Proceedings of the Conference
STRATEGIC HIGHWAY RESEARCH
PROGRAM AND TRAFFIC SAFETY
ON TWO CONTINENTS in
Gothenburg, Sweden,
September 18 – 20, 1991, Part 6

- Long-Term Pavement Performance
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– Long-Term Pavement Performance
Abstract (background, aims, methods, results) max 200 words:

Papers presented at the seminar were as follows:
Early Evaluations of SHRP LTPP Data and Planning Sensitivity Analyses (Rauhut, J B, Jordahl, P, Darter, M I, Hawks, N F, Pendleton, O and Owusu-Antwi, E);
Specific Pavement Studies: Key Issues and Potential Products (Hanna, A N and Hawks, N F);
Expected Changes to the AASHTO Design Guide (Hawks, N F);
Cost Effectiveness of Asphalt Concrete Overlays - The Canadian Approach (Sparks, G A, Nesbitt, D and Williams, G);
Long Term Pavement Performance Trials and Data Analysis in the United Kingdom (Kerali, H R and Potter, J F);
SHRP-NL: A Research Project Parallel to SHRP (Sweere, G T H);
Structural Assessment, Performance and Economic Maintenance of Minor Roads (Duffell, J R);
Treatment of Bearing Capacity Results (Leben, B and Petkovsek, A);
Model of IRI for Jointed Plain Concrete Pavements (Poblete, M, Ceza, P, David, J, Gonzalez, J and Gutierrez, P);
High Speed Road Deflection Tester (Armburg, P W, Holen, AA and Magnusson, G);
PAVUE: A Real-Time Pavement Distress Analyzer (Burke, M W, Armburg, P W and Raahs, K).
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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Program I
List of participants XIII

LONG-TERM PAVEMENT PERFORMANCE

Early Evaluations of SHRP LTPP Data and Planning Sensitivity Analyses
J Brent Rauhut and Peter Jordahl, Brent Rauhut Engineering Inc, Michael I Darter, University of Illinois, Neil F Hawks, SHRP, Olga Pendleton, Tx Transp Inst and Emmanuel Owusu-Antwi, ERES Consultants Inc, USA

Specific Pavement Studies: Key Issues and Potential Products
Amir N Hanna and Neil F Hawks, Strategic Highway Research Program (SHRP), USA

Expected Changes to the AASHTO Design Guide
Neil F Hawks, Strategic Highway Research Program (SHRP), USA

Cost Effectiveness of Asphalt Concrete Overlays - The Canadian Approach
Gordon A Sparks, Clayton, Sparks & Ass Ltd, Canada, Dale Nesbitt, Decision Focus Incorporated, USA and Greg Williams, Canadian Strategic Highway Research Program, Canada

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

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

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<td>Michael W Burke and Knut Råhs, OPQ Systems AB, and Peter W Arnberg, Swedish Road and Traffic Research Institute (VTI), Sweden</td>
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WEDNESDAY SEPTEMBER 18

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
III

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 Chaussées, 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)
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, Two-Way Highway
A R Kaub, University of South Florida, Tampa, USA

Reducing Risk Taking in Passing on Two Way Roads
Krsto Lipovac, Higher School 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
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
Tapani Mäkinen

14.15 National reports on seat belt use and countermeasures (10 minutes each)
- Canada
  Brian A Grant
- Finland and other Nordic countries
  Juha Valtonen
- France
  Yves Page &
  Sylvain Lassarre
- Germany
  Hanns Ch Heinrich
- Great Britain
  Jeremy Broughton
- Netherlands
  Marjan Hagenzieker
- United States
  Robert Knaff

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)

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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, Savoy, 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 Reserach Institute (VTI), Sweden

VTI RAPPORT 372A
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 Håkan 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

VTI RAPPORT 372A
FRIDAY SEPTEMBER 20

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-Surfacings 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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Early Evaluations of SHRP LTPP Data and Planning for Sensitivity Analyses

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USA
CURRENT (LATE 1990 AND EARLY 1991) DATA ANALYSIS ACTIVITIES TO PRODUCE EARLY RESULTS FROM THE LTPP DATA INCLUDE: 1) STATISTICAL STUDIES TO IDENTIFY MISSING DATA AND TO DETERMINE DISTRIBUTIONS OF THE DATA ELEMENTS, 2) PRELIMINARY ASSESSMENTS OF RELATIVE SENSITIVITIES OF DISTRESS UNDER STUDY TO VARIATIONS IN MAGNITUDES FOR DATA ELEMENTS AVAILABLE IN THE DATA BASE, AND 3) DEVELOPMENT OF PROCEDURES FOR PRELIMINARY SENSITIVITY ANALYSES. THE PROPOSED PAPER WILL DESCRIBE THE RESULTS OF THESE EARLY ACTIVITIES, AS WELL AS THE EXPECTED PRODUCTS FROM THE DATA ANALYSIS AND THEIR POTENTIAL IMPACTS ON PAVEMENT DESIGN AND MANAGEMENT.

AS THE LTPP DATA BASE REPRESENTS A RESOURCE OF MAJOR IMPORTANCE TO THE INTERNATIONAL HIGHWAY COMMUNITY, ITS "COMPLETENESS" AND THE IDENTITIES OF MISSING DATA ELEMENTS WILL BE OF INTEREST. THE DISTRIBUTIONS OF THE DATA ELEMENTS ARE NOT ONLY OF IMPORTANCE TO THE EFFECTIVENESS OF THE ANALYSES, BUT OFFER USEFUL INFORMATION IN THEMSELVES.

THE ASSESSMENTS OF "RELATIVE SIGNIFICANCE" OF DATA ELEMENTS ARE BEING MADE BY SELECTED EXPERTS. THE ASSESSMENTS WILL BE IN TERMS OF THREE LEVELS OF SIGNIFICANCE - CLEARLY SIGNIFICANT, MODERATELY SIGNIFICANT, AND LITTLE OR NO SIGNIFICANCE. AS THE RELATIVE LEVELS ASSIGNED TO SPECIFIC DATA ELEMENTS WILL REPRESENT THE ACCUMULATED EXPERIENCE OF RECOGNIZED EXPERTS, THEY REPRESENT CURRENT KNOWLEDGE OF PAVEMENTS AND WILL BE OF INTEREST TO HIGHWAY PROFESSIONALS.

THE ANALYTICAL PROCEDURES UNDER DEVELOPMENT FOR THE SENSITIVITY ANALYSES WILL REPRESENT THE STATE-OF-THE-ART FOR SUCH STUDIES, SO OTHER RESEARCHERS WILL WISH TO KNOW DETAILS OF THE PROCEDURES TO BE APPLIED.

IDENTIFICATION OF WHAT MAY AND MAY NOT BE EXPECTED FROM THESE EARLY ANALYSES IS EXPECTED TO BE OF MAJOR INTEREST, AS WELL AS HOW THESE PRODUCTS MAY BE EXPECTED TO IMPACT HIGHWAY PRACTICE.
INTRODUCTION - CHAPTER 1

The Strategic Highway Research Program (SHRP) has contracted with Brent Rauhut Engineering Inc. (BRE) and ERES Consultants, Inc. (ERES), to conduct early analyses on the National Pavement Data Base (NPDB) from the Long Term Pavement Performance (LTPP) Studies. This work began in July 1990 and is scheduled to be completed in late 1992.

The tasks to be accomplished are:

Task 1 - Data Analysis Procedures and Workshop. This involved meeting with an Expert Task Group and SHRP personnel to develop procedures and a data analysis work plan (this task is complete).

Task 1A - Data Processing and Evaluation.

Task 2 - Sensitivity Analysis of Explanatory Variables in the National Pavement Performance Data Base.

Task 3 - Evaluation of the AASHTO Design Equations.

Task 4 - Improvement of the AASHTO Design Equations.

Task 5 - Evaluation and Improvement of AASHTO Overlay Design Procedures.

Task 6 - Future LTPP Data Analysis Plans.

The analyses are to be conducted individually for each of the following General Pavement Studies (GPS) experiments:

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>GPS-1</td>
<td>Asphalt Concrete Pavement with Granular Base</td>
</tr>
<tr>
<td>GPS-2</td>
<td>Asphalt Concrete Pavement with Bound Base</td>
</tr>
<tr>
<td>GPS-3</td>
<td>Jointed Plain Concrete Pavements (JPCP)</td>
</tr>
<tr>
<td>GPS-4</td>
<td>Jointed Reinforced Concrete Pavements (JRCP)</td>
</tr>
<tr>
<td>GPS-5</td>
<td>Continuously Reinforced Concrete Pavements (CRCP)</td>
</tr>
<tr>
<td>GPS-6</td>
<td>Asphalt Concrete Overlay of Asphalt Concrete</td>
</tr>
<tr>
<td>GPS-7</td>
<td>Asphalt Concrete Overlay of Concrete Pavements</td>
</tr>
<tr>
<td>GPS-9</td>
<td>Unbonded PCC Overlays of Concrete Pavements</td>
</tr>
</tbody>
</table>

This paper will only deal with Tasks 1A and 2. While the results from this research will be derived much later than the Gothenburg Conference, the nature of the data available, the results of relative sensitivity studies of the explanatory data elements, and the general procedures to be used in the sensitivity analyses is of interest to the international highway community. These are the subjects which will be addressed in this paper.

Calendar year 1991 is being spent in accumulating and evaluating the data as it becomes available, and then conducting preliminary sensitivity analyses to test out the procedures to be used and improve them in preparation for the actual sensitivity analyses to be conducted during the first six months of 1992. It is not possible to proceed with the sensitivity analyses before then, because much of the data will only be available toward the end of 1991. A "practice data base" for the GPS-1 experiment (A.C. over Unbound base) that includes both real data and physically realistic manufactured data has been developed and is being used for the preliminary sensitivity analyses. A similar practice data base is being developed for GPS-3 (Jointed Plain Concrete Pavements) for the same purpose. It is expected that the procedures for sensitivity analyses discussed in this paper will be modified somewhat as a result of these preliminary analyses.
This project involves the analysis of data observed on real in-service pavements, and none of the early results sought may be expected to exceed in quality the adequacy of the data base from which they are developed. Therefore, it is important to discuss the data resources available to the research team. There are certain limitations to the studies that are an unavoidable consequence of the timing of the early data analyses. For instance, very excellent traffic data will be available for future data analysts from the monitoring equipment just now being installed, while this early data analysis must rely on estimates of past Equivalent Single Axle Loads (ESAL) of very limited accuracy. While years of time-sequence monitoring data will be available later, these studies will only have distress measurements for one point in time, or at most two. For most distresses, an additional data point may be inferred for conditions just after construction; e.g., rutting, cracking, faulting of joints, etc., may generally be taken as zero initially. Analyses of distresses that deal in loss of performance (roughness and friction) will depend for many test sections on educated estimates for initial roughness and friction resistance.

The distribution in ages of the test sections will offer some assistance in overcoming the lack of time sequence data. Figure 2.1 shows the distribution of pavement ages for the GPS-1 experiment, Asphalt Concrete over Granular Base. A number of test sections are represented in all time intervals for the first 20 years.

Another characteristic of the data base that will greatly influence the success of this project will be missing items of inventory data, that data that concerns the design and construction of the pavements. Some data elements will be available for all of the test sections, while others for some test sections are not known and cannot be found.

As with any data analysis, the analysis staff must be concerned about potential biases in the data base. Several areas of concern are: 1) imbalances in the number of sections provided by different states, leading to possible undue influence from one state’s design, construction, and maintenance practices; 2) the possibility of systematic differences in the interpretation of SHRP guidelines for test section selection by the states and the four SHRP regional offices and their engineers; 3) the possibility that by allowing older non-overlaid pavements we are selecting the "survivors" which are not typical of pavements in general;
and 4) in a similar vein, the possibility that by basing much of our analysis on older pavements we may not be reflecting changes already made in modern construction and design practices.

Table 2.1 describes the data base that will be available. The various data elements are listed by source, and the inventory data elements have, in addition, been coded to indicate relative availability in terms of the percent of test sections for which the data are available. The great majority of the data, other than inventory data, will be available. The problems related to missing data, distributions of significant data elements, and potential problems of limited distress on pavements will be discussed subsequently. Those data elements printed in **bold type** are those tentatively selected as significant to performance predictions of at least one of the distresses of interest (selection of these data elements is discussed in Chapter 3).

### 2.1 Missing Data

The legend for Table 2.1 indicates that inventory data elements that are not available for more than 35% of the test sections have an asterisk "*" next to them. Data elements having values for 36 to 65% of the test sections are designated by a "#" sign. Those data elements that are available for more than 65% of the test sections are indicated with a circle "0". Those data elements that are common to both asphalt concrete and portland cement concrete pavements have two codes, with the left one representing the PCC and the right one the AC.

The data base was received in the form of ASCII files, and these were converted to Statistical Analysis System (SAS) files, using DBMSCOPY software. SAS procedure PROC UNIVARIATE was used to obtain statistical information on each variable, including percentage of missing data. The percentages were generally based on the numbers of layers involved, as many test sections included multiple layers of a particular material type. These percentages were obtained for each of the GPS experiments, but were combined for experiments with AC surfaces and for experiments with PCC surfaces to reduce the number of tables required. Similarly, the codes representing three ranges of data availability were applied to simplify communication to the reader.

