winters of 1987/88 and 1988/89.

A new test started in 1989/90 when the new MINSALT-strategy was put into practice in the counties of Kopparberg and Västerbotten.
Road conditions

Road conditions have been observed, mainly on weekdays, at a large number of places in the three counties and their reference areas in order to ascertain the standard of winter road maintenance.

On A-salted roads in the county of Kopparberg and on the island of Gotland, the number of days with icy and snow-covered roads increased 2-3 times (from 10-20 %) when salting was discontinued. In Visby, the county town of Gotland, the increase was slightly less than on the network of national roads covering the rest of the island. In Västerbotten, the increase was hardly 1/3 from about 30-40 % icy and snow-covered roads. The ice or snow removed by salting is replaced by moist or wet bare pavement. Road conditions were also observed on the network of B-salted roads and sanded roads in the county of Kopparberg. No difference in the proportion of ice or snow was observed on B-salted roads when salting was discontinued.

Traffic data

No changes in the traffic flow due to ice control methods and road conditions have been observable on Gotland. In Västerbotten, the traffic seems to be no heavier on days with bare pavement than on days with ice and snow, if measurements from the same winter period are studied. At the beginning and end of the winter, there are about 50 % more cars per day than in mid-winter.

Speed

Speed levels in similar weather and road conditions were somewhat lower in Gotland during the first unsalted winter than the winter before. The difference lies within the limits of normal random fluctuation however.

Studies in Västerbotten show that vehicle speeds on the first icy and snow-covered roads of the winter are lower than later on in the winter. In other respects, there are no major differences in vehicle speeds on the same roads and in the same conditions between the different winters. This signifies that traffic signs and other information about unsalted road tests had little effect on vehicle speeds. The results also indicate that reduced visibility in a snowfall, heavy rain, etc. has a greater effect on vehicle speeds than the condition of the road surface.

Traffic safety

An analysis of injuries reported by the hospital casualty departments on Gotland and in its reference area of Västervik shows that the category of road user most often injured in traffic consists of pedestrians and that the accidents occur mainly on ice or snow. The majority of all casualties are injured in “single” accidents, those in which no other vehicle or road user has been involved. The number of injured pedestrians increased on Gotland during the test while the number of other road users decreased. The total number of injured increased.

On Gotland and in its reference area, the days of the different winters have been classified according to the worst weather/road conditions that have occurred during the day. The accidents reported by the police on these days have been studied. What differentiates the accident rates of the two areas more than anything else is that the risk of an accident during autumn hoarfrost and also during freezing of wet roads is relatively much higher on Gotland than in the reference area. Salting rapidly eliminates road conditions such as these and it is therefore likely that the difference in accident potential is due to the difference in winter road maintenance. In the reference area, on the other hand, the risk of accidents during a snowfall is often higher than in the test area. This might be because the salted roads probably looks wetter and road users therefore do not think it is slippery.
Accidents per day

Control

Injured per accident

0.5

Number of days

0 233 0 22 22 36 8 3 48 29 1

Injured per accident

0.5

Number of days

0 233 0 22 22 36 8 3 48 29 1

O=bare pavement, no precipitation
W=bare pavement, snow-fall ≤2 mm
Q=bare pavement, wet snow-fall >2 mm
R=bare pavement, snow-fall >2 mm
X=ice/snow, snow-fall ≤2 mm
N=ice/snow, wet snow-fall >2 mm
K=ice/snow, snow-fall >2 mm
S=ice/snow, dry and cold weather
F=black ice
B=hoar frost, "autumn"
T=hoar frost, winter

Figure 1. Traffic accidents reported to the police during three unsalted winters (86/87, 87/88, 88/89) in Gotland county and in its reference area. The days have been classified according to the worst weather/road conditions that have occurred during the day.

Road safety studies carried out in the test areas in the counties of Gotland, Kopparberg and Västerbotten gave the following results for the network of national roads chosen for the tests.
The increase in accidents does not significantly differ from zero, which is to say that the increase in accidents could be due to pure chance. This applies to each of the counties individually and all of them together.

In the town of Visby, the number of accidents reported by the police has decreased by 36 % (-53 % -13 %). The confidence interval shows that the reduction significantly differs from zero, or in other words that the reduction in accidents is due to more than sheer chance.

Most of the results are not statistically significant. If spot estimates (i.e. the results obtained) are used however, with respect to the uncertainty, it will be evident that the accidents increased when salting was discontinued on roads with an annual average daily traffic (ADT) of more than 1,800 vehicles and decreased on roads with ADT <1,800 (not statistically significant as shown in figure 2).

![Figure 2](image)

**Figure 2.** Accident rate on roads that were unsalted during the test in relation to the annual average daily traffic (ADT). Regression curve with 95 % confidence region.

On the remaining A-salted roads in the county of Kopparberg, the costs of de-icing, and also the salt consumed, seem to have increased during the test winters. On these roads, a reduction in the number of accidents has also occurred. This indicates that additional resources ought perhaps to be introduced to reduce the proportion of slippery conditions on the roads with the heaviest traffic, where in actual fact winter road maintenance is already the most effective.
These conclusions agree with the results of earlier studies which have shown that in the case of a small percentage of slippery roads, the accident rate increases with an increasing percentage of icy and snow-covered roads up to about 15%. It then levels off or even decreases if the percentage of roads with a slippery surface increases even more.

**Attitude of road users**

Surveys of the attitude of road users towards salting the roads have been conducted in the three counties. The results indicate that road users would like salting to be discontinued. This preference is not as clear-cut in Västerbotten, where the negative attitude towards salting was softened after road users had tried driving on an unsalted stretch of the E4 trunk road. Professional drivers of heavy vehicles are less emphatic in their views on the salting of roads, but on the whole there is a clear majority against the use of salt. The reason Västerbotten differs somewhat from the other two counties might be because of the way the tests were arranged — in the middle of a salted road a short section was left unsalted. This could be why the change in road conditions was thought to be troublesome and undesirable. This sudden difference in road conditions was not arranged on Gotland and road users there were able to adapt more readily to the new conditions.

If reduced salting is judged to result in more accidents, then road users say they are willing to accept a lower speed limit in return for unsalted roads. Those living in the county of Västerbotten, however, are somewhat less inclined to agree to this than road users in other parts of the country.

**Corrosion**

Corrosion on the following vehicles/test panels has been studied on Gotland and its reference area in Västervik-Gamleby.

- Police cars.
- Mobile exposure of test panels on trucks and police cars.
- Stationary exposure of test panels to atmospheric corrosion.

Stationary exposure of test panels and mobile exposure on trucks were carried out during the second half of the winter before the test and throughout the first test winter.

The absence of salt on the roads of Gotland during the test period resulted in a large reduction of corrosion on unprotected steel and a reduced corrosive effect on paint and anti-rust agents. There was an 80–90% reduction of corrosion on unprotected steel. In an earlier test the effect on painted surfaces was shown to be the same, but a period ten times longer was needed to achieve an equivalent degree of corrosion. Short-time exposure of unprotected test panels also showed a strong connection between corrosion and salting frequency.

Rust damage increased just as rapidly on the Gotland police cars as on those in the reference area during the first year when the winter roads on Gotland were still salted. From the autumn of 1986, when salting in the winter was discontinued on Gotland, a distinct difference, 50% less in the winters, was observed in the number of new rusty areas. It should be pointed out here that all cars in the study had already been exposed to road salt for 1-2 years, during which period the corrosion could conceivably have started. The difference in the number of rust observations between cars on Gotland and in Västervik increased continuously with time and would probably increase still further if the study were to be continued.
Environment

Studies of the natural environment were carried out in four selected areas on Gotland during the year before salting was discontinued and for three years afterwards.

In general, the spreading of salt on the roads has had little effect on these areas, probably because salt has been used to a lesser extent on Gotland than on the mainland, for example. Despite this, a certain effect of salting could be documented in all areas. In at least two of the areas, a definite effect of not salting the roads could be shown in the form of a reduced salt content in the ground water and soil. Why the effects of putting a stop to spreading salt on the roads are so clearly evident after only three seasons is because the types of soil in these two areas are relatively coarse and well-drained. In consequence, the salt entering the soil is comparatively easily diluted and carried away. In more fine-grained soils (clay, silt, etc.) it probably takes much longer (5-10 years) for the effect on the environment to become apparent after salting has been discontinued.

Experience and work environment of the road administratives

Chemical de-icing was discontinued on Gotland five years ago. Neither the Road Administration on Gotland nor Gotland municipality could consider a return to salting the roads.

The supervisors at the local road maintenance area in the county of Kopparberg consider that the experiment exposed them to greater stress at work. A higher standard of alertness was necessary and they were worried about not being able to carry out winter maintenance in an acceptable manner.

In Västerbotten reactions to the experiment are mainly favourable, although supervisory personnel felt it caused them more anxiety and mental strain, while other personnel considered their work to be more physically strenuous. It is also considered that spreading salt is the best anti-skid measure in certain situations such as freezing rain and hoar frost in the autumn.
Road maintenance costs
Winter maintenance costs have been affected to varying degrees from no change at all up to twice their previous level.

NEW METHODS FOR SNOW AND ICE CONTROL
The harm caused by salt can be reduced by using methods and materials, both chemical and mechanical, which more effectively counteract existing or probable slippery conditions. New de-icing methods and agents have been tested in several different projects with the aim of finding more effective ways of improving skid resistance which do not have the negative effects of salt.

In regard to chemical de-icing — i.e. salting — spreading methods have progressed from earlier dry salting to the spreading of prewetted salt and saline solutions.

A number of different chemical alternatives to NaCl have been tested. In particular, calcium magnesium acetate (CMA) has been studied more closely in regard to its ice-melting capacity, corrosiveness and effect on concrete.

Pre-wetted salt, saline solution
The spreading of prewetted salt or a saline solution are methods which have been used for many years and they are now fairly well-known techniques. The salt can be prewetted either when loading it onto the spreading vehicle or when spreading it.