Review of Table 2.1 indicates that many specific data elements are missing for a number of the test sections involved. This is primarily a consequence of the development of a broad data base that would include "bins" for storing any feasible data that might be used for a number of purposes, not just performance analysis. In fact, those data elements expected to be significant to performance prediction, except in a few cases, have a circle code indicating that those data elements are available for the majority of the test sections. The only data elements printed in **bold type**, to indicate that they are expected to be significant to performance prediction but coded to indicate data are available for less than 35% of the test sections, are: 1) Gradations for course and fine aggregates in PCC mixtures, 2) Initial values of skid number, and 3) Initial values of roughness. The SHRP Regional Offices are contacting the State Highway Agencies to find data on the aggregate gradations and to find roughness and friction information early in the pavements' lives that can be used to estimate initial values. As Serviceability Loss is the primary factor in the AASHTO design equations, it will be essential to develop reasonable estimates for initial PSI for evaluating the design equations.

The statistics indicated in Table 2.1 represent an analysis of the first data base release in January 1991. Since that time, software to operate on the data base and detect potential errors has been developed and applied, resulting in identification of many potential errors in the data to be evaluated. The four SHRP Regional Offices have updated their data files to correct as many problems in the data as possible. A new release of data is scheduled for early July and the expected result will be some reductions in missing and erroneous...
TABLE 2.1. SOURCES OF GPS DATA ELEMENTS

1. Distress and Performance Measurements:

**Flexible Pavements (With or Without Overlays):**

- Alligator Cracking
- Longitudinal Cracking
- Rut Depth
- Raveling and Weathering
- Lane to Shoulder Separation and Dropoff
- Edge Cracking
- Polished Aggregate
- Transverse Cracking
- Block Cracking
- Profile in Both Wheel Paths

**International Roughness Index**

**Skid Resistance** (To Monitor)

**Reductions**

**Bleeding (Flushing)**

**Water Bleeding and Pumping**

**Reflection Cracking at Joints**

**Patch/Patch Deterioration**

**Lane to Shoulder Dropoff**

**Potholes**

**Shoving**

**Rigid Pavements (With or Without PCC Overlays):**

- Transverse Cracking
- Longitudinal Cracking
- Durability "D" Cracking
- Joint and Crack Faulting
- Water Bleeding and Pumping
- Lane/Shoulder Separation and Dropoff
- Joint Seal Damage
- Reactive Aggregate Distress
- Construction Joint Deterioration
- Profile in Both Wheel Paths

**Skid Resistance** (To Monitor)

**Reductions**

**Blow-ups**

**Spalling**

**Scaling and Map Cracking**

**Punchouts**

**Patch/Patch Deterioration**

**Corner Breaks**

**Polished Aggregate**

**Popouts**

**International Roughness Index**

**Rigid Pavements With Flexible Overlays (Additional):**

- Reflection Cracking
- Rut Depth

**Pot Holes in Overlays**

**Raveling and Weathering**

2. Deflection Testing Results:

- Type of Deflection Device
- Locations of Sensors
- Applied Load and Frequency
- Measured Deflections Under Load (Each Sensor)

**Locations of Deflection Measurements**

**Temperature Profile of the Pavement at Time of Measurements**

**Joint Efficiency**

**Load Transfer at Cracks**

3. Traffic Measurements:

- Estimated Traffic Data Prior to Monitoring:
  - Vehicle Classifications/Volumes
  - Weights of Individual Axles
  - Axe Spacing
  - Gross Weight
  - Cumulative ESAL's

**Monitoring Data:**

**Vehicle Classifications/Volumes**

**Weights of Individual Axles**

**Axe Spacing**

**Gross Weight**

**Cumulative ESAL's**
4. Material Sampling And Laboratory Testing:

<table>
<thead>
<tr>
<th>Layer Thicknesses</th>
<th>Optimum Lab Moisture Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer Stiffnesses</td>
<td>Percent Compaction</td>
</tr>
<tr>
<td>Gradation</td>
<td>Aggregate Gradation</td>
</tr>
<tr>
<td>Bulk Spec. Gravities</td>
<td>In Situ Moisture Content</td>
</tr>
<tr>
<td>Geological Class of Coarse Aggregate</td>
<td>In Situ Dry Density</td>
</tr>
<tr>
<td>Asphalt Content for A.C. Mixtures</td>
<td>% of Subgrade Soil By Wt.</td>
</tr>
<tr>
<td>Max. Spec. Gravities for A.C. Mixtures</td>
<td>Passing #200</td>
</tr>
<tr>
<td>Bulk Spec. Gravities for A.C. Mixtures</td>
<td>% of Subgrade Soil By Wt. Finer</td>
</tr>
<tr>
<td>Percent Air Voids</td>
<td>Than 0.02 mm</td>
</tr>
<tr>
<td>Unbound Base/Subbase/Subgrade</td>
<td>Indirect Tensile Strength of A.C.</td>
</tr>
<tr>
<td>Material Properties:</td>
<td>and PCC layers</td>
</tr>
<tr>
<td>AASHTO Soil Classification</td>
<td>Type of Stab. Agent for Bound</td>
</tr>
<tr>
<td>Atterberg Limits</td>
<td>Base/Subbase Mats.</td>
</tr>
<tr>
<td>Max. Lab Dry Density</td>
<td>Depth to Rigid Layer</td>
</tr>
</tbody>
</table>

5. Climatic Data Base By Month (Includes Standard Deviation, Skewness, and Kurtosis):

| Mean Daily Temperature | Total Snowfall |
| Mean Minimum Daily Temperature | Mean Daily Percent Sunshine |
| Mean Maximum Daily Temperature | Mean Daily Sky Coverage |
| Mean Wind Velocity (mph) | Mean Daily Sky Coverage (Sunrise to Sunset) |
| Mean Gust Wind Speed | Mean Daily Sky Coverage (Midnight to Midnight) |
| Air Freeze-Thaw Cycles | Mean Daily Maximum |
| Annual Precipitation | Relative Humidity |
| Mean Daily Precipitation | Mean Daily Min. Relative Humidity |
| Total Precipitation (In. of Water) | No. of Days with Max. Temperature > 90F |
| Number of Wet Days (above 0.01") | No. of Days with Min. Temperature < 30F |
| High-Intensity Precipitation | Freeze Index |
| Occurrences (>0.5"/day) | |
| Mean Daily Snowfall | |
| Thornthwaite Index | |

6. Inventory Data:

<table>
<thead>
<tr>
<th>PCC Mixture Data:</th>
<th>Longitudinal Joint Data:</th>
</tr>
</thead>
<tbody>
<tr>
<td>o Mix Design</td>
<td>o Type</td>
</tr>
<tr>
<td>o Slump</td>
<td># Width (As Built)</td>
</tr>
<tr>
<td>o Type of Cement</td>
<td># Depth of Reservoir (As-Built)</td>
</tr>
<tr>
<td># Alkali Content of Cement</td>
<td># Diameter of Tie Bars</td>
</tr>
<tr>
<td>o Entrained Air</td>
<td># Length of Tie Bars</td>
</tr>
<tr>
<td>o Admixture Type</td>
<td># Spacing of Tie Bars</td>
</tr>
<tr>
<td>o Admixture Amount</td>
<td>o Shoulder - Traffic Lane Joint:</td>
</tr>
<tr>
<td>o Type of Coarse Aggregate</td>
<td>Type</td>
</tr>
<tr>
<td>o Geological Class. of Coarse Aggregate</td>
<td>Reinforcing Steel Data:</td>
</tr>
<tr>
<td>* Gradations - Coarse and Fine Aggregates</td>
<td>o Type of Reinforcing</td>
</tr>
<tr>
<td>o Type of Fine Aggregate</td>
<td>o Transverse Bar Diameter</td>
</tr>
<tr>
<td># Bulk Spec. Gravities - Coarse and Fine Aggregate</td>
<td>o Longitudinal Bar Diameter</td>
</tr>
<tr>
<td>o Durability of Coarse Aggregate</td>
<td>o Longitudinal Bar Spacing</td>
</tr>
<tr>
<td>o Type of Paver Used</td>
<td># Depth to Re-Bars from Slab</td>
</tr>
<tr>
<td>o Method Used To Cure Concrete</td>
<td>Surface</td>
</tr>
<tr>
<td>o Method used To Finish Concrete</td>
<td># Method Used to Place Rebar</td>
</tr>
<tr>
<td># Flexural Strength</td>
<td># Design Percentage of Steel</td>
</tr>
<tr>
<td># Compressive Strength</td>
<td># Length of Steel Lap at Constr. Jt.</td>
</tr>
</tbody>
</table>
TABLE 2.1. SOURCES OF GPS DATA ELEMENTS (Cont.)

<table>
<thead>
<tr>
<th>Table entries</th>
<th>Data Element Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Joint Data for PCC Layers:</td>
<td>Tie Bars (For PCC Shoulder):</td>
</tr>
<tr>
<td>o Average Contraction Jt. Spacing</td>
<td>* Diameter</td>
</tr>
<tr>
<td>* Built-In Expansion Jt. Spacing</td>
<td>* Length</td>
</tr>
<tr>
<td>* Average Intermediate Sawed Jt. Spacing</td>
<td>* Spacing</td>
</tr>
<tr>
<td># Skewness of Joint</td>
<td>* Width (As Built)</td>
</tr>
<tr>
<td>o Type of Load Transfer System</td>
<td>HMAC Construction Information:</td>
</tr>
<tr>
<td># Dowel Diameter</td>
<td>o Mean Mixing Temperature</td>
</tr>
<tr>
<td># Dowel Spacing</td>
<td>o Mean Laydown Temperature</td>
</tr>
<tr>
<td># Dowel Length</td>
<td>Base/Subbase Material Data:</td>
</tr>
<tr>
<td># Method Used to Install Dowels</td>
<td>* CBR</td>
</tr>
<tr>
<td>o Jt. Sealant Type</td>
<td>* R-Value</td>
</tr>
<tr>
<td>o Jt. Sealant Width</td>
<td>o Type of Stabilizing Agent</td>
</tr>
<tr>
<td>o Jt. Sealant Reservoir Depth</td>
<td>o Percent Stabilizing Agent</td>
</tr>
<tr>
<td>o Method Used to Form Jts.</td>
<td>o Type of Admixture</td>
</tr>
<tr>
<td>* Initial Skid Number</td>
<td>o Percent of Admixture</td>
</tr>
<tr>
<td>* Initial Roughness</td>
<td>* Compressive Strength</td>
</tr>
<tr>
<td>o o Age of Pavement</td>
<td>* Calcium Carbonate Content</td>
</tr>
<tr>
<td>o # Age of Overlay</td>
<td>* Percent Compaction</td>
</tr>
<tr>
<td>Plant Mix HMAC Aggregate Properties:</td>
<td>Subgrade Data:</td>
</tr>
<tr>
<td>* Composition of Coarse</td>
<td>* CBR</td>
</tr>
<tr>
<td>* Composition of Fines</td>
<td>* R-Value</td>
</tr>
<tr>
<td>* Type of Mineral Filler</td>
<td>** Relative Density - Cohesionless</td>
</tr>
<tr>
<td># Durability</td>
<td>Soil</td>
</tr>
<tr>
<td>* Polish Value for Coarse</td>
<td>* Soil Suction</td>
</tr>
<tr>
<td># Effective Spec. Gravity</td>
<td>* Expansion Test Results</td>
</tr>
<tr>
<td>Original Asphalt Cement Properties:</td>
<td>* Swell Pressure</td>
</tr>
<tr>
<td>o Viscosity (or penetration or grade)</td>
<td>* Avg. Rate of Heave (Frost)</td>
</tr>
<tr>
<td>* Source</td>
<td>** Frost Susceptibility Class.</td>
</tr>
<tr>
<td>* Specific Gravity</td>
<td>Shoulder Data:</td>
</tr>
<tr>
<td>* Type of Modifiers</td>
<td>o o Width</td>
</tr>
<tr>
<td>* Quantity of Modifiers</td>
<td>o Surface Type</td>
</tr>
<tr>
<td>* Ductility</td>
<td>o Surface Thickness</td>
</tr>
<tr>
<td>* Ring and Ball Softening Point</td>
<td>o Base Type</td>
</tr>
<tr>
<td>Lab-Aged Asphalt Cement Properties:</td>
<td>o Base Thickness</td>
</tr>
<tr>
<td>* Viscosity at 140F</td>
<td>Subsurface Drainage:</td>
</tr>
<tr>
<td>* Ductility at 77F</td>
<td>o o Type (If Any)</td>
</tr>
<tr>
<td>* Penetration at 77F</td>
<td>* Location</td>
</tr>
<tr>
<td>* Ring and Ball Softening Point</td>
<td>* Diameter of Longitudinal Pipe</td>
</tr>
<tr>
<td>* Weight Loss</td>
<td>o Spacing of Laterals</td>
</tr>
<tr>
<td>Original A.C. Mixture Properties:</td>
<td>o o Number of Lanes in Travel</td>
</tr>
<tr>
<td># Bulk Spec. Gravity</td>
<td>Direction</td>
</tr>
<tr>
<td>o Percent Air Voids</td>
<td>Note: Where a data element</td>
</tr>
<tr>
<td>* Marshall Stability, Lbs.</td>
<td>is common for AC and PCC</td>
</tr>
<tr>
<td>* Marshall Stability, Blows</td>
<td>pavements, the PCC code</td>
</tr>
<tr>
<td>* Marshall Stability, Flow</td>
<td>appears on the left and</td>
</tr>
<tr>
<td>* Hveem Cohesiometer Value</td>
<td>the AC code on the right</td>
</tr>
<tr>
<td>o Type of Plant</td>
<td>next to the data element.</td>
</tr>
<tr>
<td>* Moisture Susceptibility</td>
<td></td>
</tr>
<tr>
<td>* Type of Antistripping Agent</td>
<td></td>
</tr>
<tr>
<td>* Amount of Antistripping Agent</td>
<td></td>
</tr>
</tbody>
</table>

Legend:
* = Have Data for 0 to 35% of Test Sections
# = Have Data for 36 to 65% of Test Sections
o = Have Data for More Than 65% of Test Sections
data. The SAS procedures will be used to operate on the data base once more to update our understanding of the data base.

Procedures for dealing with missing data are being finalized. Reasonable estimates may be included for some data elements without seriously affecting the results, while others will have to just be treated as missing data. Where a number of significant data elements are missing for a particular test section, that test section may be omitted from the analysis. Where only one or two data elements are missing, such test sections will likely be included, depending upon the specific analytical activity.

2.2 Distributions of Significant Data Elements

The primary objective of the factorial designs of the GPS experiments was to encourage reasonable distributions for those data elements believed to be most significant to the performance of pavements. Therefore, another important step in the data base evaluation is to study the distributions of the variables for the test sections selected for study. While values for many of the data elements selected for the sensitivity analyses were not yet available when this paper was written (June 1991), some are available and a few are presented.

Figure 2.1, referred to previously, indicates that a very favorable distribution of pavement ages has been achieved, although pavement age was treated as a co-variable in the experiment designs. Figure 2.2 provides the distribution for air voids in AC pavement layers from the inventory data for the GPS-1 experiment. This distribution appears to be reasonably representative of practice in North America. Additional air void data will accrue later from the laboratory testing of the materials sampled near each end of the GPS test sections. Figure 2.3 shows similar distribution data on the combined thickness of base and subbase for the GPS-1 test sections.

While much of the data is still to be collected and entered, the distributions evaluated so far appear to be reasonable.

2.3 Potential Problems From Limited Distress in Pavements

The reduction of data from the first round of photographic distress surveys had produced very little distress data for the analysis team at the time that this paper was written. However, it is clear that some of the distresses to be studied will not have occurred on large numbers of the test sections in the experiments. This is a consequence of the distributions in pavement ages and the fact that this is a long-term experiment. This will not be a serious problem for future analysts as more and more test sections will experience the distresses of interest in time, but does pose an immediate problem.

The analytical team has not as yet decided how this problem is to be dealt with, and in fact cannot until we get the majority of the distress data. A major concern is the possibility that in some cases so few test sections will have experienced a distress that the number of observations will be too few to directly support meaningful prediction equations. A partial solution to this problem, should it arise, will be the clustering (or combining) of independent variables into physically logical single independent variables, which is planned for all of the model development. While the primary purpose for clustering variables is to integrate current knowledge into the analyses, it will also reduce the numbers of independent variables in the regression. Theoretical mechanics, dimensional analysis, and empirical data from other studies will be considered in the formation of the clustered variables.

If there are sufficient test sections experiencing a distress, models may be developed from the set of data for these test sections. These models can then be applied to test sections not experiencing the distress to see if zero distress is predicted.
Figure 2.2. Distribution of Percent Air Voids in AC mixtures, Experiment GPS-1, AC over Granular Base

Figure 2.3. Distribution of Combined Base and Subbase Thickness, Experiment GPS-1, AC over Granular Base
This problem will be addressed during the preliminary sensitivity analyses so that appropriate procedures will be available when the real sensitivity analyses occur in 1992.

2.4 Biases

Potential approaches for alleviating problems with biases such as those discussed above are:

1. Limit the inference space of a model where the data are limited or grossly unbalanced. Consider regional models where the data do not warrant a national model.

2. Combine experiments (where distress mechanisms may be similar) to achieve better balance (specifically Experiments GPS-1 and GPS-2).

3. Review regional operational practices to identify possible sources of selection or testing bias.

4. Examine the distributions of independent and dependent variables for non-normality, bi-modalism, and extreme values; where such are found, attempt to determine their source.

5. Conduct a thorough residual examination as soon as preliminary models are available, comparing residuals to project age, state, season tested, and others to determine possible sources of bias.

Where it is not possible to overcome the effects of bias, or bias is suspected but cannot be determined, appropriate discussion will be provided in reports to identify potential limitations in the analytical results.

3. PRELIMINARY SELECTION OF DATA ELEMENTS FOR SENSITIVITY ANALYSES - CHAPTER 3

The National Information Management System (NIMS) has "bins" for 117 data elements for pavements with asphalt concrete surfaces, 128 data elements for jointed concrete pavements, and 120 data elements for continuously reinforced concrete pavements. As it clearly would not be practical to attempt to model pavement performance with so many independent variables and literally hundreds of potential interactions, it is necessary to considerably reduce the number of variables (data elements) in order to develop meaningful performance prediction equations and reasonable estimates of relative significance of the independent variables to occurrence of specific distress (dependent variables). The approach adopted for preliminary elimination of less significant variables was to obtain relative significance rankings from experts in pavement performance modeling. This offered a means for bringing expert knowledge into the analysis at an early stage, as well as offering insight for use in selecting the variables to be considered in the analyses. These selections required balancing relative significance, data availability, and correlations with other variables. Tables were developed for the three pavement types that listed the data elements as rows and the significant distresses selected for study as columns. These significance tables were distributed to various experts who had agreed to participate.

3.1 Relative Significance Studies

Three levels of significance were considered. Assignment of the number "1" indicated that the rater considered the data element to be clearly significant to prediction of the distress of interest. Assignment of the number "2" indicated moderate significance, and the number "3" indicated little or no significance. Space was also included on the tables for listing other
data elements that were believed to be correlated with the one identified on that line. The layout for entry of the significance levels was very similar to Tables 3.1 and 3.2, except that the tables in this paper include only those data elements that were rated as significant.

The experts filling out the significance rating forms would enter either a 1, 2, or 3 in each block representing a data element and a distress. When the significance rating forms were returned, the entries were averaged for each block to arrive at average rating. If the average score for a data element and distress combination was less than 2, that data element was considered to be significant for prediction of that distress. If the average score was exactly 2, these were retained in the significance studies in some cases and not in others on the basis of judgement. Data elements with scores greater than 2 were not considered further at this time, but those that were considered to be very significant by at least one of the raters (Entered a "#") have been identified by entering a "#" symbol in Table 3.1 for further consideration in future analyses when more data and time are available. For Table 3.2, a "#" symbol has been entered for JCP and a "+" symbol for CRCP. This process considerably reduced the numbers of data elements to be dealt with in the sensitivity analyses.

3.2 Criteria for Selection of Data Elements

Upon completion of the relative significance studies discussed above, sets of independent variables had been developed that were believed individually to be significant to the prediction of specific distresses, but significant independent variables to be included in the studies must also be available in the data base. Therefore, the percentages of data availability referred to in the previous chapter had to be considered in selection of the data elements to be included in the sensitivity analyses.

It was soon apparent that many of the data elements considered to be individually significant were not going to be available in sufficient numbers to support the analyses. However, a great many of these variables were correlated to various degrees with other independent variables that were represented in greater percentages of the test sections involved. These correlations were considered and it became apparent that the "explanation" of variations in the distresses (dependent variables) could for the most part be offered by other data elements with which they were correlated. That is, the growth in the error pool through omitting many of the variables would be manageable because of the inclusion of other correlated variables. In one example, the contribution of subgrade CBR to predicting performance is adequately "explained" by subgrade stiffness, AASHTO soil classification, and insitu moisture content. Similarly, Marshall Stability will be adequately represented by asphalt concrete stiffness, asphalt content, and percent voids.

The expected degrees of correlation were considered and in general appeared to be adequate. However, there remained a few of the significant data elements that were not replaceable with other correlated data elements. The level of the effect on the results was evaluated, as well as the probability of finding values for them, and this resulted in a very small group of data elements for which the SHRP Regional Offices are seeking values as discussed in the previous chapter.

As an example, the data base includes "bins" for grade, penetration, and viscosity of the original asphalt cement for flexible pavements. As this data cannot be obtained by testing the hardened asphalt taken from the inservice pavements, there is no source other than the inventory data furnished from files in the offices of the State and Provincial Highway Agencies. It was concluded that approximate values of the other two could be obtained if any one of the three was known. Therefore, values are being sought in the few cases where none of the three values were furnished. Another example is percent compaction of asphalt concrete, which is considered to be adequately explained by percent air voids and asphalt concrete stiffness. Similarly, CBR or R-value for base and subbase would be represented by stiffnesses, gradations, and percent compaction.
## TABLE 3.1 SIGNIFICANT DATA ELEMENTS FOR PREDICTING DISTRESSES IN PAVEMENTS WITH ASPHALT CONCRETE SURFACES

<table>
<thead>
<tr>
<th>Significant Data Elements</th>
<th>Distress Types</th>
<th>Alligator Cracking</th>
<th>Transverse Cracking</th>
<th>Rutting</th>
<th>Roughness</th>
<th>Friction Loss</th>
<th>Raveling/Weathering</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Thickness</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base/Subbase Thickness</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Surface Stiffness</td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unbound Base/Subbase Stiffness</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bound Base/Subbase Stiffness</td>
<td></td>
<td>X</td>
<td>#</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Subgrade Stiffness</td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Age of Pavement</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Cumulative ESAL's</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
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<tr>
<td>Asphalt Viscosity</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>#</td>
<td></td>
<td></td>
</tr>
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<td>Asphalt Content</td>
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<td>X</td>
<td>X</td>
<td>#</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Percent Air Voids</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>#</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>HMAC Aggregate Gradation</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>#</td>
<td>#</td>
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<tr>
<td>Percent Compaction of Base/Subbase</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Subgrade Soil Classification</td>
<td></td>
<td>#</td>
<td></td>
<td></td>
<td>X</td>
<td>#</td>
<td></td>
</tr>
<tr>
<td>In Situ Moisture Content of Subgrade</td>
<td></td>
<td></td>
<td>#</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Subsurface Drainage Yes/No</td>
<td></td>
<td>#</td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
<td></td>
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<td>Geological Classification of Course Aggregate in HMAC</td>
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<td></td>
<td></td>
<td></td>
<td></td>
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<td>Significant Data Elements</td>
<td>Distress Types</td>
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<td></td>
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<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Alligator Cracking</td>
<td>Transverse Cracking</td>
<td>Rutting</td>
<td>Roughness</td>
<td>Friction Loss</td>
<td>Raveling/Weathering</td>
<td></td>
</tr>
<tr>
<td>% of Subgrade Soil Passing #200 Sieve</td>
<td>#</td>
<td>#</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plasticity Index of Subgrade Soil</td>
<td>#</td>
<td>#</td>
<td>X</td>
<td></td>
<td></td>
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<td>Liquid Limit</td>
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<td>#</td>
<td>#</td>
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<tr>
<td>% of Subgrade Soil Finer Than 0.02 mm</td>
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<td>#</td>
<td>X</td>
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<td>Type of Environment</td>
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<td>#</td>
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<td>X</td>
<td>X</td>
<td>X</td>
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</tr>
<tr>
<td>Average Max. Daily Temp. by Month</td>
<td>#</td>
<td>#</td>
<td>X</td>
<td>#</td>
<td>#</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average Min. Daily Temp. by Month</td>
<td>#</td>
<td>#</td>
<td>#</td>
<td>#</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Thornthwaite Index</td>
<td>#</td>
<td>#</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Freeze Index</td>
<td>#</td>
<td>X</td>
<td>#</td>
<td>#</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>No of Days Min. Temp. &lt;30F</td>
<td>X</td>
<td>X</td>
<td>#</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
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<tr>
<td>No. of Days Max. Temp. &gt; 90F</td>
<td>X</td>
<td>#</td>
<td>#</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of Air Freeze-Thaw Cycles</td>
<td>X</td>
<td>X</td>
<td>#</td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Annual Precipitation</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
</tr>
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</table>

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<table>
<thead>
<tr>
<th>Significant Data Elements</th>
<th>Distress Types</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Transverse Cracking</td>
</tr>
<tr>
<td>PCC Surface Thickness</td>
<td>X</td>
</tr>
<tr>
<td>Base Thickness</td>
<td>#</td>
</tr>
<tr>
<td>PCC Surface Stiffness</td>
<td>X</td>
</tr>
<tr>
<td>Base Stiffness</td>
<td>X</td>
</tr>
<tr>
<td>Subgrade Stiffness</td>
<td>X</td>
</tr>
<tr>
<td>Age of Pavement</td>
<td>X</td>
</tr>
<tr>
<td>Cumulative 18-kip ESAL</td>
<td>X</td>
</tr>
<tr>
<td>Type of Coarse Aggr. for PCC</td>
<td>X</td>
</tr>
<tr>
<td>Gradation of Coarse Aggr. for PCC</td>
<td>#</td>
</tr>
<tr>
<td>PCC Compr. Strength</td>
<td>X</td>
</tr>
<tr>
<td>AASHTO Soil Class Base/Subbase</td>
<td></td>
</tr>
<tr>
<td>% Compact. of Base/Subbase</td>
<td>#</td>
</tr>
<tr>
<td>Coarse Aggregate Gradation of Base/Subbase</td>
<td>#</td>
</tr>
<tr>
<td>Fine Aggr. Gradation of Base/Subbase</td>
<td>#</td>
</tr>
</tbody>
</table>
### TABLE 3.2
SIGNIFICANT DATA ELEMENTS FOR PREDICTING DISTRESSES
IN PAVEMENTS WITH PORTLAND CEMENT CONCRETE
SURFACES AVAILABLE FOR ANALYSIS CONT'D.