Water, NaCl or CaCl₂ solutions or other suitable solution can be used for prewetting the salt. Water and NaCl solution have been used in Sweden. Water is used for the simpler method of prewetting the salt when loading it, while a saturated NaCl solution is used for prewetting with salt spreader and when a saline solution is spread on the roads.

Special spreaders for prewetted salt were developed and put into road maintenance service during the 1980s. In addition to the hopper for the dry salt, these spreaders have a solution tank (capacity approx. 2 m³), pump, spray nozzle and electrical equipment for regulating the amount of solution.

As a rule, 30 % by weight of the saturated NaCl solution is added to the dry salt. The rate of dry salt is similarly reduced by 30 % at the same time, which means that the amount of salt spread on the road is automatically reduced.

The advantages of using prewetted salt instead of dry salt, as shown by the tests carried out with wet salt spreaders and the results of subsequent practical winter road maintenance applications, include:

- The salt is spread more uniformly with less wastage at the roadside.
- The salt adheres to the road surface better.
- Prewetted salt has a faster and more durable effect.
- The method can be used at lower temperatures.
- Spreading speed can be increased.
- In some cases the road surface dries out quicker.

Dry salt can also be prewetted in a simpler manner by spraying a saline solution or water over it as it is loaded onto the spreading vehicle. The advantages of this prewetting method are that conventional spreaders can be used and that little capital need to be invested in new equipment.
Practical experience of the method as used by road maintenance crews has been gathered through two questionnaires and the results are summarized as follows:

- The proportion of water ought to be 80-100 litres per tonne, in the light of special tests.
- It must be possible to measure the amount of water because equipment malfunctions if insufficient water is used.
- The method has been tested at temperatures down to about -12°C, but is generally used down to -6°C.
- Spreading speed has been 50-60 km/h.
- As a rule, 2-3 tonnes of salt has been prewetted, although tests have been carried out up to 8 tonnes. The amount of salt which can be prewetted depends to some extent on the size of the spreader.
- Most of the road maintenance areas (about 90 %) have reported that the method produces good results and that they intend to continue using it.

Simple prewetting with water makes it possible to: Gain the advantages of prewetted salt with conventional spreaders at an extremely low investment cost.

One limitation of the method is that the spreader should not be loaded with more than 2-3 m³ of salt to ensure that it is thoroughly prewetted before spreading. However, this is enough for about 60-90 km of preventive salting. The limitations of the method make it better suited to road maintenance areas with lower traffic densities on their salted roads. In areas with more trafficked roads, on the other hand, prewetted-salt or saline-solution spreaders are more suitable for chemical de-icing.

**Spreading of saline solution**

De-icing with a saline solution entails spreading a saturated salt solution containing about 20-25 % by weight of NaCl. Spreading this solution on the roads therefore corresponds to only about 1/4 of the amount of dry salt.

Two types of spreading equipment for saline solution were tested, both speed-independent, which is to say that the amount spread is not dependent on the speed of the
vehicle. The normal capacity of the tank of saline solution is 8 m³. After an initial testing additional spreader units were acquired by the Road Administration for the 1989/90 season and the total number of spreader units in use was about 80. In the same way as in the previous winter, the spreading of saline solution was followed up with a questionnaire concerning the methods and spreading equipment used. The 26 local road maintenance areas and two municipalities covered by the study gave their views and reported on the results they had obtained.

Figure 5. Spreaders for saline solution: 1 Nozzles, 2 Spinners.

Summarizing, experience gained during the three winters shows that:

- The method is considered to be extremely effective as a preventive measure and for dealing with hoarfrost on the roads.
- During a snowfall, the method is of doubtful merit. On wetter roads and where ice has already formed, the method is similarly of doubtful merit or downright unsuitable.
- A saline solution of 20 g/m² (corresponding to about 5 g/m² of dry salt) is sufficient in the majority of cases.
- The method has been tested on roads and motorways with ADTs ranging from 1,500 to 12,000.
- Spreading has been possible at speeds of up to 60 km/h.

**CMA — Alternative to NaCl?**

Studies of CMA were carried out in Sweden before the MINSALT project was initiated. These studies were included in the project once it was under way. To begin with, a small quantity of CMA was manufactured on a laboratory scale but when it was marketed commercially during the latter part of the project, tests were conducted with the proprietary product. The studies that were carried out mainly at the VTI, as well as various studies conducted by other research institutes, were concerned with CMA's melting properties, corrosiveness and effect on cement concrete.

CMA's freezing point reduction, the lowest temperature at which melting can occur, varies according to the Ca/Mg ratio between about -10°C and -28°C (NaCl about -21°C). The lowest and optimum freezing point is obtained with a Ca/Mg ratio of about 3/7-2/8 (in mol). The two CMA products, ICE-B-GON and Clearway CMA, have a ratio of 3/7.
The melting effect of CMA does not vary so widely, however, because of the Ca/Mg ratio, but depends more on the shape, size and density of the particles. Melting ability has been tested on blocks of ice at different temperatures, CMA having a slightly poorer total melting effect than CaCl₂ and NaCl but slightly better than urea. To notice is although that CMA has a very slow initial melting reaction while NaCl and especially CaCl₂ has a very rapid melting effect.

Perhaps the greatest positive effect of CMA as compared with NaCl is reduced corrosion. Several different corrosion studies have been carried out with CMA. Corrosion tests on car body steel showed that CMA is much less aggressive than NaCl and CaCl₂, for example.

Several different studies concerning the effect of CMA on cement concrete have been carried out by the VTI and other Swedish institutes. Additionally, the National Testing Institute has performed a large number of analyses on cubes of cement concrete that were exposed to CMA and other substances during a series of tests at the VTI.

As shown by freeze/thaw tests, the chloride salts clearly peak at a concentration of 3-4%. After declining with slightly rising concentration, the degree of damage increases dramatically for high concentrations of CaCl₂ and MgCl₂. Here the chemical effect manifests itself. For NaCl, on the other hand, the degree of damage clearly diminishes with a rising concentration and is extremely small for a saturated solution. According to freeze/thaw tests, CMA's incidence of damage rises in direct proportion to its concentration up to the same level as NaCl's maximum (which occurs at about 3%).

![Scaling kg/m²](image)

**Figure 6.** Concrete-Frost Testing acc. to Swedish Standard 137236. Tests with 3 % solutions of NaCl, CaCl₂, MgCl₂ and 3—25 % solutions of CMA. Weight loss after 56 cycles. Trends for NaCl, CaCl₂ and MgCl₂ according to Verbeck & Klieger, 1957.
To study the effect of de-icing agents, and CMA in particular, on cement concrete under more realistic and varying conditions, field experiments were carried out at the VTI during the years 1986-1990. At the end of the period of exposure, the National Testing Institute tested and analysed the concrete specimens in respect of compressive strength, tensile splitting strength, carbonation depth, frost resistance, chloride and acetate content and also carried out a thin grinding analysis.

The results of the tests show that all exposure damage in respect of CMA was less than for NaCl and other chloride salts. The results of the tests are summarized as follows: "there is nothing in the analyses performed to indicate that de-icing with CMA would cause more damage to cement concrete than that caused by NaCl. The spraying of the various agents on the specimen cubes that was carried out during the tests is certainly more intensive than occurs in the course of ordinary road maintenance work, but it would no doubt be unwise to draw too far-reaching conclusions from the results at hand. After all, the periods of exposure were extremely short in relation to the expected lifetime of the structures".

CMA’s biggest drawback as an alternative de-icing agent is its price, which is about 15-20 times higher than the price of NaCl, and there is nothing to indicate that this could be reduced to any great extent in the future, not even in the case of large-scale production.

The dominating expense connected with the use of CMA is the direct cost of production. The principal advantages to be gained from switching to CMA are reduced corrosion of the vehicle fleet, reduced chemical aggression on bridges and other cement concrete structures, and reduced environmental damage (ground water, vegetation).

Against the above background, the likelihood of CMA replacing NaCl must be considered small. Furthermore, the cost/benefit sides of the balance sheet are somewhat complicated in a scenario where a changeover to CMA is made. The use of CMA could result in major savings for the individual motorist in the form of reduced corrosion, while the major costs associated with CMA would be borne by the road maintenance authority.

**PROPOSAL FOR NEW WINTER ROAD MAINTENANCE STRATEGY**

The MINSALT project has resulted in a proposal for a new winter road maintenance strategy which, in the light of the results of research and the experience gained from the MINSALT project, shows how winter road maintenance can be organized so that its objectives can be attained. By adopting the proposed strategy, we believe it is possible to reduce salt consumption by about 20-40 % compared with what it was before the MINSALT project.

**Rural areas**

In rural areas these aims can be achieved by meeting the following standard requirements (described in functional terms) of roads in the winter:
As far as is practicable, the following requirements apply to national and regional roads:

- if there is a danger of slippery road conditions, anti-skid measures should be taken to prevent the occurrence of such conditions.
- de-icing and other anti-skid measures should be carried out before peak traffic periods, and the time for these measures on national and regional roads respectively should be 1 and 2 hours.
- snow deeper than 3 cm should not be allowed to remain on the road.
- the road should be free from snow not more than 2-3 hours after the snow has stopped falling. Strings of slush should not be left on the road.
- snow should be cleared away from the verges when the roadway is free of snow.

As far as is practicable, the following requirements apply to local roads:

- in the event of extreme slipperiness caused by precipitation, the roadway should be non slippery not more than 4 hours after the precipitation has stopped.
- snow deeper than 8 cm should not be allowed to remain on the road.
- the roadway should be cleared of snow not more than 8 hours after the snow has stopped falling.

Adoption of the following measures, methods and resources is proposed. It is important to use the right method at the right time.

- Where chemical de-icing is concerned, the method used should be the one with the lowest salt consumption.
- In the case of preventive salting, the salt should always be prewetted with 80-100 litres of water per tonne of salt if equipment for spreading a saline solution or prewetting salt with a saline solution is not available.
- Prewetted salt should not be spread wider than 4 m, regardless of the road width. The application rate compensates for the actual width to be de-iced.
- Salting in conjunction with snow ploughing should only be carried out when there is a danger of compaction or freezing.
- Chemical de-icing should normally not be done on local roads. If possible, salt free abrasives like crushed stone aggregate should be used.
The lowest temperature for chemical de-icing should be
-12°C on national roads
-8°C on regional roads
-3°C on local roads

At lower temperatures than the above, the material with the best adhesion, having regard to durability and availability, should be chosen.