<table>
<thead>
<tr>
<th>Significant Data Elements</th>
<th>Distress Types</th>
<th></th>
<th></th>
<th></th>
<th>Friction Loss</th>
<th>Joint Faulting</th>
<th>Joint/ Crack Spalling</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Transverse Cracking</td>
<td>Longitudinal Cracking (JCP)/ Localized Failures (CRCP)</td>
<td>Pumping</td>
<td>Roughness</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AASHTO Soil Classification of Subgrade</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Subgrade % Passing #200 Sieve</td>
<td></td>
<td>+</td>
<td>X</td>
<td>#</td>
<td>X</td>
<td>#</td>
<td>+</td>
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<tr>
<td>Moisture Content of Subgrade</td>
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<td>#</td>
<td>X</td>
<td>#</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Joint Efficiency</td>
<td></td>
<td>#</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Thornthwaite Index</td>
<td>#</td>
<td>#</td>
<td>X</td>
<td>X</td>
<td>#</td>
<td>X</td>
<td>#</td>
</tr>
<tr>
<td>Annual Precipitation</td>
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<td>X</td>
<td>#</td>
<td></td>
<td>X</td>
<td>X</td>
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<tr>
<td>Precipitation Days by Year</td>
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<td>X</td>
<td>#</td>
<td></td>
<td>X</td>
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<tr>
<td>Shoulder Type</td>
<td>X</td>
<td>#</td>
<td>#</td>
<td>#</td>
<td></td>
<td></td>
<td>+</td>
</tr>
<tr>
<td>Subsurface Drainage Type</td>
<td>X</td>
<td>#</td>
<td>#</td>
<td>#</td>
<td></td>
<td></td>
<td>+</td>
</tr>
<tr>
<td>Avg. Max. Daily Temp. by Month</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Avg. Min. Daily Temp. by Month</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>O</td>
</tr>
<tr>
<td>No. of Days Min. Temp. &lt; 32F</td>
<td>X</td>
<td>X</td>
<td>O</td>
<td>O</td>
<td>O</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. of Days Min. Temp. &gt; 90F</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Air Freeze-Thaw Cycles</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

Legend: *"X" for JCP and "O" for CRCP
"#" for JCP and "+" for CRCP
In summary, the criteria for selecting the data elements to be included in the sensitivity analyses were: 1) The data element must have been rated by a group of experts as significant, 2) the data must be available for sufficient numbers of the test sections to be utilized, and 3) the data element should not be highly correlated to other data elements considered to be more significant or having data for more test sections.

3.3 Data Elements Selected for Sensitivity Analyses

The data elements that survived the preliminary selection process described above are listed in Tables 3.1 and 3.2. For Table 3.1, an "X" in a box representing a specific data element and specific distress indicates that that data element will be included in the development of predictive equations for the specific distress, and will be further considered in the sensitivity analyses if the statistical studies support the opinions of the experts as to its importance.

Table 3.2 provides combined information for both jointed concrete and continuously reinforced concrete pavements. An "X" in a box indicates selection for a JCP distress, whereas an "O" in a box indicates selection for the CRCP studies. As an example, PCC surface thickness is considered to be significant for transverse cracking, longitudinal cracking, pumping, roughness, and joint faulting for jointed concrete pavements, but is only considered significant for localized failures, pumping, and roughness for continuously reinforced concrete pavements.

It can be readily seen that a number of data elements are available for alligator cracking, transverse cracking, rutting, roughness, and raveling/weathering of pavements with AC surfaces. However, only four data elements will be available for studies of friction loss. Unfortunately, there is little data on polishing or durability of the course aggregates for use in the equations and sensitivity analyses. For pavements with portland cement concrete surfaces, only three data elements will be available for friction loss.

There are no surprises in the data elements selected as significant. Thicknesses and stiffnesses of layers control strains in the pavement structure, while other data elements reflecting material properties (e.g., asphalt viscosity, percent air voids, gradations of aggregates and base materials, strengths, etc.) affect layer stiffnesses and durability while experiencing the impacts of loads and the environment. Plasticity indices of the subgrades affect roughness through differential volume change, interacting with moisture content (or perhaps percent saturation would be better). Drainage can affect moisture contents in base, subbase, and subgrade, which in turn affects layer stiffnesses and loss of fines. Performance of jointed concrete pavements depends heavily on "joint efficiency" from deflection measurements, which indicate movements in joints under loads.

Large sets of deflection measurements are available for each test section, but these are not going to be included in the sensitivity analyses. It is quite true that it is perfectly logical to include deflections in a predictive equation to be used for overlay design or other purposes, but it is not appropriate to include them into models built for sensitivity analyses, as the responses to load are already "explained" by other data elements representing the pavement structure. To include the deflection responses would be to account twice for the same effects. Consequently, deflections will not be considered directly in the sensitivity analyses, except to the extent that layer moduli are calculated from deflection data.

The results of the significance ratings and studies described above are the selections of the data elements (or independent variables) that will be included in the sensitivity analyses for each of the distress types. It should be noted that separate analyses will be conducted for each of the distress types that appear in Table 3.1 and 3.2 and for each of the applicable GPS experiments. Those in Table 3.2 will be individually studied, as appropriate, for GPS-3, GPS-4, GPS-5, and GPS-9. For those experiments with overlays, data will need to be utilized for both the original surface layer and the new surface layer (overlay).
4. PRELIMINARY PROCEDURES FOR SENSITIVITY ANALYSES -
    CHAPTER 4

The objective of the sensitivity analyses is to identify those independent variables that have
significant impact on pavement performance and to quantify that impact. The procedure
selected requires the development of multiple regression equations which, for each
dependent variable (distress), relate the various independent variables to that distress, and
examining the resulting coefficients of those models. Those coefficients, in equations
derived using "standardized" variables (variables transformed to deviations from the mean
in units of the standard deviation) indicate the relative effect of changing a specific
independent variable on the dependent variable. Several statistical procedures from the
SAS (Statistical Analysis System) library will be used for developing the mathematical
models.

While the primary objective of the sensitivity analyses is to establish the relative significance
of the explanatory variables to the performance of the pavements, the necessity to develop
multiple regression equations is fortuitous because these equations will have value of their
own. Distress specific prediction equations are critically needed for pavement management
systems and for use in design. These equations can serve as design checks in future
editions of state and national design procedures, and represent the beginning of a desirable
initiative to design pavements to resist all common forms of distress in the future; and in
the nearer term, to analyze how design or materials changes will affect distress accumula-
tion rates. A distress specific rutting model can be used to check if efforts to control low
temperature cracking, for instance, will cause an acceleration of rutting.

4.1 Analytical Limitations Reflecting Data Limitations

Several significant problems exist in the data and its use in such analyses. These include
incomplete data and possible large errors in traffic estimates (cumulative ESALs) for this
initial analysis (discussed above), collinearity between different independent variables, and
the fact that several of the dependent variables are not measured as one quantity but as
two (severity and extent).

Collinearity is one of the most serious problems faced by the research team. This refers
to a situation where two independent variables are highly correlated, such as (for a specific
highway section) age and total traffic since opening. Because of this correlation, the
coefficients on these variables may depend on the order in which they are entered into the
regression equation. Special techniques will be used to detect these collinearities, and also
to study the effects of outliers which may mask the collinearities.

Multivariate analysis has been suggested to obtain the effects of one distress on the
development and growth of another. This is a very important area of research, but the data
available to the study will most likely not support such analyses, since very little if any "time
series" data permitting correlation of the development of different distresses will be
available.

Several distresses to be analyzed are measured in terms of two variables, severity of the
distress, and extent (e.g., area) of that severity level. The severity is usually a categorical
variable (low, medium, or high) while the extent is a continuous variable. Several
procedures have been suggested for performing the required analyses on this type of
distress variable, but the one tentatively adopted is to combine the extent of the distress
at medium and high severity levels, and to use that combined extent as the dependent
variable. This decision was based on the fact that normally maintenance and rehabilitation
decisions are based on these higher-severity levels rather than on the extent of low-severity
distress.
Another related problem (discussed in Chapter 2) is the fact that many highway sections will show little or no distress, or only low-severity distress of the particular type under study. Very serious thought must be given to selection of a method to properly and consistently include the fact that a test section having certain properties does not show distress at specific times during the period of analysis.

4.2 Preliminary (Pilot) Analyses

Since there will be a considerable period of time before data is available to perform these studies on all experiments, sections, and distresses, pilot studies will be implemented on a small scale for one or two distresses each for rigid and flexible pavements, using available data and, where necessary, data from other sources or manufactured data, to refine the statistical procedures to be used on the whole data set, as well as to gain insight into the distresses to be studied. These analyses will be carried out in as much detail as time and the data quality permit, including studies of relevant data transformations and combinations, and will furnish the data analysts an opportunity to iteratively optimize the techniques and procedures to be applied. The basic idea behind the preliminary or pilot studies is that when the data to be analyzed become available (late in 1991) the data manipulation and statistical procedures will have been honed so that analysis can proceed quickly and efficiently.

4.3 Tentative General Procedure for Sensitivity Analyses

Although subject to change as the data is evaluated and pilot analyses conducted, the general procedures (or steps) planned are as follows:

1. Select candidate equation forms and independent variable "clusters" for multiple regressions.
   - Use theory knowledge, logic, and dimensional analysis to arrange some data into logical "clustered variables" to simplify regressions and to strengthen and simplify equations.
   - Apply engineering knowledge and experience to develop an array of equation forms for each distress/pavement type for regression trials.

2. Develop distress models for sensitivity analyses.
   - Apply procedures previously "debugged" during the pilot sensitivity analyses.
   - Develop multiple linear regression equations using SAS PROC REG (and SAS PROC RSQUARE).
   - Study of residuals versus independent variables to seek transformations or improvements in equation forms to try. Analyze residuals to detect outlier data points.
   - Conduct regressions on revised equation forms excluding outliers.
   - Select best equations and use principal component procedure to examine for collinearity and potential hidden outliers.
   - Identify variables that yield best predictive models using PROC RSQUARE.
   - Select distress models for sensitivity analyses.
3. Conduct sensitivity analyses.
   - Apply procedures previously "debugged" during the pilot sensitivity analyses.
   - "Standardize" variables to be considered.
     - Calculate the deviation of each value in a data set from the mean for that data set.
     - Divide the deviation by the standard deviations.
   - Regress the equations again to obtain new coefficients for standardized equations.
   - The magnitudes of the resulting adjusted coefficients may be interpreted as the relative "strength" of that variable for predicting the distress of interest.
   - Tabulate and/or graph sensitivity analysis results.
   - Prepare summaries of results and conclusions.

4.4 Mechanistic Considerations

Dr. Gilbert Baladi at Michigan State University is also conducting studies on flexible pavements using his mechanistic-empirical model MICHPAVE. BRE will furnish available data on flexible pavements, so that relevant mechanistic responses can be computed with this state-of-the-art finite element program. These responses (strains, stresses, or deflections) will not be included in the sensitivity analyses, as they would duplicate information provided by other independent variables that are to be studied. However, BRE will conduct additional regressions to include mechanistic responses and thus develop mechanistic-empirical models to predict distresses. The mechanistic responses may be expected to take the place of certain other independent variables or combinations of variabilities, since they are themselves the result of many variables working together. The resulting mechanistic-empirical equations may allow extrapolation of the equations (at lower reliability) to the areas (combinations of independent variables) where few pavements are presently available for analysis.

Mechanistic considerations will also be taken into full account for rigid pavements, using finite element programs and closed form solutions for stresses, deflection, and load transfer. ILLISLAB will be one of the programs utilized.

5. EXPECTED PRODUCTS FROM SENSITIVITY ANALYSES - CHAPTER 5

The specific products expected from the sensitivity analyses are:

1. Identification of the significant independent variables affecting each distress.
2. An indication of how strongly each of these variables, independently and in interaction with others, affect the chosen-distress.
3. Empirical regression equations relating distress variables to the significant independent variables.
4. Mechanistic-empirical equations relating distress variables to significant independent variables and mechanistic responses.

5. Measures of predictive accuracies of the above regression equations.

As discussed previously, the available data for these early analyses is limited, so the predictive equations to be developed may also be expected to be affected by these limitations. However, the analysis team expects the predictive equations to be sufficiently precise to support identification of the independent variables that have major impacts on the performance of pavements, and to allow reasonable precision in the quantification of these impacts.

Both empirical and mechanistic-empirical predictive equations are to be developed. While the statistical adequacy of these equations will vary, it is hoped that most of them will be adequate for use in pavement management systems and as "design checks" in conventional design procedures, such as the AASHTO Design Guide, that do not presently address all types of significant distresses directly. The trend in pavement materials research is toward mix designs that directly consider all major distress types that pavements experience. Design procedures need to reflect this trend as well, seeking to minimize the occurrence of all potential distresses.

Another important product from these early analyses will be the application of the experience gained to develop recommendations for future analyses, when much more time-sequence data will be available. As increasingly better predictive equations are produced during the long-term monitoring of the pavements, design procedures may be broadened to be more comprehensive, dealing directly with all of the major distress types that pavements experience.
The Specific Pavement Studies
Key Issues and Potential Products

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THE SPECIFIC PAVEMENT STUDIES: KEY ISSUES AND POTENTIAL PRODUCTS
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ABSTRACT

The Long Term Pavement Performance studies of the Strategic Highway Research Program includes two types of in-service pavement studies: The General Pavement Studies (GPS) and the Specific Pavement Studies (SPS). The GPS are studies of a broad range of existing in-service pavements with varied design factors and site conditions. The SPS are also sites on in-service roads, but they are constructed to develop better understanding of the effects on performance of a few targeted factors not widely covered in the GPS and thus help achieve the LTPP objectives that cannot be completely met by the GPS.

The initial SPS program consists of eight experiments that address structural factors, pavement maintenance, pavement rehabilitation, and environmental effects. The experiments on structural factors will provide data to help select more economical designs for new and reconstructed flexible and rigid pavements. The experiments on maintenance will yield information to identify the most effective treatments for maintaining asphalt and concrete pavements. The experiments on rehabilitation will identify the most economical methods and strategies for rehabilitation of asphalt and portland cement concrete pavements. The experiment on environmental effects will yield data to improve the prediction of serviceability loss due to environment.

The paper summarizes the experimental plan for the SPS experiments, identifies the potential products and their applications, and highlights the potential benefits to participating agencies.
THE SPECIFIC PAVEMENT STUDIES: KEY ISSUES AND POTENTIAL PRODUCTS
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1. INTRODUCTION

The Long-Term Pavement Performance (LTPP) portion of the Strategic Highway Research Program (SHRP) consists of two sets of studies: The General Pavement Studies (GPS) and the Specific Pavement Studies (SPS). The General Pavement Studies covers many objectives of the LTPP through monitoring of in-service existing pavements with varied design factors and site conditions. Test site selection for the GPS has been in process for over three years and approximately 800 test sections have been identified in the U.S. and Canada. However, existing pavements simply do not provide all the comparisons and parameters needed to study the effect of certain important factors on pavement performance. The Specific Pavement Studies have been structured to develop better understanding of the effects on performance of a few targeted factors not widely covered or well controlled in the General Pavement Studies.

2. STUDY TOPICS

During the course of SHRP's research design, eighteen initial SPS topics were proposed. Over the last several years, SHRP's advisory groups and highway agencies selected the highest priority features - those where improvement potential appears most significant, where design procedures are not well established, or where current practices are most unreliable. Through this process, eight experiments, designated SPS-1 through SPS-8, have emerged as top priorities. These experiments are grouped into four categories as follows:

1. Structural Factors
   SPS-1: Strategic Study of Structural Factors for Flexible Pavements
   SPS-2: Strategic Study of Structural Factors for Rigid Pavements

2. Pavement Maintenance
   SPS-3: Preventive Maintenance Effectiveness of Flexible Pavements
   SPS-4: Preventive Maintenance Effectiveness of Rigid Pavements

3. Pavement Rehabilitation
   SPS-5: Rehabilitation of Asphalt Concrete Pavements
   SPS-6: Rehabilitation of Jointed Portland Cement Concrete Pavements
   SPS-7: Bonded Concrete Overlays of Portland Cement Concrete Pavements

4. Environmental Effects
   SPS-8: Study of Environmental Effects in the Absence of Heavy Traffic
The Specific Pavement Studies on structural factors (SPS-1 and SPS-2), pavement rehabilitation (SPS-5, SPS-6, and SPS-7), and environmental effects (SPS-8) are part of the LTPP program while the studies on preventive maintenance effectiveness (SPS-3 and SPS-4) are part of the Highway Operations portion of the SHRP. The paper addresses the key issues and potential products for the SPS experiments within the LTPP program.

3. EXPERIMENT DESIGN

To ensure practical and implemental experiments, the experiment designs for the SPS experiments were developed in cooperation with state and provincial highway agencies and the Federal Highway Administration. A detailed experiment has been developed for each study to include different levels of climate, subgrade soil, traffic, and factors pertaining to pavement type. Therefore, each SPS experiment requires a number of test sites located in the four climatic regions (wet-freeze, wet-no freeze, dry-freeze, and dry-no freeze). For this purpose, the United States and Canada have been divided into the four climatic regions as shown in Figure 1. Study parameters for each SPS experiment are summarized in Table 1 and described below.

3.1 Strategic Study of Structural Factors for Flexible Pavements (SPS-1)

This experiment examines the effects of climatic region, subgrade soil (fine and coarse grained), and traffic rate (as a covariant) on pavement sections incorporating different levels of structural factors. These factors include drainage (presence or lack of it as provided by an open-graded permeable asphalt-treated drainage layer and edge drains), asphalt concrete surface thickness (4 and 7 in.), base type (dense-graded untreated aggregate, dense-graded asphalt-treated, and combination thereof), and base thickness (8 and 12 in. for undrained sections and 8, 12, and 16 in. for drained sections). This experiment, designed in a fractional factorial manner to enhance implementation practicality, includes 196 test sections located at 16 test sites.

3.2 Strategic Study of Structural Factors for Rigid Pavements (SPS-2)

This experiment examines the effects of climatic region, subgrade soil (fine and coarse grained), and traffic rate (as a covariant) on dowelled jointed plain concrete pavement sections incorporating different levels of structural factors. These factors include drainage (presence or lack of it as provided by an open-graded permeable asphalt-treated drainage layer and edge drains), concrete thickness (8 and 11 in.), base type (dense-graded untreated aggregate and lean concrete), concrete flexural strength (550 and 900 psi at 14 days), and lane width (12 and 14 ft). The experiment, designed in a fractional factorial manner to enhance implementation practicality, includes 192 test sections located at 16 test sites.
Figure 1. SHRP-LTPP Environmental Zones
Table 1. Study Parameters for Specific Pavement Studies

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Study Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPS-1: Structural Factors for Flexible Pavements</td>
<td>Subgrade Type: Fine, Coarse In-Pavement Drainage: Yes, No Base Type: AGG, ATB, ATB/AGG (Undrained Sections); PATB/AGG, ATB/PATB (Drained Sections) Base Thickness: 8, 12 in. (Undrained Sections); 8, 12, 16 in. (Drained Sections) Surface Thickness: 4, 7 in.</td>
</tr>
<tr>
<td>SPS-2: Structural Factors for Rigid Pavements</td>
<td>Subgrade Type: Fine, Coarse In-Pavement Drainage: Yes, No Base Type: AGG, LCB, PATB Slab Thickness: 8, 11 in. Concrete Strength: 550,900 psi (flexural) Lane Width: 12, 14 ft.</td>
</tr>
<tr>
<td>SPS-6: Rehabilitation of Jointed Portland Cement Concrete Pavements</td>
<td>Pavement Type: JPCP, JRCP Pavement Condition: Fair, Poor Restoration Method: Minimal, Full CPR, Crack &amp; Seat AC Overlay: None (Minimal and full CPR) 4 in. (Minimal, full CPR, Crack &amp; Seat) 8 in. (Crack &amp; Seat)</td>
</tr>
</tbody>
</table>
### Table 1. Study Parameters for Specific Pavement Studies (Continued)

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Study Parameters</th>
</tr>
</thead>
</table>
| **SPS-7: Bonded Portland Cement Concrete Overlays** | **Pavement Type:** JPCP, JRCP, CRCP  
**Surface Preparation:** Cold Milling & Sand Blasting, Shot Blasting  
**Bonding Material:** Neat-Cement Grout, None  
**Overlay Thickness:** 3, 5 in. |
| **SPS-8: Environmental Effects in Absence of Heavy Traffic** | **Subgrade Type:** Fine (non-active, swelling, frost-susceptible), Coarse  
**Pavement Structure:** Flexible  
(4 in. AC on 8 in. AGG, 7 in. AC on 12 in. AGG);  
Rigid  
(8 in. JPCP on 6 in. AGG, 11 in. JPCP on 6 in. AGG) |

AGG = Dense-graded untreated aggregate base  
ATB = Dense-graded asphalt-treated base  
PATB = Open graded permeable asphalt treated base  
LCB = Lean concrete base  
AC = Asphalt concrete  
JPCP = Jointed plain concrete pavement  
JRCP = Jointed reinforced concrete pavement  
CRCP = Continuously reinforced concrete pavement  
CPR = Concrete pavement restoration
A supplementary experiment, designated SPS-2A, addresses undoweled plain concrete pavements with skewed joints. This experiment includes the same factor levels for drainage, base types, concrete thickness, and lane width covered in the main experiment, but only one level of strength (550 psi).

Another supplementary experiment, designated SPS-2B, addresses jointed reinforced concrete pavements. This experiment includes the same factor levels for drainage, concrete thickness, concrete flexural strength, and lane width covered in the main experiment, but only one level of base type (lean concrete).

3.3 Rehabilitation of Asphalt Concrete Pavements (SPS-5)

This experiment examines the effects of climatic region, condition of existing pavement (fair and poor) and traffic rate (as a covariant) on pavement sections incorporating different methods of rehabilitation with asphalt concrete overlays. These rehabilitation methods include surface preparation (routine preventive maintenance and intensive preparation with cold milling and associated repairs), type of asphalt overlay (virgin and recycled), and overlay thickness (2 and 5 in.). The experiment includes 128 test sections located at 16 test sites.

3.4 Rehabilitation of Jointed Portland Cement Concrete Pavements (SPS-6)

This experiment examines the effects of climatic region, type of pavement (plain and reinforced), condition of existing pavement (fair and poor) and traffic rate (as a covariant) on pavement sections incorporating different method of rehabilitation with and without asphalt concrete overlays. However, jointed reinforced concrete pavements are not included in the dry zones as the use of this pavement design is not widespread in this climatic zone. These rehabilitation methods include surface preparation (a limited preparation and full concrete pavement restoration) with a 4-in. thick asphalt concrete overlay or without an overlay, crack/break and seat with different asphalt concrete overlays (4 and 8 in.), and limited surface preparation with a 4-in. thick asphalt concrete overlay with sawed and sealed joints. The experiment includes 154 test sections located at 22 test sites.

3.5 Bonded Concrete Overlays of Concrete Pavements (SPS-7)

This experiment examines the effects of climatic region, type of pavement (jointed and continuously reinforced) and condition of existing pavement and traffic (as covariants) on pavement sections incorporating different rehabilitation methods and concrete overlays. These rehabilitation methods include different surface preparation methods (cold milling plus sand blasting and shot blasting), bonding agents (neat cement grout or none) and overlay thickness (3 and 5 in.). The experiment includes 96 test sections located at 12 test sites.
3.6 Environmental Effects in Absence of Heavy Traffic (SPS-8)

This experiment examines the effect of climatic factors in the four environmental regions, subgrade type (frost-susceptible, expansive, fine, and coarse) on pavement sections incorporating different designs of flexible and rigid pavements and subjected to very limited traffic as measured by the Equivalent Single Axle Load accumulation. Pavement structure will include two levels of highway design for each of flexible and rigid pavements. Flexible pavement sections will consist of 4 and 7 in. of asphalt concrete surface on a 8 and 12 in. thick dense-graded untreated granular base, respectively. Rigid pavement test sections will consist of 8 and 11 in. thick doweled jointed plain concrete slabs on 6 in. thick dense-graded granular base. The experiment is designed to include 48 test sections located at 12 to 24 test sites, depending on the practicality of constructing both flexible and rigid pavement test sections at some of the test sites.

4. KEY PRODUCTS

The SPS experiments within the LTPP program will include nearly one hundred test sites with almost 1,000 test sections. As these test sections are monitored from their infancy, a comprehensive data base will provide complete information on the construction, materials, traffic, environment, performance and other features of these sections. This data base will provide a reliable tool for accomplishing the objectives of the Specific Pavement Studies, and will assist other researchers and highways agencies in extending the SPS findings to specific situations of local or regional interest. This data base will be part of the National Pavement Database to be maintained by the Transportation Research Board.

Products that will result from the SPS experiments can be grouped into three categories: general products, specific products, and other products. The general products are those common to all experiments such as evaluation of existing design equations, and the development of improved design procedures for new and reconstructed pavements. The specific products are those obtained from each experiment because of its unique features and study parameters such as the effect of concrete strength or widened lane on the performance of concrete pavements, the effect open graded permeable base on performance of flexible and rigid pavements, and the effect of base type on performance of asphalt concrete pavements. Other products are those resulting from the supplementary studies performed at the test locations such as the effect of tied concrete shoulders on the performance of rigid pavements and those developed in the progress of work to assist in the performance evaluation of SPS test sections or characterization of the pavement materials used in the test sections.