Sand mixed with salt should not be used when it is possible to use crushed limestone, natural sand or crushed stone aggregate.

Snow-clearing equipment should be adapted to the prevailing snow and temperature conditions.

Snow should be cleared from the roads as quickly as possible. If possible, salting should be delayed until the snow has stopped falling.

CONSEQUENCES OF DIFFERENT WINTER ROAD MAINTENANCE STRATEGIES

The consequences of the MINSALT strategy are compared with winter road maintenance before MINSALT.

Advantages and disadvantages of the MINSALT strategy as compared with the earlier salting strategy:

Advantages

- More bare roads (wet/moist) because slippery conditions are prevented from occurring to a greater extent than earlier and because spreading takes less time.
- Shorter journey times because of less ice and snow.
- Reduced salt consumption on each spreading.
- Less salt used in population centres. Total salt consumption reduced on national roads unless the number of spreadings increases excessively.
- Natural environment affected to a lesser degree if smaller amounts of salt are spread on the roads.
- Improved working environment because spreading can be better planned and be carried out more often during off-peak periods.
- Traffic is less disturbed by spreading vehicles.
- Cleaner street environment owing to the use of less salt.

Unchanged

- Fuel consumption.

Disadvantages

- Corrosion may increase if salting is carried out on more occasions. The smaller amount of salt used on each occasion will probably result in no change.
- More crushed stone aggregate are spread in population centres, resulting in higher costs for cleaning sewers etc.
Unknown
- Effect on traffic accidents.
- Effect on concrete constructions.
- Use of studded tyres.
- Dirt spray.
- Effect on the road administrativ's total costs.

Reference
DEICING SALT
Its Use and Effect on Road Safety and the Living Conditions of Roadside Trees and Shrubs

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DEICING SALT

Its use and effect on road safety and the living conditions of roadside trees and shrubs

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Abstract

Deicing salt is used to avoid ice or snow on road surfaces and to facilitate snow clearing. For ecological and economical reasons wet salt should always be used; dry salt represents an adequate alternative only in very few circumstances and even then wet salt can be used without any disadvantages. As short a time as possible should elapse between ice formation and salt spreading. This can be accomplished by improving the regional meteorological service and by routing optimization. In the interests of safety, common economic and traffic regions, e.g. the EC-member countries, demand identical levels of service as far as winter service is concerned.

Investigations conducted by Darmstadt Technical University have shown that the accident rate after spreading deicing salt falls approx. to one-third of the values recorded before road clearance commenced. In the Federal Republic of Germany (without the five new states) during one winter period some 1,000 deaths or serious injuries, approx. 1,500 slight injuries and some 6,000 accidents with material damage could be avoided within one hour after deicing salt being used outside built-up areas (excluding motorways). The theoretical assumption that motorists drive considerably slower on slippery roads, thus obviating the need for deicing salt, is not verified; evidence shows that speeds are reduced, but not to the extent required to compensate the lower friction coefficient. The accident figures increase substantially in case no deicing salt is used.

The number of injuries suffered by pedestrians has increased considerably in areas where deicing salt is not permitted on footways.

Long-term ecological investigations at Giessen University, have revealed that no systematic damage to roadside shrubs outside built-up areas can be determined. Only on motorways the larger amounts of salt spread give rise to individual cases of salt induced damage to plants at a distance of up to approx. 10 m from the edges. These almost exclusively involve contact damage which does not lead to an accumulation of chloride in the plant. The occurrence of such cases can be reduced to an acceptable level by an appropriate choice of plants and locations, suitable plant care, structural measures and optimization of winter service.

The importance of protecting life and health of persons by deicing salt in conjunction with economic benefits outweigh the negative effects on roadside shrubs that suffer avoidable or reparable damage locally in very few cases.
DEICING SALT - Its use and effect on road safety and the living conditions of roadside trees and shrubs

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1. THE USE OF DEICING SALT

Due to the use of deicing agents in winter services, traffic safety and economy of the traffic flow have been improved considerably. Comparing studies on the effects of dry salt and of wet salt show that considerable savings of salt can be achieved with the use of wet salt, without affecting the efficiency. Both wet salt technologies "Wet Salt 5 (FS5)" and "Wet Salt 30 (FS30)" base on the principle of moisturing, while the latter number indicates how much percentage by weight of liquid are admixed to the salt.

In case of Wet Salt 5, the salt is sprayed with approx. 5 % of water in that moment of loading, when it is falling from the conveying belt to the spreader; it can also be sprayed with NaCl or CaCl2 solution. This quantity of liquid is sufficient to create the required adhesive effect of the salt, which it retains when it is spread from the spreader to the road, later.

In case of Wet Salt 30, the liquid, exclusively saline solution, is admixed in such a form that the solution is carried along on the spreader vehicle in a tank and is sprayed to the salt at the moment of spreading. However, for this procedure approximately 30 percentage by weight of liquid are required, in order to achieve immediately such a consistency, which permits adhesiveness on road surfaces.

The studies executed by the Technical University Darmstadt on behalf of the Federal Ministry for Traffic, show that FS30 is clearly advantageous to FS5. With the use of FS30 salt savings between 24 and 44 % are possible; with the use of FS5 savings from 5 to 20 % are possible. Thus, it is recommended that deicing salt shall become the standard in winter services while the use of dry salt should be an exceptional case. Due to its cost-favouring and quick realisation, FS 5 can be accepted as an intermediate solution, however, a general adaptation to FS 30 should be the long-term-target.

To reach the optimum in using wet salt it requires that the capacity of the alkaline tanks, the length of the routes to be maintained, as well as the location of the filling stations where salt and alkaline can be loaded, have to correspond to each other.

The effects of the deicing salt are the more favourable the shorter the periods between the formation of ice on the road
and spreading of salt are. This can be achieved by reliable information on the road condition and by intensive training of the personnel. The formerly customary reliance on weather forecasts on radio and television are no longer sufficient for optimum control of the activities of the winter services. By means of intensification of the cooperation with the meteorological stations, informations concerning the climatological conditions of the regions and areas, as well as informations on air humidity, temperature, direction and speed of weather fronts have to form part of the service planning. Various European countries have achieved considerable progress in this context over the recent years.

Moreover, ice prediction equipment, which are installed in the pavement can be used in connection to, their warning effect to furnish informations to the meteorological stations.

2. EFFECTS ON TRAFFIC SAFETY

Over a long time there were no reliable, generally acceptable, informations concerning the effectiveness of winter service with use of deicing salt on traffic safety. Another study, also performed by the Technical University of Darmstadt on behalf of the Federal Ministry for Traffic, resulted in quantifiable results.

4,700 accidents from four winter seasons, with 80 dead persons, 573 severely wounded persons, 1,321 lightly wounded persons and a material damage of approximately 55 Mio. Deutsche Mark have been analysed and evaluated. Moreover, in connection with this examinations 60,000 speeds have been measured, 13,000 of these on winter-icy roads. The study proves that during winter conditions the rate of accidents - this is the number of accidents related to the driven kilometers (accidents/1 Mio Veh.km) - is approximately six times as high as during non-winter conditions. In sloping areas this accident rate during glazed frost rises to the 10-fold of this value compared with normal road conditions. Upon removal and spreading with salt, the accident rate decreases to approximately one third to one fourth of the value before deicing salt had been spread on the road.

Another perception is that accidents with severe personal damages are approximately five times as high on roads with winter glaze, under normal conditions. Thus, the erroneous assumption that only the number of small accidents with property damages are rising during ice and snow due to the throttled speed has been refuted. Fig. 1 shows the driving-dynamical connection.

The rate of accident costs - this is the costs of accidents related to the driven kilometres (accident cost DM/1 Mio Veh.km) - on roads with glazing frost is approximately six
times higher than during periods without icy conditions.

The projection found of the figures in the investigated area on all roads outside build up areas in former West Germany — Autobahnen excluded — shows that already within the first hour after activity of the winter service with use of deicing salt has started.

- approx. 1,000 dead persons or severely wounded
- approx. 1,500 lightly wounded
- approx. 6,000 accidents with damages

can be avoided.

Furthermore, considerable gasoline quantities are saved if the roads are cleared and deiced: a considerable factor concerning environment protection.

Safety first is the main objection for the use of deicing salt in winter services; the efforts for economic progress and better traffic flow are only the second important factors.

There are indications that at locations where the use of deicing salt had been avoided on walkways, the accidents of passengers have been increased considerably during glazed frost over the recent years. However, there are no statistically based figures, because passengers' accidents are not registered by the police.

3. INFLUENCE OF THE DEICING SALT TO THE LIVING CONDITIONS OF ROADSIDE TREES AND SHRUBS

In discussions concerning environment protection and deicing salt and its use in winter services has been evaluated critically, variously. The importance and sensitivity of and for environment protection in our society require serious discussion with raised reproaches and assumptions.

Unfortunately, the discussions concerning the damages of deicing salt to vegetations lack well-founded perceptions. Very often only general arguments concerning the influences to the ecological system were brought up; which might be conclusive in certain individual cases. The most important finding of Paracelsus, "Sola dosis facit venenum", however, has not been taken into consideration: Not the quantity of the deicing salt has been discussed, only the fact that snow and ice are fought with salt.

In order to gain representative results, the Highway Administration of Hessen placed in 1985 an order to the University of Gießen concerning a long-term-study of the connections between use of deicing salt and its influence on
the living conditions of roadside trees and shrubs. The study covered all trees and shrubs outside of cities and villages, which are placed along federal and state roads up to 10 m distance along Autobahnen up to and at 20 m distance from the road edge. Soon it showed that only the influence of the salt on plants along the federal and state roads was of such minor importance, that further studies along these roads could be stopped after 2 years (1985 and 1986) upon request of the Plant Ecological Institute of the University of Gießen. After more than 30 years of winter service no systematical damages of the road plants could be determined; it goes without saying that even on this roads the winter service authorities raise all measures to use as less salt as possible.