4.1 General Products

Evaluation of existing design methods and performance equations is a key product of the Specific Pavement Studies. For example, the AASHTO pavement design equations can be evaluated by comparing the
observed serviceability index as derived from profile and distress measurements against that predicted by the design equations. Similarly, distress-predictive equations can be evaluated and their reliability in predicting specific distress types can be assessed.

Development of new or improved design equations is another key product of the Specific Pavement Studies. This development may include the development of predictive equations for the significant distress and performance measures and the calibration of mechanistic-empirical model for design. For example, the findings on the influence of climate may permit a more accurate quantification of this factor into empirical design models. Similarly, validation, calibration and/or further development of the more fundamental (mechanistic) models can also be achieved through improvements in the empirical relationships between mechanistic formulated variables and measures of pavement distress, such as the relationship between the computed horizontal strain on the bottom of the asphaltic concrete layer and the development of fatigue cracks as used in the Shell Research and Asphalt Institute design models.

4.2 Specific Products

Each SPS experiment will yield a number of products related to the significance of specific design features and their interaction with other variables, such as climate, on pavement performance. These products will be incorporated in the "general products" to develop improved design procedures for new and rehabilitated pavements and help identify the optimum pavement structure or rehabilitation option for a specific project.

4.2.1 Structural Factors for Flexible Pavements - The experiment on strategic study of structural factors for flexible pavements (SPS-1) will develop conclusions concerning the significance of in-pavement drainage and base type to pavement performance, the long-term effectiveness of in-pavement drainage, and the contribution of base and surface thickness to pavement performance.

The incorporation of in-pavement drainage as a design feature for flexible pavements is an illustration of the "specific products" of this experiment. The climatic conditions, subgrade soil, and pavement layer materials and thicknesses will influence the benefits and long-term effectiveness of in-pavement drainage systems. For example, use of in-pavement drainage may improve pavement performance in wet climates but not necessarily in dry climates. Also, for pavements constructed on fine-grained subgrade, in-pavement drainage may prove beneficial shortly after construction, but contamination with fines may make it less effective in future years. 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.

4.2.2 Structural Factors for Rigid Pavements - The experiment on strategic study of structural factors for rigid pavements (SPS-2) will develop conclusions concerning the significance of in-pavement
drainage and base type to pavement performance, the long-term effectiveness of in-pavement drainage, and the contribution of surface thickness, concrete strength, and widened lane to the performance of dowelled jointed portland cement concrete pavements. The experiment will also yield conclusions concerning the effects of these parameters on the performance of undoweled portland cement concrete pavements with skewed joints and jointed reinforced concrete pavements.

The use of widened lanes and high strength concrete as design features for rigid pavements are two of the "specific products" of this experiment. Each of widened lanes and increased concrete strength will improve pavement performance to varying degrees. In some situations, the contribution of a widened lane to improving pavement performance may outweigh that of an increased concrete strength or slab thickness while in other situations increased pavement thickness may provide best option for performance improvement. Consequently, the "specific product" of this experiment will be a methodology for establishing the optimum combinations of design features for each specific project.

4.2.3 Rehabilitation of Asphalt Concrete Pavements - The experiment on rehabilitation of asphalt concrete pavements (SPS-5) will develop conclusions concerning the contribution of overlay thickness to pavement performance, the significance of AC overlay material (virgin or recycled), pavement condition prior to overlay and the extent of pavement preparation prior to overlay placement to the performance of the rehabilitated pavement.

Selection of the optimum strategy for rehabilitation of asphalt concrete pavements is the most significant of the "specific products" of this experiment. Several rehabilitation techniques and options requiring different initial investments may be feasible. However, each option will require a different level of maintenance and will have a different service life. The optimum strategy is the one providing the least life-cycle cost while ensuring acceptable performance. The experiment will provide the data and tools necessary to compare the performance and economics of eight different rehabilitation options and provide a methodology for selecting the optimum option for each specific project.

4.2.4 Rehabilitation of Portland Cement Concrete Pavements - The experiment on rehabilitation of jointed portland cement concrete pavements (SPS-6) will develop conclusions concerning the contribution of an AC overlay and "saw and seal" technique to the performance of rehabilitated jointed PCC pavements, the significance of the extent of pavement preparation and/or restoration, and pavement condition prior to rehabilitation with or without an AC overlay to pavement performance, and the effectiveness of crack/break and seat as a rehabilitation option.

The procedure for determining the optimum option for rehabilitation of jointed portland cement pavements is the key "specific product" of this experiment. For example, while limited restoration of pavements in poor condition may require the least initial investment, it may require frequent maintenance and eventually major rehabilitation. However, full concrete pavement restoration with
or without an asphalt concrete overlay would require larger investment but less frequent maintenance. The experiment will provide the tools needed to compare the performance and economics of seven different rehabilitation options and provide a methodology for selecting the optimum option for each specific project.

4.2.5 Bonded Concrete Overlays - The experiment on bonded concrete overlays (SPS-7) will develop conclusions concerning the contribution of the bonding grout and overlay thickness to the performance of rehabilitated pavement, and the significance of pavement type and the method of pavement preparation prior to overlay placement to pavement performance.

The use of cement grout for bonding portland cement concrete overlays to existing concrete pavements is an illustration of the "specific products" of this experiment. Although bonding agents have been frequently used for bonded concrete overlays, proper surface preparation may provide the conditions needed to ensure adequate bond and thus eliminates the need for a bonding material. This design feature could result in substantial savings while ensuring acceptable performance. The experiment will identify the conditions and details for using bonded concrete overlays an optimum option for the rehabilitation of portland cement concrete pavements.

4.2.6 Environmental Effects - The experiment on the study of environmental effects in the absence of heavy traffic (SPS-8) will develop conclusions concerning the environmentally induced serviceability loss, the contribution of environment and subgrade soil to distress of flexible and rigid pavements, and the effects of base and surface thickness variation on retarding the appearance of environmentally driven distress.

The specific products of this experiment will include reliable inputs regarding the environmentally-induced serviceability loss and the effects of environment and subgrade type and properties on pavement distress and performance. These inputs will be incorporated in the "general products" to develop improved design procedures for flexible and rigid pavements.

4.3 Other Products

In addition to the "general" and "specific" products anticipated from the SPS, other products will be generated. These "other products" can be grouped into four categories as described below:

1. Test methods developed specifically for evaluation of materials, construction, and performance of SPS test sections. Examples of these are the tests for determining coefficient of thermal expansion of portland cement concrete, test for determining bond shear strength between an existing PCC pavement and a PCC overlay, and a method for determining moisture damage present in asphalt concrete cores. These tests will become standard design and/or material acceptance tests.
2. Correlations between material properties determined by different methods. Because of the familiarity of highway agencies with certain monitoring and testing techniques, additional data will be generated by the participating states that might lead to correlations between the different methods. For example, results of the CBR test used by some states to characterize unbound granular base, subbase, and subgrade materials might show performance correlations to the resilient modulus test data performed on the test sections. Similarly, correlations can be developed between different monitoring equipment such as falling weight deflectometer, Dynaflect, and Roadrater. Where correlations cannot be identified, the importance of the different test methods and/or monitoring techniques for a particular purpose can be established.

3. Study of other features and materials. As supplemental test sections will be constructed as an extension of the SPS test sites, an opportunity will exist for evaluating the contribution of new materials and other pavement details to pavement performance. For example, the effectiveness of tied concrete shoulder in improving pavement performance can be established if a number of sections are constructed on supplemental test sections. Similarly, the effectiveness of innovative materials and features in improving pavement performance can be evaluated as an extension to the SPS experiments.

4. Technology transfer. The interaction between SHRP, highway agencies, and SHRP contractors will provide a means for exchange of ideas that should produce invaluable benefits to the participating organizations.

5. BENEFITS AND IMPACT ON PAVEMENT PRACTICES

The implementation of the SPS program will yield numerous benefits to the participating highway agencies and to other highway authorities in North America and abroad. The following are examples of those benefits that can be easily utilized by the highway agencies as a result of the SPS program:

- Reliable pavement design procedures and standards for new, reconstructed, and rehabilitated pavements
- Reliable pavement distress and performance prediction models
- Reliable maintenance procedures and standards
- Improved cost allocation analysis
- Improved life cycle cost analysis
- Improved pavement management systems

These benefits will enable pavement engineers to identify the optimum pavement design and/or rehabilitation strategy for a given situation. This will result in changes in pavement design and construction practices that will lead to better performance at a lower cost. Examples of potential changes in pavement design and
construction include the following:

- Optimized use of open-graded permeable bases for flexible and rigid pavements through new design procedures that determine long term benefits
- Increased use of recycled asphalt in overlays
- Design procedures for new and reconstructed pavements that permit selection on a site specific basis of design features to insure suitable performance and long-term cost efficiency. Examples would include use of dowels in jointed concrete pavements when load transfer is shown to be a critical factor. Widened lanes, tied concrete shoulders, and concrete strength adjustments are other possible design features
- Increased use of "saw and seal" for asphalt concrete overlays of jointed concrete pavements
- Design procedures for crack/break and seat for restoring concrete pavements in poor condition
- Increased use of bonded concrete pavements for strengthening portland cement concrete pavements to accommodate increased traffic levels
- Timely rehabilitation of pavements with cost efficient rehabilitation

These potential changes are expected to result in improved pavement performance and better utilization of resources.
Expected Changes to the AASHTO Design Guide

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CHAPTER 1  INTRODUCTION

The Long-Term Pavement Performance (LTPP) of the Strategic Highway Research Program has recently completed its third year of field activities. Within this relatively brief period, only 15% of the projected life of the studies, enough progress has been made so that sound projections of the near-term impact of the studies on highway engineering are possible. The earliest products have already been delivered and have been discussed elsewhere (References 1 and 2). These products relate to laboratory and field testing and data gathering techniques.

Within the next few years, products that influence pavement design and, to some degree, pavement construction will become available. These products will derive from the initial analyses of data from the General Pavement Studies (GPS) undertaken by Brent Rauhut Engineering (BRE) and Michigan State University (MSU) under SHRP contracts P-020a and P-020b, respectively, and from work done by the Texas Research and Development Foundation (TRDF) under SHRP contract P-001. Other early data analyses sponsored by SHRP (contracts H-101 and A-005 with the Texas Transportation Institute), the U. S. Federal Highway Administration (FHWA), the Science and Engineering Research Council of the United Kingdom and the Canadian Strategic Highway Research Program will also produce results that will impact pavement engineering practices.

In the United States, the pathway to implementation of such products is the GUIDE FOR DESIGN OF PAVEMENT STRUCTURES (known herein as the GUIDE) published by the American Association of State Highway Officials (AASHTO). This publication is revised periodically by AASHTO, most recently in 1986 (Reference 3) and reflects the consensus opinion of the various state highway agencies on the most appropriate pavement design procedures. The individual state highway agencies and the FHWA rely heavily on the GUIDE in developing design standards and related specifications. Future revisions of the GUIDE will be heavily influenced by the results of the LTPP studies. This
paper suggests what the earliest LTPP influenced revisions are likely to be.

1.1 Objectives of Early Data Analyses

The AASHTO Task Force that created the SHRP research plans set five objectives for SHRP in addition to creating the National Pavement Performance Database. These objectives are to:

1. evaluate existing design methods;
2. develop improved methodologies and strategies for the rehabilitation of existing pavements;
3. develop improved design equations for new and reconstructed pavements;
4. determine the effects of loading, environment, materials properties and variability, construction quality and maintenance levels on pavement distress and performance and
5. determine the effects of specific design features on pavement performance. (Reference 4)

Each of these objectives will be addressed in the data analysis projects noted above, although not all of the objectives are addressed by each of the projects. In this paper the primary focus is on the impacts that the BRE, MSU and TRDF research will have on the GUIDE although there is passing mention of influences from the other research projects. This research is so structured that initial efforts are aimed the fourth and fifth goals first, the determination of the sensitivity of performance to specific factor effects, before evaluating and improving existing design procedures.

It must be noted that the anticipated impacts are derived only from from the first round of data collection for the GPS. Further data collection for GPS and the Specific Pavement Studies will refine the results of this first analyses and permit progress toward other products currently inhibited by the small data set now available.

The research plans for the BRE and MSU projects are set forth in LONG-TERM PAVEMENT PERFORMANCE: Proceedings of the SHRP Midcourse Assessment Meeting (Reference 5).

CHAPTER 2 - RESULTS OF THE "SENSITIVITY" ANALYSES

As noted, the initial SHRP sponsored data analyses will not follow the sequence of objectives as set forth in the research plan but rather follow a more logical path
of focusing first on the factors influencing long-term performance and their assemblage into functional forms of engineering significance before evaluating the current AASHTO design procedures or other general pavement design procedures. While this seems to put off amendment of the GUIDE for some indefinite period, it is, in reality, the most efficient way to ensure that the early results of the LTPP studies will improve design procedures. The two principal products of these sensitivity analyses, individual factor effects and distress specific models, can enhance the use of current procedures by focusing attention to factors that are truly significant to performance in a given design situation and by providing tested distress specific models that engineers can use to supplement general design procedures.