However, the plants along the Autobahnen (federal motorways) showed larger damages in the nearer area. The results of the long-term study prove that the most efficient influence results the salt-containing spraying fog, which is whirled up by passing vehicles. Clearly, most of the damages base on such contact with the salt foam. Shrubs up to 6 m from the carriageway edge are especially damaged: approximately 80% of the averagely and largely damaged shrubs can be found at these locations. Fig. 2 shows the percental portion of the individual stages of damage of the examined shrubs, at different distances of the carriageway edge. One will realize, that damages decrease, with increasing distance from road edge, there are no damages caused by the use of salt in surrounding areas. The chloride concentration shown in Fig. 3, using a pine-grove as example, shows that, moreover, a clear Cl-concentration decrease can be registered in the course of the vegetation period.

Shrub damages due to salt contamination of the soil were only to be found in a small scope and locally limited. They are only clearly recognizable in the course of the vegetation period and they are, contrary to contact damages due to the chlorid stored in the plant fibers, more durable. Enduring recovering of damaged shrubs is recognizable after various successive winter seasons with the use of small deicing salt quantities. However, in no case it could be determined that soil contaminations due to salt caused an accumulation of chlorid in the vegetable fibers.

The soil examinations show that the salt contamination of the soil is small and that it can no longer be proofed at 2 m distances from the carriageway edge. Only in some cases, such as sloping embankments or on separation strips of parking lots on Autobahnen, contaminations due to the deposit of salt-containing snow or the penetration of draining-off surface water could be proofed.

Damages of shrubs can be preserved by means of choosing such plants, which are resistant against salt contact. Moreover,
new planting should consider a closed, graduated plant vegetation, in order to avoid large distribution of the salt foam to the depth of the terrains. Thus, such narrow new planting of shrubs could protect the trees therebehind. In addition, plant distances to the carriageway should be large, where ever possible: the more distant from the road edge, the less endangered.

By means of workmanlike care of the shrubs, early cutting, well considered irrigation and adapted fertilization, damages due to the use of deicing salt can be improved, the resistance of the plants can be increased.

4. CONCLUSION

If one considers the advantages and damages of modern winter services based on scientific studies concerning safety and the influence on road plants, one can determine that the proofed increase of safety for life and health of the road users as well as the economic advantages are disproportionate higher than locally limited, and partially avoidable or removable, damages to road plants.

Other examinations in Sweden and Finland concerning the effects of deicing salt to road vegetation and traffic safety resulted in similar perceptions.
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Fig. 1:
Physical connection during slowing-down process.

Fig. 2:
Percental portion of stages of damage at different distances from carriageway edge, after winter with great use of salt.

Fig. 3:
Cl-concentration in the course of the vegetation period (Example: Pine-grove) for different classes of damage.
Improving Concrete Pavements Through SHRP Research

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IMPROVING CONCRETE PAVEMENTS THROUGH SHRP RESEARCH

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ABSTRACT

Substantial improvements in concrete pavements will be achieved through studies performed in three of SHRP research areas: the Long-Term Pavement Performance (LTPP), the Highway Operations and the Concrete and Structures.

The LTPP studies aim a better understanding of the effects on performance of traffic, environment, material properties, and other details. The Highway Operations studies encompass studies that are concerned with the effectiveness and cost of alternative methods, materials, and equipment for preventive pavement maintenance and for pavement surface repair under a range of climatic and load conditions. The Concrete and Structures studies encompass studies that are concerned with the different aspects of the concrete used in highway pavements and structures. These studies will lead to substantial improvements in concrete pavements through refinement of pavement design methods, optimization of rehabilitation strategies and maintenance options, enhancement of concrete constructibility and performance, improvement of concrete durability, and assurance of uniformity and quality.

The paper identifies the concrete pavement-related products resulting from SHRP research and highlights their contribution to improving concrete pavements.
1. INTRODUCTION

The Strategic Highway Research Program (SHRP) is an accelerated research program seeking solutions to common, but complex problems. Substantial improvements in concrete pavements will be achieved through studies performed in three of SHRP research areas: the Long-Term Pavement Performance (LTPP), the Highway Operations and the Concrete and Structures.

The LTPP studies aim a better understanding of the effects on performance of traffic, environment, material properties, and other details. This improved understanding of pavement performance will result in methods for rational pavement design, understanding of the effectiveness of certain design features and their impact on pavement life, and methodologies for the selection of optimum pavement rehabilitation strategies.

The Highway Operations studies encompass studies that are concerned with the effectiveness and cost of alternative methods, materials, and equipment for preventive pavement maintenance and for pavement surface repair under a range of climatic and load conditions. These studies will lead to the development of guidelines for selection of the most effective materials, processes, and equipment for pavement maintenance and surface repair.

The Concrete and Structures studies encompass studies that are concerned with the different aspects of the concrete used in highway pavements and structures. These studies will lead to substantial improvements in concrete pavements through enhancement of concrete constructibility and performance, improvement of durability, and assurance of uniformity and quality.

A summary of concrete pavement-related improvements resulting from SHRP research are listed in Table 1.

2. PAVEMENT DESIGN IMPROVEMENTS

Much of the current pavement design procedures are based on the results of the AASHO (American Association of State Highway Officials that later became American Association of State Highway and Transportation Officials, AASHTO) Road Test that was completed in 1960. This Road Test dealt with limited ranges of pavement designs, material types, climatic conditions, and other details. In addition, the Road Test involved limited traffic volume and type applied over a short duration. As a result, the AASHO original performance models included many limitations, some of which were later revised through the introduction of empirical parameters. The
Table 1. Concrete Pavement-Related Improvements Resulting from SHRP Research

<table>
<thead>
<tr>
<th>Research Area</th>
<th>Pavement-Related Improvement</th>
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<tr>
<td>Pavement Design, Construction, Maintenance and</td>
<td>Reliable, accurate design methods</td>
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<td>Rehabilitation</td>
<td>Improved construction features and details</td>
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<td>Methods for selection of optimum rehabilitation strategies</td>
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<td>Guidelines for selection of the most effective materials, processes, and equipment for</td>
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<td>Durability</td>
<td>Alkali-Silica Reactivity (ASR):</td>
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<td>Rapid test for ASR detection</td>
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<td>Freeze-Thaw:</td>
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<td>Rapid test for screening D-cracking susceptible aggregates</td>
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<td>Rehabilitation of D-cracking affected pavements</td>
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<td>Water content determination of fresh concrete</td>
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<td>Optimization of Highway Technology</td>
<td>Documentation of concrete mix design, curing, etc. for specific applications</td>
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<td></td>
<td>Expert system</td>
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The LTPP studies will evaluate the long-term performance of over 500 concrete pavement test sections in their first construction period to establish the effect on performance of design features, climate, traffic, material properties, and details. Also, the LTPP studies will evaluate the performance of some 400 test sections to assess the effectiveness of different strategies and techniques for the rehabilitation of concrete pavements. These studies will result in substantial improvements in concrete pavement design, construction, and rehabilitation by achieving the following:

1. Reliable design methods that will enable better selection of pavement structure and optimization of construction materials. For example, these methods will identify the conditions for which the use of a stabilized bases will be necessary to insure adequate pavement life and the conditions for which less expensive bases may be adequate. Consequently, the optimum design in certain situations may require a thin concrete slab on a stabilized base while a thick slab on an untreated base may provide an optimum design for other situations.

2. Identification of improved construction features and structural details, their benefits, and their applications. For example, the use of tied shoulders or widened lanes may be more effective and economical means for improving performance than increasing pavement thickness. Also, in-pavement drainage and open-graded permeable bases may be an effective measure for improving performance and extending pavement life in wet climates while dense-graded aggregate bases may be adequate in dry climates. Consequently, the optimum design for some situations may require installation of in-pavement drainage in conjunction with thin pavement layers while thick pavements without drainage may provide the optimum design for other situations.

3. Optimization of rehabilitation strategies. LTPP research will provide the means for comparing the performance and economics of possible rehabilitation alternatives and will identify methodologies for selecting the optimum option for each specific condition. For example, CPR (Concrete Pavement Restoration) with or without an asphalt overlay may be an effective means for rehabilitating pavements in poor condition in spite of the larger investment, as infrequent maintenance will be required. However, a thin asphalt concrete overlay together with limited restoration may prove more economical for pavements in a fair condition.
The Highway Operations studies will evaluate the long-term performance of over 100 concrete pavement test sections to assess the effectiveness of joint and crack sealing, and undersealing for preventive maintenance. In addition, several hundred concrete pavement test sections will be used to assess the effectiveness of several spall repair materials and crack and joint sealants. These studies will result in improvements in concrete pavement performance by achieving the following:

1. Reliable methods that will enable better selection of the maintenance treatments that will work best under given site and climatic conditions together with necessary guidelines for quality control. Highway Operations research will provide the means for comparing the performance and economics of possible preventive maintenance alternatives and will identify guidelines for selecting the optimum option for each specific condition.

2. Methods that will enable selection of the most effective materials and procedures for repairing pavement surfaces. Highway Operations research will provide the means for comparing the performance and economics of potential spall repair materials and crack and joint sealants and will identify methodologies for selecting the optimum material for each specific condition.

3. DURABILITY

Deterioration of concrete pavements may result from the reaction between alkalis in the concrete and certain aggregates. Also, pavement deterioration may result from the effects of cyclic freezing and thawing. SHRP research deals with these two aspects of concrete durability to help enhance the ability of pavements to resist the adverse effects caused by these actions.

3.1 Alkali-Silica Reactivity

Although potentially reactive aggregates are not widespread, aggregate supply sources are becoming depleted in many parts of the country. Therefore, there is an economic incentive to utilize such aggregates when the possibility of reactivity can be avoided. SHRP research will develop rapid and more accurate tests to identify reactive aggregate. Thus, the use of such aggregate can either be avoided or limited to different types of concretes and environments. Also, SHRP research has resulted in a rapid method to identify ASR causing pavement damage. Consequently, remedial steps can be taken to mitigate such damage and extend pavement life.

The rapid test for alkali-silica reactivity (ASR) identification consists of applying or spraying a dilute solution of uranyl acetate on the concrete surface and viewing the treated surface under ultraviolet light in a dark background. The presence of ASR gel is revealed by a yellow-green fluorescent glow. This test takes only a few minutes to complete. The use of this test in the field will
help inspectors detect the presence of alkali-silica reactivity and take appropriate measures to mitigate its adverse effects and increase pavement life. To keep moisture from entering ASR affected pavements and thus retard its effect and extend pavement life, SHRP is investigating a range of chemical and physical treatments.

Another development resulting from SHRP research is a rapid and reliable test for the detection of alkali-silica reactive aggregates/cement combinations. The current practice is to screen the aggregates through a combination of petrographic analysis and laboratory tests. In addition, the "mortar-bar" test is used for evaluating cement-aggregate reactions. The test is slow, it may take one year to produce expansion, and interpretations of the validity of the test procedures and the significance of the results may vary. SHRP research will yield a rapid and more accurate test for identifying reactive aggregate/cement mixtures prior to construction. The use of low alkali cement, fly ash or other pozzolans with reactive aggregate may counteract the alkali-silica reactivity and produce a durable concrete mix that can be used for highway pavements. To accomplish this objective, SHRP research will also develop engineering criteria and specifications for pozzolanic mixtures that can be used as agents to counteract alkali-silica reactivity. With this rapid test and specifications, inexpensive aggregate/cement mixtures that are excluded currently on the basis of the existing non-conclusive tests, may prove adequate. Consequently, durable, but less expensive, concrete pavements can be constructed.

3.2 Freeze-Thaw Durability

Deterioration of concrete pavements due to freezing and thawing effects results from the inability of the water-saturated concrete pavement to resist the internal pressures caused by this cyclic action. Intentional air entrainment has been, and continues to be, most helpful. Also, aggregates can play a significant role in the capability of concrete to resist freezing and thawing. For example, D-cracking is a form of deterioration caused by cracking that originates in saturated aggregate in concrete that has frozen and then thawed. SHRP research will develop rapid and reliable test methods that will identify such aggregates and how they can be used in concrete without causing such distress. Also, the research will develop techniques for extending the service life of existing pavements that exhibit distress due to D-cracking.

The rapid aggregate test will identify aggregates that are susceptible to D-cracking. The test is based on the hypothesis that freeze-thaw damage of aggregate is caused by the internal pressure generated by water inside the pores during freezing. It is also based on an observation that aggregates associated with D-cracking exhibited a predominance of pore size in the 0.04 to 0.2 mm diameter range. The test simulates the pressure experienced by the aggregates during freeze-thaw cycles by subjecting the aggregate, submerged in water, to a high pressure. In this test, a high pressure is applied to force water into the pores. When this pressure is released rapidly, the compressed air tries to force water out of the pores, thus simulating the internal pressures
generated during freezing. Aggregate susceptible to D-cracking will not withstand this high pressure and thus fracture. This test will enable identification of those aggregates susceptible to D-cracking and thus avoid their use in concrete pavements to ensure improved performance.

4. CONCRETE MATERIALS AND PERFORMANCE

There are potential advantages for using high performance concretes in highway pavements. Such concretes include high strength concretes typical of those used in tall buildings and high early strength concretes, those made with high range water reducers and with pozzolens such as fly ash and silica fume, and other admixtures designed to enhance performance. SHRP research in the area of high performance concrete will identify the materials, mix design, and other details necessary to produce durable concretes with the following strength properties:

   a. Very high strength, i.e. 10,000 psi within 28 days
   b. High early strength, i.e. 5,000 psi within 24 hours
   c. Very early strength, i.e. 3,000 psi within 6 hours

In addition to the improved strength properties, this concrete will exhibit improved performance characteristics, particularly with regard to alkali-silica reactivity, freeze thaw, abrasion, shrinkage, and fatigue.

The use of high performance concrete in highway pavements will yield the following benefits:

   a. Reduced construction time and rapid pavement repair
   b. Improved pavement durability and resistance to wear
   c. Ability to perform repair during adverse weather conditions
   d. Increased pavement life

The use of high performance concrete for concrete pavements will provide the optimum use of materials for certain applications, particularly where lengthy lane closure and traffic disruption cannot be tolerated.

5. QUALITY CONTROL/QUALITY ASSURANCE

Concrete pavement performance can be improved through the use of better quality control/quality assurance measures. SHRP research in this area has resulted in a quality assurance/quality control (QC/QA) system for new construction and a system for evaluation of existing concrete. The QC/QA system for new construction will ensure the structural soundness of the finished concrete and its adequacy for the intended purpose through a series of field tests applied at critical points in the construction process. This system will include existing tests as well as newly-developed, rapid, reliable non-destructive tests and acceptance criteria. Examples of these are tests for measurement of water content in the concrete
mix and air entrainment of fresh concrete.

Rapid measurement of the in-situ air content of fresh (plastic) can be accomplished by a newly-developed "Fiber Optic Air Meter" at many stages during construction, such as mixing, placing, consolidating, and finishing. The device is based on the measurement of the changes in light intensity reflected from a glass optical fiber tip inserted into the concrete as a function of the air content of the concrete. With proper calibration, the amount of reflected light can be correlated with the air content of the concrete. This device is now being refined and evaluated on pavement construction projects. Measurement of air content can be completed within 10 minutes at any stage during the construction.

The water content of the concrete can be determined quickly and accurately at the construction site prior to placement with the use of a microwave oven. This technique should provide better indicator of concrete quality than the current slump test which provides a measure of concrete workability.

6. OPTIMIZATION OF HIGHWAY CONCRETE TECHNOLOGY

To help better utilize the results of SHRP concrete research and recent developments in concrete technology, SHRP will produce documentation that will enable highway agencies select the concrete-processing techniques, materials, and materials combinations that have the best potential for superior performance. Also, SHRP will develop an "expert system" that will allow highway agencies to diagnose concrete pavement problems, select appropriate materials, mix designs, and curing procedures for both new construction and rehabilitation to ensure optimum performance. With this documentation and "expert system," pavements with properties superior to those of currently constructed will be designed, built, and rehabilitated. Consequently, substantial improvements in performance and service life of future highway pavements will be expected.

7. SUMMARY AND CONCLUDING REMARKS

Improvements in concrete pavement technology will result from the implementation of SHRP research findings. These improvements will be attributed to the following SHRP research results:

1. Improvement of concrete pavement design methods. These design methods will allow optimum selection of pavement layers, use of improved structural details, and selection of optimum rehabilitation strategies and thus result in low life cycle costs.

2. Improvements in maintenance practices. The implementation of suitable preventive maintenance treatments and use of proven maintenance materials and processes will extend pavement life and result in low life cycle costs.
3. Techniques for enhancement of concrete durability. These techniques will identify those durable materials that can be used in concrete pavements. It will also identify less durable materials that can be modified with other additives or treatments to produce durable concrete mixes suitable for use in highway pavements. Also, these techniques will identify appropriate means for improving existing pavements that suffer from durability damage.

4. Techniques for quality assurance and quality control. These techniques will ensure that a good quality concrete is delivered for use prior to its placement. Also, consistency and uniformity in construction will be achieved thus reducing the need for frequent repair.

5. Concrete materials and performance. This work will identify concrete mixes with superior strength and durability properties that are suitable for pavement applications, allowing shorter construction time and lower life cycle cost.

6. Documentation of optimization of highway technology. This documentation will help highway agencies select suitable materials, mix designs, curing procedures, and other details for new construction and rehabilitation that will ensure improved performance, durability, and low life cycle costs.

SHRP research in the areas of Long-Term Pavement Performance, Highway Operations and Concrete and Structures will yield several products that will help improve concrete pavement technology. The products illustrated in this paper clearly indicate that through the implementation of SHRP research, concrete with improved strength, enhanced durability, and consistent quality will become the everyday's concrete for concrete pavements. These qualities together with the improved design methods developed in this research will enhance the prospects of concrete pavements and their competitive role in the pavement market.
Optimization of Highway Concrete Through Combined Use of Particle Packing Modelling, Rheological Studies, Computer Simulations and Compaction Simulations

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President
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Denmark

and

Per Just Andersen
PhD, Industrial Researcher
G M Idorn Consult A/S
Denmark
OPTIMIZATION OF HIGHWAY CONCRETE THROUGH COMBINED
USE OF PARTICLE PACKING MODELLING, RHEOLOGICAL STUDIES,
COMPUTER SIMULATIONS AND COMPACTION SIMULATIONS

by

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&
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Birkeraed, Denmark

ABSTRACT

Concrete technology for highway construction is undergoing rapid development. New cementing materials such as fly ash, slag and silica fume are being used increasingly and the concrete tends to incorporate an increased number and range of chemical admixtures. In concert, the construction approach and equipment are constantly being revised. As such, the classical, comprehensive experience and commensurate guides developed for "pure" portland cement concrete is in need of supplementation. However, due to the very dynamic nature of the recent developments in highway concrete technology, developing comprehensive, new guidelines is likely an insurmountable task; such guides would further very quickly become outdated.

In realization of the increased need to be able to tailor highway concrete to particular conditions, G.M. Idorn Consult A/S has further developed technology originally produced under SHRP C201 and in related, supplementary research.

The developed approach involves the tailoring of the concrete technology based on a number of logical steps and supported by tests and analyses the basis of which was developed in-house.

Computerized particle packing models are used to analyze the available aggregate sources in order to achieve the highest degree of aggregate packing while observing, where relevant, the needed adjustments to accommodate air-entrainment. Rheological studies are used to optimize the choice and dosage of chemical admixtures using both instrumentation which measures on pastes and on mortar as well as equipment which measures the rheological behaviour of fresh concrete. Using a specially designed controlled compaction instrument the compaction of fresh concrete, in particular low-slump concrete, can be simulated. For instance, if a given concrete composition using a particular piece of construction equipment can be compacted to a certain density, the compactive effort can be modelled. On this basis it is possible rationally to test a range of candidate mix designs in the laboratory at low cost rather than full scale at higher cost. Where required, as for instance for slip-formed median barriers, the form stability of the freshly molded concrete is measured using equipment developed based on experience from the porcelain industry.