For example, The current AASHTO procedures for design of flexible pavements are based on a definition of serviceability loss heavily weighted toward slope variance (or roughness). While such a procedure can be very effective in comparing relative performance among the components of a highway network, it can overlook the distresses that cause a particular pavement to fail. Rutting, for instance, frequently prompts rehabilitation of pavements even though those pavements have not reached terminal serviceability as defined by the AASHTO procedure.

2.1 PERFORMANCE FACTORS

The paper by Rauhut et al. also presented at this conference shows the "data elements" or performance factors that will be considered in the SHRP sponsored analyses of the early LTPP data. These performance factors will be the set of independent variables to be tested in the sensitivity analyses and later to be utilized in evaluation of existing design models. As should be expected so early in the studies, the factors selected are those that are found in current performance models or, like the climatic factors, are well-known to influence performance and frequently included in recent distress specific models but are absent from most currently used general models. The inclusion of any factor at this stage does not mean that it is, in fact, highly significant to performance. Nor does it mean that other factors are not significant. Their selection means only that current pavement engineering knowledge has judged them important and they are available for study from SHRP's National Pavement Performance Database. Many of these
variables are highly correlated with others in the database and after analysis the choice of the best data element (or combination of elements) may change. In some cases, data from the GPS studies will serve as surrogates for related data that will be provided later through the Specific Pavement Studies or other SHRP research. This is especially true for the factors related to asphalt binders and mixtures. The SHRP Asphalt Research studies are expected to identify new performance related factors or redefine existing ones.

As a product, the identification of these factors and the analysis of the sensitivity of long-term performance to them will probably not result in dramatically new design models. As noted above, these factors are drawn from engineering knowledge and are addressed in existing general design equations or in distress specific predictive models. In some cases, demonstration of their significance may lead to inclusion in procedures that currently ignore them or to substitutions of related factors. These substitutions will occur because the new factor is more strongly correlated with performance or is easier to measure accurately. A prime example of the "substitution" process will be the replacement of pavement serviceability index with roughness in the AASHTO equations.

2.2 Distress Specific Models

Of greater practical importance to highway engineers will be the distress specific models that will come from the sensitivity analyses. These models will not spring from an empirical examination of the data. Rather they will be tested and refined functional forms or equations that were used in the sensitivity analyses. These functional forms and equations, like the factors they relate, will be drawn from current pavement engineering knowledge. In the case of the Michigan State University project, these functional forms will be those included in the finite analysis program "Michpave". The BRE project will look at a wider variety of distresses and can look at several different models related to these distresses.

The BRE research team has completed the selection of significant data elements and the next task is to winnow the number of potential equations or functional forms to a manageable number. Two considerations will dictate this selection. Data availability will drive the selection at first. Both the independent variables
and the predicted responses (or suitable surrogates) must be present in the National Pavement Performance Database. Even then the number of potential models may be quite large and require further reduction. The research team will make an initial proposal of the forms to be included, but additional review by the SHRP Pavement Performance Advisory Committee and the Expert Task Group on Experiment Design and Analysis will ensure that promising options are not neglected. The final group of forms or models to be employed in the sensitivity analysis will no doubt include many currently in use and others that have never been widely used because of the absence of sufficient field data to verify or calibrate them. For some distresses, such as weathering and raveling, well developed predictive forms are scarce and the researchers will have to rely on empirical forms suggested by the data themselves, at least initially.

2.2 Distress Models as Products
In practice, the more successful distress specific predictive models that emerge from this process will be used to supplement current design procedures. Pavement engineers would continue to use existing procedures and then test the final design with these distress specific models to ensure that no specific distress may prove more significant to long term performance the general model suggests or is simply not addressed by the general design procedure. Certainly a reliable procedure to test AASHTO derived designs for rutting would be of great benefit. This scheme of "design and test" will permit the incremental improvement of pavement design procedures as the results of the LTPP studies come on line. It will also allow the orderly introduction of mechanistic models that can handle variations in climate and materials properties that are outside the scope of general models. Highway engineers will be able to test the long term performance benefits or costs in terms of specific distress occasioned by changes in construction materials or unusual site conditions.

Every highway agency could develop a unique set of supplementary models based upon the relative significance of various performance factors within its jurisdiction. Florida obviously would not need a model to predict frost heave or thaw weakening related distress but might find a model that related subgrade stiffness to soil gradation and precipitation indispensable.
CHAPTER 3 -- EVALUATION AND IMPROVEMENT OF THE AASHTO DESIGN EQUATIONS

For the first time, a sufficiently broad database is being made available to make a thorough evaluation of the long term predictive capability of the AASHTO design equations. These equations have not been static but have undergone evolution since first adopted by AASHTO subsequent to the AASHO Road Test of 1958-60 in Ottawa, Illinois. The most recent changes were in 1986 (Reference 3). These changes were prompted both by the experience of agencies using the AASHTO procedures and by advances in pavement design theory. The basic form of the equations remains largely unaffected, however. They are based upon empirically derived relationships among load repetitions, summations of, or surrogates for, materials properties, layer thickness and loss of serviceability.

The equations themselves have proved remarkably resilient. After 30 years, they remain the dominant performance prediction equations used in pavement design. The have also proved to be exceptionally adaptable. Many agencies, both in the U. S. and elsewhere have tested the equations against local conditions and have been able to adjust them accordingly. Recent experiences with pavements that have failed to perform to expectations have suggested that the AASHTO equations may not be robust enough to successfully predict long-term performance under modern traffic conditions or with the wider range of materials used in pavement construction. Consequently, the first objective set for LTPP was the evaluation of the equations and their possible improvement.

An evaluation task is set forth in both of the SHRP sponsored analysis contracts. Further evaluations will be conducted as more time series performance data becomes available. The current evaluations will be handicapped because two key elements, axle load repetitions and initial pavement serviceability, must be estimated. With time, actual distress accumulation can be related to measured axle load repetitions as it was at the AASHO Road Test. This will be particularly true when the specially constructed Specific Pavement Studies begin generating data.

Even the current evaluations will, however, result in improvements to the current AASHTO design procedures. The LTPP database is sufficiently large to provide the first real validation of the equations. While the basic form of the design equations may not change
substantially in the next few years, the confidence with which designers can use the equations will change. The sources for this improved confidence will include:

- Equation Calibration
- Replacement of PSI
- Improved Measures of Materials Variation
- Improved Measures of Traffic Loadings

3.1 Equation Calibration

One of the best ways to appraise the validity of a design equation such as the AASHTO equations is to use data from data bases other than those used to develop the equations. Although the National Pavement Performance Database is in its infancy, there is still an opportunity to use early data from the GPS to assess the validity and precision of the AASHTO equations. The procedure to be used will be to compare predictions from the existing design equations with observed conditions and the analyze the deviations to create adjustment factors. As can be seen from Figure 1, 340 flexible pavement test sections still in their initial service period will be available for this exercise. Similarly, Figure 2, shows 255 rigid pavement sections will be available. The GPS site selection process provided a strong distribution of values across the key parameters for this process. Figures 3 and 4 show the distribution of pavement types across environmental regions and subgrade types by region. The distribution of age of pavement, shown in figure 5 for a single GPS experiment, will offer assistance in overcoming the constraint posed by the absence, at this early stage, of time sequence performance data.

It is impossible at this time, prior to any analyses, to predict the nature of the adjustments or their magnitude, but it seems likely biases in the equations will be revealed that will lead to changes in the coefficients of the terms or even "regionalization" of the equations with different terms carrying slightly different weight depending on the environmental region.

3.2 Replacement of Pavement Serviceability Index

The current equations predict the loss of serviceability of a pavement as a function of traffic,
materials properties and layer thickness. The scale of measure for this loss is the pavement serviceability index or PSI. "PSI is obtained from measurements of roughness and distress, e.g., patching, cracking and rut depth (flexible), at a particular time during the service life of the pavement. Roughness is the dominant factor in estimating the PSI of a pavement. Thus a reliable method for measuring roughness is important in monitoring the performance history of pavements" (Reference 5). This statement from the current GUIDE certainly seems to portend a move away from PSI to measures of roughness that are more easily obtained and easier to compare from time to time and place to place. No standard roughness measure is suggested, however. It is almost a certainty that a standard measure of roughness will be adopted as a result of the LTPP studies. Currently, it appears that the International Roughness Index adopted by the World Bank for the Highway Design and Maintenance Standards Model (HDM) (Reference 6) will become the standard. The IRI seems the likely standard because it correlates consistently well with existing measures of road roughness whether subjective or acquired with profiling devices or response type roadmeters. It is also time stable and reproducible from elevation data.

The early analyses will likely show that IRI correlates just as well with PSI on the GPS test sections. The 1986 Guide indicates that most users have modified the procedures used to calculate PSI to reflect local experience and have adopted a variety of measures for roughness when it is measured. By adopting a common standard, development of roughness under varying conditions can be compared and the local experiences will be more meaningful to all.

3.3 New Serviceability Equations

The adoption of a roughness standard such as IRI to replace PSI opens the door to eventual reformulation of the AASHTO equations into distress specific equations that can predict the accumulation of roughness with time or traffic and even isolate the causes of roughness with roughness progression models such as that employed in the World Bank HDM (Reference 6). While such new serviceability equations are beyond the scope of any current research efforts utilizing LTPP data, they are clearly on the horizon. Their introduction will permit the same equations to be employed in pavement management that were employed in design. Easily acquired observations of roughness or
other distress progression can be used to periodically test the design assumptions. If divergence from predicted progression is noted, the independent variables such as traffic rates or materials properties can be re-examined and service life predictions adjusted accordingly. Thus a mechanism to account for independent parameters that vary with time, of which traffic is undoubtedly the most significant, becomes available to highway engineers.

CHAPTER 4 MATERIALS CHARACTERIZATION AND CONSTRUCTION CONTROL

Engineering designs are only as good as the materials with which they are constructed. A major thrust of the SHRP research effort is the determination of true relationships between highway materials and performance and using this knowledge to improve their durability. The asphalt studies seek to determine the performance relationships of asphalt binders and mixtures and use those relationships to better predict performance and guide materials improvements. The portland cement concrete studies are aimed at identifying materials characteristics or interactions that present long term performance problems, such as D-cracking and alkali-silica reaction, and preventing their occurrence and at improving construction control procedures so that design objectives are met in the field.

4.1 The Role of LTPP

The LTPP studies will determine realistic field ranges and variations of key materials properties and structural features and the sensitivity of long term performance to these variations. Few of these materials concerns actually show up in the current AASHTO equations. Their is no direct link between the current design models and actual field materials properties and only imperfect ways to estimate how variations in those properties affect service life predictions. Nor have agencies undertaken a concerted effort to improve this situation. In fact, agencies may inadvertently adopt policies that run counter to satisfying design assumptions. Figure 6 shows actual field pavement thickness information versus the designed thickness from one SPS test site. The construction control or payment procedures employed in this state make the designed thickness an effective maximum rather than the actual average "as built"
thickness. Further, it is the design thickness that is carried forward into the pavement management system so service life expectancy will be in error. Figure 7, also derived from SPS data, shows that construction policies or practices in a neighboring state might lead to thicker than designed pavements. These two projects are illustrative because the real, or "effective" thickness of these two pavements is roughly the same as is the estimated traffic. Both will probably provide the same service, but because the "as built" pavement section in both varies from the design, one will be regarded as a poor performer and one as a good performer and the variation will be unaccounted for, or worse, attributed to the wrong causes.

4.2 Dealing with Variation

Variation in thickness from the design value and the consequences of this variation are relatively easy to identify. Far more subtle are variations from assumptions regarding materials properties or construction requirements that underlie the structural design method but are not explicitly stated. Field achieved densities, air voids in asphalt pavements, entrained air in concrete, natural moisture, layer moduli, and aggregate particle shape are good examples. In past practice, designers would consciously assume low design values to protect themselves from such variation. For example, many pavements were designed with California Bearing Ratios of 3 or less when the actual values were likely to be somewhat higher.

The 1986 revision of the AASHTO GUIDE introduced the concept of reliability and encouraged designers to use reliability factors as a more rational way of dealing with variation from design values or assumptions. It is in refining the components of the reliability factors that the immediate impact of the LTPP studies will felt. The results of the LTPP studies and other SHRP research will show the sensitivity of long term performance to variation of particular parameters. The field and laboratory test data can then be used to estimate realistic ranges and variability of these parameters. Individual agencies can then determine if their own practices create narrower or broader bands of variation than the national, or international studies, determine.

This sequential process can allow designers to select appropriate reliability factors and direct agencies in improving materials and construction quality control efforts where they will provide the greatest performance pay-off.
As noted above, much of the information to be used in determining and refining reliability will come from the SPS studies and other SHRP research. Some early results from the GPS studies can be put to work very soon, however. Among the materials properties where field variation will be examined are layer moduli, air voids, achieved densities, natural moisture, asphalt content, compressive strength of portland cement concrete and others. The significance of the refinement of estimates these variations is difficult to exaggerate. Volume 2 of the 1986 GUIDE states: 
"...40-45 percent (of the overall prediction variance) is attributed to construction deviations in design factor levels.