The paper will include examples of applications of the above approach.
High-Performance Road-Surfacing Concrete
with Good Resistance to Wear by Tyre Studs

Mårten Nilsson
Head of Section
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Sweden
HIGH-PERFORMANCE ROAD-SURFACING CONCRETE WITH GOOD RESISTANCE TO WEAR BY TYRE STUDS
Speaker: Mårten Nilsson
Head of Section
Swedish Society of Civil and Structural Engineers

1. INTRODUCTION

Rutting of roads is due to the following factors (see Fig. 1 in the figure appendix).

* Wear/abrasion
  - removal and abrasion of material from the top of the road surfacing due to wear by tyre studs and erosion.

* Deformation
  - compacting of the material at different levels in the body of the road
  - fragmentation of the material, above all in the unbonded layers
  - lateral displacement of the masses due to instability.

* Deformation due to frost damage

In Sweden, Norway and Finland, about 60-90% of the total rutting on the busy, heavily loaded tarmac road network is considered to be caused by tyre stud wear. On the Stockholm motorway slip roads - carrying an annual daily traffic of around 50 000 - 100 000 vehicles and with 85% of the cars normally being fitted with studded tyres in the winter - maintenance resurfacing is usually carried out every other year, in spite of the fact that the resistance of the tarmac surfacing to wear by tyre studs has been improved very substantially in recent years. This cyclic maintenance work causes appreciable disturbances to the flow of traffic.

Against this background, the National Road Administration in Sweden took renewed interest in concrete roads as an alternative and supplement to tarmac roads.

We have established that earlier concrete roads, built between the 1920s and 1960s, proved to be very resistant to wear by tyre studs.

In the spring of 1989, it was therefore decided to build two new concrete roads in Sweden, one of which was the E4.65 Arlanda road - a 1.7 km test stretch of motorway - and the E6 Falkenberg bypass - a 16 km stretch of motorway.
As one element in the planning of these roads, systematic development work was undertaken with the aim of acquiring knowledge of the wear-affecting factors on concrete wear surfaces. The evaluation was carried out principally in the Norwegian "Veijslitern" road-testing machine (see Fig. 2 in the figure appendix).
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2. WEAR AND RUT DEPTH DEVELOPMENT

In addition to the material properties, a number of traffic and climatic parameters also affect wear by tyre studs. The most important of these are:

* speed of the traffic
* humidity of the road surface
* stud weight and stud projection
* axle load on studded axles.

The rut depth and rut form (concentrated ruts or wide ruts) are besides the wear and deformation properties also affected by:

* road width and road geometry
* traffic intensity and traffic composition
* side obstacles, such as tunnels, and slope gradients.

In order to compare the stud wear and deformation properties of various wear layer materials as objectively as possible, the following concepts have been created:

**Stud wear material parameter = NSm, defined as**

* mass worn away at the end of the service time, excluding intial wear, in g per km of road and passage of cars with studded tyres on an ice-free and snow-free road surface at an average speed of 95 km/h and 30% humid road surface.

As an example, a car with studded tyres on all four wheels wears away 2 kg of mass on a 100 km journey at an NSm value of 20, which is a substantial amount.

For concrete roads in Sweden, the NSm-value varies between about 4 and 20. For tarmac roads, the value varies between about 10 and 50.

In tests on the "Veijisliten"-machine, the wear properties are specified in the unit SPSvs, where NSm = 0.63 x SPSvs.

**Rut depth development material parameter = NSU mm/year, defined for a concrete road without deformation as**

* rut depth increase, including initial wear, on
average per year, during the service time of the wear layer, for the slow lane on a motorway

* with annual daily traffic = 10 000 equivalent car passages
  - studded tyre season = 4 months
  - stud proportion = 75%
  - speed = 95 km/h
  - wet/dry road surface = 30%/70%
  - ice-free and snow-free road surface = 90%
  (= 900 000 studded-tyre equivalent car passages per year).

Fig. 3 in the figure appendix shows the relationship between the stud wear material parameter (NSm), wear value from "Veijslitern" (SPSvs), rut depth development per year (U), rut depth development material parameter (NSU) and service time in number of years (Funktionstid, ÅR) for varying maximum rut depth (12, 15, 18 mm). According to the example, the service time is 10 years for a maximum rut depth of 15 mm at an NSm of 10 under the conditions in accordance with the NSU definition.

3. STUD WEAR PROPERTIES OF CONCRETE

3.1 Wear relationship and recipes for road concrete

Most of the experience of modern concrete has been obtained from surfacing concrete on bridges built from 1985 onwards. The wear on six of these is measured annually out in the field. The first Swedish tests in Veijslitern were carried out on fibre-reinforced pre-mixed dry mortar concrete developed specifically for use as wear surface concrete on bridges. Numerous recipes were tested in conjunction with the current road projects. Work is in progress on calibrating the road wear machine tests (principally Veijslitern) with measurements of wear and rut depth out in the field.

The results of wear measurements on concrete produced to Swedish concrete recipes are plotted in Fig. 4 in the figure appendix.

It is very difficult to isolate the influence of various parameters on the wear properties. Extremely few parameters in a recipe can be varied without other parameters also being affected. However, regardless of the concrete quality and the component materials, road concrete must always meet reasonable requirements on workability.

Unfortunately, no good method is available for reliable measurement of the workability of road concrete.
—produced for pouring by a slip-form-paver— and it is therefore difficult to specify which of the above recipes meet reasonable requirements. However, the reforming meter has been employed to a limited extent in an endeavour to evaluate the workability.

3.2 Aggregate and its composition for road concrete

Natural materials are often preferable for laying reasons, whereas it has been clearly demonstrated in Norway that crushed material produces higher resistance to wear. However, it is impractical to employ more than a minor proportion of crushed material.

Fine material (smaller than 4 mm)

Reports from Norway also indicate that the wear resistance is improved with a declining proportion of fines, specified as material smaller than 4 mm. The recommended proportion of material smaller than 4 mm should not exceed 32% (see Fig. 5, ideal curves for aggregate in accordance with the Norwegian model and in accordance with SABEMA, i.e. the current Swedish recommendation).

The quality of the fines also affects the wear resistance, although probably to a relatively marginal extent. Norwegian results suggest a total influence of between 20 and 30%, with greatest influence being on wet wear. Finnish experience indicates that the results are partially dependent on the testing machine employed.

In Germany, principally for laying reasons (see Fig. 6), it is recommended that:

* the sum of cement, any other additives and fine sand smaller than 0.25 mm should be less than 450 m³
* the proportion of aggregate passing a 1 mm screen should be less than 27%
* the proportion of aggregate passing a 2 mm screen should be less than 30%.

Maximum particle size

If the particle size is increased, more aggregate can be used and less binder is necessary for a given strength. It will be shown later that an increased aggregate content results in higher wear resistance. In countries where tyres stud wear is non-existent or negligible, the maximum aggregate particle size used is normally 32 mm. The relative wear resistance, under conditions that are otherwise as similar as possible, e.g. same K value, is specified in the table below.

<table>
<thead>
<tr>
<th></th>
<th>12mm</th>
<th>16mm</th>
<th>22mm</th>
<th>32mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>VTI RAPPORT 372A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
According to Norwegian studies, the difference is not particularly great in the range which is of interest from the comfort aspect, i.e. up to around 22 mm. However, the difference will be greater for wet wear. Finnish experience suggests that the difference is greater in the Finnish wear testing machine (more abrasion of the surface and lower speed). However, it is uncertain which machine best reflects realities.

**Particle larger than 8 (4) mm**

Concrete used for housbuilding construction normally has a content of around 40% of crushed rock or gravel (larger than 8 mm) with a maximum particle size of 16 mm. The objective for wear-resistant road concrete is to have as much rock as possible. The rock (larger than 8 mm) content is therefore normally about 50% with a maximum particle size of 16 mm, both in Norway and in Sweden. The exact values are dependent on the composition and rock characteristics.

One way of achieving a higher rock content is to carry out gap-grading of the aggregate. A higher wear strength is obtained in this way. Gap-graded aggregate is normally used in Europe for road concrete, and up to 70% rock content can then be used with 32 mm maximum particle size (see Fig. 6).

Economic reasons are the primary incentive for using a high proportion of rock. The disadvantage of various types of gap-graded concretes is that they are sensitive to variations in the component materials, and separation in the concrete mass can therefore easily occur, which may have devastating consequences when exposed to wear by studded tyres.

Norwegian and Finnish results both demonstrate that a smooth grading curve results in about 20% lower wear than a traditional "European" curve with particle size gaps. Norwegian results also demonstrate that the fineness modulus is of major importance to wear -up to around 20%. This shows that the risk of separation is serious. The use of gap-graded aggregate should therefore be restricted, so that only minor grading gaps are employed.

Most of the recipes used in Sweden have so far followed the Norwegian recommendations. However, it would appear as though interest is growing in achieving a wear-resistant concrete on the basis of the European experience of gap-graded concrete.
3.4  **Binders (cement, etc.) for road concrete**

The choice of cement is guided principally by characteristics other than merely wear resistance. One of the factors guiding the choice of cement is that, in higher quality classes, it must be strong and must still offer good workability. In addition, alkali-silicon reactive aggregate is often used in strong concrete, and a low-alkaline cement therefore guarantees good performance over a long period of time.

The quantity of cement is relatively high for normal wear-resistant road concretes, being up to 380kg/m³, as against around 320 kg/m³ in Europe. For a well-formulated concrete, this is guided by workability. Studies have otherwise shown that the reduced quantity of cement need not be basically cause reduced wear resistance (see Fig. 7).

The effect of different binders on wear is not particularly well documented. Different types of cement have been tested in Norway, with and without silica dust additive. Cement with and without slag has been tested in Finland. In Sweden, we have principally tested slow construction cement, in certain cases with silica dust additive. Minor experiments have also been carried out in Sweden with finely ground slow construction cement.

One problem in the use of silica dust is to ensure adequate dispersion, in order to avoid major agglomerations of dust, which may cause alkali/silicon reactions.

3.5  **Compressive strength**

The strength of concrete is dependent on many factors. For road concrete, one of the items discussed is how the mixing-in of air (to safeguard resistance to frost) affects the strength and wear resistance.

Early Norwegian investigations indicate that the air content has other negative effects on the wear resistance, in addition to increasing the porosity. However, comprehensive laboratory studies in Sweden have revealed that the air content as such does not have greater negative effect than that resulting from increased porosity (lower density).

This means that the strength of a given concrete is determined by the "water-air binder factor", i.e. \((\text{water} + \text{air})/\text{binder}\).

The table below specifies the relative wear resistance under conditions that are otherwise as similar as
possible.

<table>
<thead>
<tr>
<th>MPa</th>
<th>K55</th>
<th>K70</th>
<th>K85</th>
<th>K100</th>
<th>K150</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finland, VTT</td>
<td>0.4</td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Norway, SPSvs</td>
<td>0.6</td>
<td>1.0</td>
<td>1.4</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>Assessed in Sweden</td>
<td>0.7</td>
<td>1.0</td>
<td>1.2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In all probability, it is actually the tensile strength which is decisive to the wear characteristics.

It is not easy to isolate the influence of compressive strength, since the recipes are of necessity fairly dissimilar. Later results from Norway (analyses of the development of wear with time on existing road concrete, rolled compacted concrete and rut repair concrete) indicate greater relative differences than those indicated in the table above. This may be explained by the fact that concretes with higher strength levels normally contain a great deal of cement, which means that the reaction time is fairly long.

From the Finnish studies, it is claimed that the compressive strength coefficient is relatively higher for initial wear and wet wear. From the material that has been analysed, it is difficult to assess whether the above relationships apply to different proportions of aggregate, maximum particle size and different sand qualities. It would be extremely interesting to be able to remove a series of plates at intervals of a few years from a given road to obtain a better means of determining the relationships.

3.6 Rock quality (geology)

The relative wear resistances of different rock types are specified in the table below, under conditions which are otherwise as similar as possible:

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Finland</th>
<th>Sweden</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartzite</td>
<td>1.0</td>
<td>0.7-1.4</td>
</tr>
<tr>
<td>Granite</td>
<td></td>
<td>0.7-1.2</td>
</tr>
<tr>
<td>Dura X-100</td>
<td></td>
<td>1.3</td>
</tr>
<tr>
<td>Porphyry</td>
<td></td>
<td>1.3</td>
</tr>
<tr>
<td>Gabbro splitt</td>
<td></td>
<td>1.4-1.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.6</td>
</tr>
</tbody>
</table>

Very wide differences are reported from Norway in the wear of different rock qualities. Particularly wide differences are reported for wet wear. The content of water in different rock materials appears to have a major influence on the wear (fast compression wave?).

Crushed aggregate material appears to have better wear properties, probably because it has better adhesion to the mortar and higher tensile strength. On the other hand, the concrete is difficult to work. Different granites and quartzites have proved to have very wide differences in wear properties.
It is difficult to make a comparison between countries, since we do not have entirely identical laboratory methods for describing the rock quality, and because a geological description is very complicated to refine. The results indicate that the same relative results cannot be expected as for asphalt masses. The method of operation of the various testing machines also appear to influence the results.

### 3.7 Wet and dry wear

The relationship between wet and dry wear varies within fairly wide limits. In the Veijslitern, the wet wear is tested last, which may possible underestimate its relative magnitude. In the Finnish testing machine, dry and wet wear are tested alternately. It is very difficult to convert the relationships to reality, since the humidity varies on the road, whereas the plate in laboratory testing is "saturated with water".

In the context of concrete, it is generally stated that the relationship is between 1.5 and 3 times the dry wear. Both the rock material and the mortar are of importance. The rock quality is claimed in Finland to be of major importance, and the relationship is stated to be lower for reduced particle size. For the concrete recipes employed in Sweden, they can be assumed to be in the lower range.

Much great variations are reported from laboratory tests on asphalt. In the model for rut development in the field employed for forecasting the rut depth development, the following relationships are employed in Sweden:

\[
\begin{align*}
Cu=0/100 & \quad Cu=10/90 & \quad Cu=20/80 & \quad Cu=30/70 & \quad Cu=40/60 \\
0.83 & \quad 0.88 & \quad 0.94 & \quad 1.0 & \quad 1.06
\end{align*}
\]

where \(Cu=30/70\) denotes 30% humid road surface during the season when studded tyres are used.

### 3.8 Initial wear

The initial wear depends on how well a given concrete mixture is adapted to a specific item of laying equipment. In the field, the initial wear can be expected to be around 2—3 mm.

### 3.9 Speed

Norwegian investigations in the Veijslitern machine indicate that wet wear is proportional (1) to the speed, whereas dry wear is exponential (1). The Norwegian relationships in Veijslitern can roughly be expressed as tabulated below:

<table>
<thead>
<tr>
<th>Km/h</th>
<th>55</th>
<th>75</th>
<th>95</th>
<th>110</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cu</td>
<td></td>
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The following relationship is employed in Sweden in the model for rut development in the field employed for forecasting the rut depth development.

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ORSAKER TILL SPÅRBILDNING

SLITAGE

PACKNING NEDBRYTNING

SIDOFÖRSKJUTNING KONSTANT VOLUM

HASTIGHET

80%

PÅMONTERADE SUPER SINGEL

VTI RAPPORT 372A
FIG 3
FIG831AV.XLC

Provning i Veislitern, SPSvs

Uppmätt/antagen tryckhållfasthet efter 28 d (MPa)

NSM

NSU (mm/år)

VTI RAPPORT 372A
Exempel tyskland

sikt, mm

spänningsgrad

jämn kurva

siktkurva
CEMENTMÅNGD – SPS

Norskt cement MP

Mängd cement kg/m³

Fig 7
Maintenance and Repair of Highway Concrete Bridges: A Case Study

Ibrahim Al-Babtain
Engineer
Ministry of Communications
Kingdom of Saudi Arabia

and

Adil M Abbas
Dr
Ministry of Communications
Kingdom of Saudi Arabia
Maintenance and Repair of Highway Concrete Bridges: A Case Study

Eng. Ibrahim Al-Babatain
Maintenance Dept.
Ministry of Communications
Saudi Arabia

and

Dr. Adil M. Abbas
UN Team
Research and Materials Dept.
Ministry of Communications
Saudi Arabia

Abstract

Studies carried out in Saudi Arabia have shown that, the hot climate and salt contaminated environment lead to premature loss of durability for many reinforced concrete highway bridges, caused primarily by corrosion of reinforcement. This paper presents a case study for three of these bridges. In the first one conventional repairs have been carried out, whereas in the other two bridges cathodic protection systems have been installed on the substructure of one of them and the superstructure of the other one. The performance of these systems under the prevailing environmental conditions is thoroughly evaluated.
MAINTENANCE AND REPAIR OF HIGHWAY CONCRETE BRIDGES: A CASE STUDY
Eng. Ibrahim Al-Babtain & Dr. Adil M. Abbas
Ministry of Communications
Kingdom of Saudi Arabia

1. INTRODUCTION

1.1 General

The construction environment in the Eastern Province, Saudi Arabia, is characterized by many features. The first of these is the use of salt contaminated aggregates and salt laden mixing and curing water during construction. This has led to a substantial increase in chloride contents in some concrete structures to a peak value of 31.65 lbs/yd³. This is significantly well above the threshold level of chloride content of 1.13 lbs/yd³ to 1.65 lbs/yd³ which can activate the corrosion, in presence of humidity and oxygen, for normal range of cement content of 335 kg/m³ to 490 kg/m³ used in the kingdom[1].

The second feature is the high water/cement ratio used to compensate the loss in workability resulted from the increased temperature. This may develop somewhat a porous microstructure and facilitate plastic and shrinkage cracking which would break the protection system provided by concrete to the reinforcement and make it more vulnerable to corrosion. In addition to that in some cases the concrete cover was found to be very poor in quality and inadequate in thickness.

Moreover, the high ground water levels in some localities and the prevailing hot and humid weather conditions have combinedly created an environment highly conducive to corrosion.

These factors have led to a wide pre-mature loss of durability of concrete structures in that region caused primarily by corrosion of reinforcement. Many of relatively recently constructed concrete highway bridges have shown remarkable signs of deterioration demonstrated by corrosion induced cracking and damages.

The large scale of the problem has raised the alarms and brought the attention to set up a criteria and programme for an intensive repairs and rehabilitation of these bridges. Among the choices available are (i) conventional repairs and (ii) cathodic protection treatment. This paper discusses our first trials in this field. The outcomes of these trials is believed to be of special importance as it may laid the foundation for future trends in rehabilitation.
and maintenance of concrete bridges of similar problems in the Kingdom.

1.2 Overview

The deck of Al-Nabba/ Abu-Maan bridges and sub structure of Aramko bridge were observed to be exhibiting signs of concrete distress. To study the reasons and discover the extent of the problem, a site survey and structural analysis were undertaken. The results of this survey clearly demonstrated that chloride ions had penetrated the concrete matrix and had reached the level of the reinforcement in sufficient quantity to initiate steel corrosion. This in turn led to expansive forces, caused by generation of corrosion products, rupturing the concrete cover resulting in gemination. The levels of contamination observed ranged from two to five times the level at which corrosion will initiate. Furthermore, the limited investigation indicated that this level of contamination was spread throughout the whole structure.

At that time, the level of concrete deterioration was such that it was evident the corrosion reaction had passed the slow initiation stage and reached the accelerating propagation stage. Therefore, for remedial works to be cost effective immediate action had to be undertaken not only repair the areas of existing damage but to prevent further damage to the rest of structure. To accomplish this objective, only two long term solutions are available. The first of these, is the removal of all chloride contaminated concrete, application a protective coating to the rebar and replacing the concrete with an impermeable concrete followed by a waterproofing membrane. In this case, however, this would effectively entail entire deck replacement. The second alternative is the installation of cathodic protection. The basis of cathodic protection is to reduce the potential of the the metal structure which is to be protected. There are two ways of achieving this reduction in potential. One is to use an external direct current source (impressed current cathodic protection) and an auxiliary anode. The other is to connect the structure to a metal such as aluminum, magnesium or zinc, which takes up more negative potential than steel and which will naturally corrode to provide cathodic protection current[^2].
3. CONDITION SURVEY

3.1 Chloride Profiles

The chloride profiles were determined for Abu-Maan and Nabya bridge decks. As shown in Fig. (1) the distribution of chloride ions is not uniform through the depth of the slab. The maximum chloride content for both decks is in the outer layer and decreases with depth to a minimum value. This may be attributed to the evaporation of mixing which may leaves the soluble salts at the outer layers.

3.2 Delamination Survey

For Abu—Maan bridge deck after loose and severely damaged concrete cover of the deck has been chiselled out and removed, the deck’s top surface area was mapped carefully to outline the boundaries of the delaminated zones as indicated in the Appendix. Over 80% of the deck slab has already damaged by gemination. Poor cover and high level of chloride are believed to be the main reasons of delamination of concrete.

3.3 Potential Map

In Al Nabya bridge deck the picture is different were field observations has revealed that the damage was not so excessive as in Abu-Maan bridge. The deck was potentialy mapped and the following ASTM C 876 (80) has been used for the interpretation of the results:

- Less than 200 mv = 90% probability no corrosion
- Greater than 350 mv = 90% probability corrosion
- Between the above two figures = uncertain region

For Span 2 of Al-Nabya bridge, the results indicate that about 7% of the area is in the 90% no corrosion region, about 27% is in in 90% corrosion of which 13% above the pitting level of -500 mv, but the greatest part of about 66% is within the uncertain region.

For span 4, the survey showed a different picture. Again about 7% of the area is within the 90% no corrosion region, but the uncertain region is reduced only to about 24%. the remaining part of about 67% is in the uncertain region of which about 43% above the pitting threshold of -500 mv.

The pitting potentials observed were confirmed by the existence of many pits and by the extremely localized necking observed once steel had been exposed (Appendix).
It is worth mentioning, the potential survey is more closely correlated with the actual corrosion status and concrete delamination than the asphalt survey. For example, span 2 had 27% of its area greater than -350 mv and about 32% delamination but showed almost no signs of patching (Appendix).

3.4 Corrosion Damage to Reinforcement

3.4.1 Abu-Maan Deck Slab

Inspection of the deck’s surface after the removal of the damaged and delaminated concrete cover revealed that the entire top steel of the slab has disintegrated and vanished from vast areas of the deck, pieces of discontinuous rebars are found at some locations and at some others only traces of the rebars could be found. It is fair to assume that the entire top steel reinforcement layer has ceased to function structurally for sometime.

3.4.2 Al-Nabya deck slab

The steel reinforcement underlying most delaminations was found to average 30% loss of section from the top down giving the classical "D" shape to the bars. In the worst areas, however total loss of section had occurred or, more commonly, pitting and necking down to 10% steel remaining. In these areas additional steel was placed and tied into the existing framework. In all cases the lower mat of steel has been found to be corroded only to less than 2.3% in some locations and most of it was completely free from corrosion. This may indicate that this steel mat was acting as the cathode in the operating corrosion cell.

4. REPAIR STRATEGY

4.1 Conventional Repair

Since over 80% of the deck slab of Abu-Maan bridge has already been damaged by delamination and most of steel reinforcement of top slab has been corroded, a conventional repair scheme found to be more suitable for the bridge. Briefly, it consisted from removal of the top 20 cm of the delaminated concrete in the top slab after supporting the bridge for structural purposes, cleaning and sand blasting the main steel, placing the new steel reinforcement and casting a new layer of concrete after using a bonding agent to provide adequate structural bond between the old concrete and the new one.
4.2 Cathodic Protection Trial

The amount of corrosion present on the deck slab of Al-Nabyia bridge and on the substructure (Pier + Pile cap) of Aramco bridge has favoured the option of CP as a possible repair technique for this bridge.

By installing the CP system, the Ministry of Communications will be able to make full technical judgements on the following issues:

* Does CP halt corrosion of steel reinforcement in concrete.
* How much effective is it when compared with conventional repair methods.
* Can it be effectively engineered to be applicable to a wide range of structures.
* How the operating conditions experienced in the Kingdom of Saudi Arabia, which are different from those found elsewhere, will affect this method of repair.

What is the extent of ongoing monitoring/maintenance required.
Is it a cost effective solution.

4. COST ANALYSIS

4.1 Cathodic Protection

As it is the first trial the system has been designed to more strict requirements than may prove necessary in the future. However, it is considered too important trial to risk any possibility of poor report. The system has been designed for a minimum operational life of 20 years before first maintenance, although experience of other CP systems worldwide indicate that lifetimes in excess of this value may be expected.

The breakdown cost in US Dollars for CP installation as follows:

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<th>Substructure</th>
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<td>Anode Installation</td>
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<td>Overlay Concrete</td>
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<td><strong>500 m²</strong></td>
<td><strong>240</strong></td>
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<tr>
<td><strong>m² Total Cost/Square Meter</strong></td>
<td><strong>183.4</strong></td>
<td><strong>$ 227.7</strong></td>
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</table>
The increase in cost/m² for substructure over that of deck is due mainly to increased cost of overlaying concrete as it requires forming and scaffolding.

4.2 Conventional Repair

The break down cost (In US Dollars) for the conventional repair carried out for Abu Maan Bridge is as shown in the following table:

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<td>concrete overlay</td>
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<td>Reinforcement</td>
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<td>Waterproofing</td>
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<td><strong>Total cost</strong></td>
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<td>Total area</td>
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<td>Time (Days)</td>
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<tr>
<td>Cost/square meter</td>
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5. RECOMMENDATIONS

Unfortunately at the time of writing this paper the depolarization stage for cathodic protection is still under progress and accordingly the questions regarding the technical feasibility of the Cathodic Protection technique would remain unanswered for a while. The full technical assessment will be complete by the conference date.

Regarding the economical feasibility the Cathodic Protection is believed to be more feasible than the conventional methods on the basis of direct expenditure and time.

6. REFERENCES


2. "Corrosion of Metals in Concrete", ACI Committee 222.

APPENDIX

FIG (1) Chlorid Profile For Abu–Maan & Nabya Bridge Decks

FIG (2) Sections Of Deck Slab Showing Variation Of Concrete Slab Thickness Over Voids, Abu–Maan Bridge Deck.

FIG (3) Aplan Of Abu–Maan Bridge Deck Showing Delamination Zones

FIG (4) Location of Cathodic Protection Installation For Nabya Bridge

FIG (5) Cracks Exist On Surface Of Asphalt As A Result Of Delaminated Concrete Slab, Nabya Bridge Deck

FIG (6) Potential Map Of Al–Nabya Bridge, Span–2

FIG (7) Delamination Survey / Concrete Breakout / Sandblasting And Replace Concrete, Al–Nabya Bridge Deck

FIG (8) ARAMCO Bridge / Cathodic Protection Installation
FIG (1) CHLORIDE PROFILE FOR ABU-MAAN AND NABYA BRIDGE DECKS
SOUTH SIDE SECTION OF DECK SLAB

NORTH SIDE SECTION OF DECK SLAB

Figure 2. Sections of Deck Slab Showing Variations of Concrete Slab Thicknesses Over the Voids
Figure 3. A Plan of Bridge Deck Showing Delamination Zones as of 20/3/1990

VTI RAPPORT 372A
FIG 5 CRACKS EXIST ON SURFACE OF ASPHALT AS A RESULT OF DELAMINATED CONCRETE SLAB.
REPLACE CONCRETE
BY SHOTCRETE

NOT DELAM AREAS

FIG 7 DELAMINATION SURVEY / CONCRETE / BREAKOUT
SANDBLASTING AND REPLACE CONCRETE

VTI RAPPORT 372A
BASE PLAN

SIDE ELEV. OF BASE & COL.

FIG8 ANODE ZONE/CONNECTOR

VTI RAPPORT 372A
BASE PLAN

SIDE ELEV. OF BASE & COL.

ANODE ZONE / CONNECTOR

ANODE MESH SPACED AS SHOWN & SPOTWELDED ON TI STRIP

CONDUCTOR (TI) STRIP
ELEVATION COL - 2

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VTI RAPPORT 372A
| CELL NO. | 4H ON | 4H OFF | 4H SHIFT | 1-DAY ON | 1-DAY OFF | 1-DAY SHIFT | 2-DAY ON | 2-DAY OFF | 2-DAY SHIFT | 3-DAY ON | 3-DAY OFF | 3-DAY SHIFT | 4-DAY ON | 4-DAY OFF | 4-DAY SHIFT | 5-DAY ON | 5-DAY OFF | 5-DAY SHIFT | 6-DAY ON | 6-DAY OFF | 6-DAY SHIFT | 7-DAY ON | 7-DAY OFF | 7-DAY SHIFT | 8-DAY ON | 8-DAY OFF | 8-DAY SHIFT | 9-DAY ON | 9-DAY OFF | 9-DAY SHIFT | 10-DAY ON | 10-DAY OFF | 10-DAY SHIFT |
|----------|-------|--------|-----------|----------|----------|------------|----------|----------|------------|----------|----------|------------|----------|----------|------------|----------|----------|------------|----------|----------|------------|----------|----------|------------|----------|----------|------------|----------|----------|------------|----------|----------|------------|----------|----------|------------|----------|----------|------------|
## COST ANALYSIS COMPARISON
### CATHODIC PROTECTION

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<tr>
<th></th>
<th>Super Structure</th>
<th>Sub Structure</th>
<th>Conventional Repair</th>
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<td>Concrete Repair</td>
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<td>Overlay Concrete</td>
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<td>Time (Days)</td>
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