"The relatively large share attributed to design factor variance is, of course, justification for quality control measures and expenditures that can reduce this variance and thus increase the reliability level."

CHAPTER 5 PRELIMINARY IMPROVEMENTS IN TRAFFIC CHARACTERIZATION

The initial SHRP sponsored data analysis activities, because of their early start are constrained to use estimates of traffic volumes and equivalent single axle loads provided by the states and provinces. While care will be exercised in using these estimates, their reliability will be suspect, although no more unreliable than estimates that agencies now use for design and management purposes. Certainly, the analyses will show the sensitivity of specific distresses to variations in traffic as well as quantifying the variance of service life predictions developed with the AASHTO design procedures that is attributable to variation in traffic data. What these early analyses will not provide is any guidance in improving these estimates. However, the states and provinces have now embarked on an ambitious effort to provide continuous, site specific traffic data collection at the LTPP test sites. This activity will allow improvements in traffic estimation for design purposes within a few years and will be reflected in GUIDE revisions.

5.1 Traffic Monitoring

In discussing design prediction variation, The 1986 GUIDE, reported that traffic prediction variance is around 20-25% of the overall performance prediction variance and as there are very large differences among
reported values for traffic prediction variance "it appears that further definitive research should be performed, perhaps in the context of long-term monitoring studies." The state and provincial traffic monitoring efforts in support of LTPP are just such studies. By the end of 1993, one full year of data will have been collected at virtually all of the GPS sites as well as at 50 special regional weigh-in-motion sites. The GPS site data will be, as a minimum, continuous volume counts and vehicle classifications supplemented by week long weigh-in-motion sessions conducted seasonally. For the majority of sites, some measure of continuous weigh-in-motion data will also be provided. Analysis of this data will provide algorithms for estimation of key traffic related design parameters, such as seasonality, that accurately reflect variations in those parameters. With these new estimation procedures, the overall performance prediction variance will be reduced substantially.

In addition, because all of this traffic data will be site specific, analysis will show which traffic parameters correlate best with the accumulation of specific distresses. For example, loss of friction may correlate much more precisely with total volumes or truck volumes than with equivalent single axle loads. Some types of rutting may be more sensitive to total axles above a certain weight than with ESALs or total volumes.

While neither of these suggested relationships may be true, traffic data provided pavement researchers in the past would not have allowed the examination of such hypotheses with any sort of confidence.

Chapter 6 EVALUATION OF REVISED REHABILITATION DESIGN PROCEDURES.

The data on pavement rehabilitation in the National Pavement Performance Database are currently too few to permit, by themselves, much progress in developing new design procedures for pavement rehabilitation. For many of the sections in the GPS rehabilitation studies, little or no data were provided on the pavement condition prior to overlay nor is there sufficient time series data to demonstrate the rates at which distresses reappear. Nonetheless, the LTPP studies will have a direct influence on changes to the AASHTO GUIDE recommendations for rehabilitation design.
6.1 NCHRP Project 20-7, Task 39

Recently, AASHTO requested the National Cooperative Research Program (NCHRP) to undertake revision of the Rehabilitation Design Chapter of the current version of the GUIDE. There currently exists no broadly based data set that would permit a thorough test of the NCHRP recommended revisions. SHRP has agreed to cooperate with the AASHTO Joint Task Force on Pavements in evaluation of the new procedures. Under terms of the BRE data analysis contract, the contractors will run iterative tests of the new procedures by comparing the predicted performance of the LTPP test sections with actual observations of performance and using the errors to guide adjustments in the new procedures. Although the LTPP data currently available are but a small portion of what will be eventually be collected, it still contains design and performance information on 185 rehabilitated pavement sections with data well distributed across the key parameters needed for the evaluation of the NCHRP recommended design procedures. As such it will lead to improved precision and expanded applicability of these revised procedures.

This exercise will be completed by the close of 1992 and the new revised procedures will probably be the first revisions to the GUIDE directly influenced by the LTPP studies.

Chapter 7  CONCLUSION

Although still very young, the SHRP LTPP studies are making significant progress toward each of the objectives set forth in the research plans laid by the AASHTO SHRP Task Force in 1986. This progress will be reflected in changes to the AASHTO GUIDE FOR DESIGN OF PAVEMENT STRUCTURES at the time of its next revision, probably around 1995. LTPP derived amendments to the guide will include:

1. newly calibrated design equations;
2. improved procedures for rehabilitation;
3. predictive equations for specific distresses;
4. improved materials characterization;
5. guidance on productive construction quality control activities;
6. new techniques for traffic characterization.

Taken individually, each of these potential revisions would greatly impact real pavement performance or our ability to more accurately predict that performance.
In the aggregate, they will form the core of the largest improvement in practical pavement engineering to occur since the results of the AASHO Road Test were implemented.

REFERENCES


FIGURE 1

Distribution of SHRP Sections
Flexible Pavements

FIGURE 2

Distribution of SHRP Sections
Rigid Pavements
FIGURE 3

GPS Experiments
Rigid and Flexible Pavement sections
per Environmental Regions

FIGURE 4

GPS Experiments
Subgrade Types for SHRP Sections
per Environmental Regions
FIGURE 5

- Distribution of Pavement Age, Experiment GPS-1, AC over Granular Base
FIGURE 6

SPS-S

FIGURE 7

SPS-S
Cost Effectiveness of Asphalt Concrete Overlays
The Canadian Approach

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COST EFFECTIVENESS OF
ASPHALT CONCRETE OVERLAYS
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1.0 BACKGROUND

The Canadian Long Term Pavement Performance (C-LTPP) project is a key element of the Canadian Strategic Highway Research Program (C-SHRP). C-LTPP currently involves pavement performance monitoring of a range of alternative asphalt concrete overlays over asphalt concrete pavements on granular bases on 53 test sections at 20 test sites throughout Canada.

In 1990, an analysis project was initiated as part of C-LTPP. [1] The objective of this analysis project was: "to review from an analysis perspective, the type, quantity and quality of the data being assembled under the C-LTPP program so as to ensure that this data could support the statistical and other analysis required to determine the cost effectiveness of alternative rehabilitation methods and to determine the effect of traffic, environment, soil type, etc. upon pavement performance".

This review resulted in a number of conclusions, including:

1) While the analysis framework proposed for the C-LTPP monitoring data had been previously designed in conceptual terms, it had not been developed in detail. There was thus a clear need to develop the analysis framework in full detail.

2) The data currently being collected under C-LTPP is insufficient to support the required cost effectiveness or statistical analysis. This is attributable to a number of factors. First, the data collection activities were long on agency costs and short on user costs. There is therefore a need to collect data related to user costs if the cost effectiveness of rehabilitation alternatives is to be determined. Second, the sample sizes under the C-LTPP program are orders of magnitude too small relative to the number of variables involved to expect meaningful results using classical regression analysis. Third, the classical regression approach will not deliver useful results for 15-20 years, even without the small sample
size problem. That is, it would be necessary to observe the performance of pavements through a full life cycle before meaningful results could be obtained.

A number of recommended corrective actions resulted from the review of the proposed analysis framework. The recommended corrective actions included:

1) to expand the C-LTPP data collection activities to include the data required to determine user costs, and

2) use Bayesian statistical methods to address the small sample size problem. [2]

2.0 DETAILS OF THE C-LTPP ANALYSIS FRAMEWORK

C-LTPP is directed at getting more value for the dollars spent on pavements. There are two basic ways to get more for the dollar spent. These are often not separated nor well articulated. They are:

1) making better decisions given the technology of the day, (ie. better management), and

2) improving today's technology, (ie. better science).

C-LTPP has built and is evaluating test sections using different materials, methods, etc. C-LTPP is thus a full scale, real-time experiment. This at least implies that C-LTPP is directed primarily at better science, with the premise that better science will lead to better management.

Within the context of C-LTPP, the analysis framework must be able to determine for any given section of road, which pavement rehabilitation strategy is the most cost effective. While there is some considerable controversy as to how this cost effective analysis might be undertaken, there is compelling evidence in the research literature to suggest that the appropriate approach is life cycle costing. This involves the total costs throughout the life cycle of the pavement. The most cost-effective strategy is the one that minimizes life cycle costs. Therefore the job at hand is to develop the details of an analysis framework that will determine the cost-effectiveness of various pavement rehabilitation strategies. That is, from all of the rehabilitation strategies available, (R1, R2, R3 ...etc.), determine for road segment i, the rehabilitation strategy that minimizes life cycle costs.
The details of the C-LTPP analysis framework are illustrated in Figure 1. The analysis framework consists of four basic modules, including from left to right, the database module, the performance prediction module, the life cycle costing module and the prioritization module.

In reviewing this analysis framework, it is convenient to begin with the output of the analysis framework, namely the output of the prioritization module, and use the output requirements of each module to dictate the input requirements for the same module, and in turn to use the input requirements of one module to determine the output requirements of the previous module. This in essence is working from right to left in Figure 1, with the output requirements determining the type of analysis required which in turn determines the data requirements.

Reviewing Figure 1 from right to left, the output of the C-LTPP analysis framework, and thus the output of the prioritization module, must be for any particular road segment, the prioritization or ranking of rehabilitation strategies R1, R2, R3...etc. according to life cycle costs, minimum to maximum.

This output requirement in turn dictates the requirement to do life cycle costing (ie. input into the prioritization module and thus output of the life cycle costing module).

In terms of life cycle costing, there is a need to include both agency and user costs in this analysis. This in turn dictates the requirement to have a user cost model, an agency cost model, and be able to predict pavement condition as a function of time. In particular with respect to predicting pavement condition, there is a requirement to be able to predict for any particular road segment, and rehabilitation strategy, the condition of the pavement as measured in terms of the severity and extent of various distress types, at all times throughout the pavement's life cycle. We must know the condition of the pavement at all times in order to be able to determine agency and user life cycle costs.

The input requirement for the life cycle costing module becomes the output requirement of the performance prediction module. This in turn dictates the input requirements to the performance prediction module, namely a combination of information in terms of objective data, (ie. C-LTPP and other) and expert judgment in terms of priors, combined with an analysis methodology capable of producing predictive models of
Figure 1
C-LTPP Analysis Framework

Database Module

Input
- Referencing System
- Inventory
  - Details of initial construction of segment
- Monitoring
  - Maintenance
  - Condition history
  - Traffic
  - Etc.

Output
- Computerized (Relational) Database
- Summary Information
  - Statistics
  - Maps
  - Other

Performance Prediction Module

Input
- Information
- Database
  - C-LTPP
  - Other
- Expert Judgement
- Prior knowledge
- Bayesian Statistical Methodology

Output
- Predictions
  - For road segment i and rehabilitation strategy r, predict pavement condition (severity and extent of distress type k) at time t.
  - Deterioration
  - Gain

Life Cycle Cost Module

Input
- For road segment i and rehabilitation strategy r, predict pavement condition (severity and extent of distress type k) at time t.
- Agency cost model
- User cost model

Output
- Life cycle agency costs for road segment i under rehabilitation strategy r
- Life cycle user costs for road segment i under rehabilitation strategy r
- Total life cycle costs for road segment i under rehabilitation strategy r

Priorization Module

Input
- Life cycle costs for road segment i under rehabilitation strategies r1, r2, r3...

Output
- For road segment i, rank rehabilitation strategies r1, r2, r3... according to total life cycle costs (minimum to maximum)

New Technology Module

Input
- New Methods (management, design, construction, maintenance, etc.)
- New Products (asphalt binder, additives, etc.)

Output
- Cost-Effectiveness of New Methods/New Products

- Missing Element
pavement condition as a function of time. The predictive modelling methodology utilized in the C-LTPP analysis framework involves a Bayesian statistical methodology.

Information input requirements of the performance prediction module dictate the output of the database module. The C-LTPP data will be available within a computerized relational database.

A review of the C-LTPP analysis framework indicates that there are some major missing links relative to available analysis methodologies. In particular, with respect to agency and user cost models, and performance prediction capability. In terms of agency cost models, satisfactory analytical methodology exists, required data is available, therefore it is possible to develop the required agency cost models.

The situation is somewhat different in the case of user cost models. These have proven to be very difficult to develop, in spite of the fact that considerable resources have been directed to development of user cost models in the past. Further, there is considerable strongly held, highly divergent views with respect to user costs and how they ought to be estimated. The issue of user costs must be resolved. There is reason for optimism in that recent attempts at estimating user costs, using a fundamental engineering approach, have proven rather successful. [3] In short, the user cost issue needs to be resolved and can be with a concentrated effort.

Performance prediction has proven to be a very elusive target. This area has long been recognized as being the key to understanding the performance of pavements, (ie. better science), as well as the key to better pavement management systems (ie. better management).

Classical regression analysis has been widely used in attempting to predict the performance of pavements. These conventional statistical methods have experienced at best limited success, primarily because of limited and "dirty" data which have resulted in highly uncertain results (ie. large uncertainties in the estimates provided by the regression models).

C-LTPP will provide both more data and clean data. Unfortunately, even C-LTPP suffers from a small sample size problem when classical regression methods are applied. Further, C-LTPP results would not be available for 15-20 years if classical regression methods were used.
To overcome the limitations of the classical regression methods, Bayesian statistical methods have been utilized within the C-LTPP analysis framework. Bayesian statistical methods directly address the issue of small sample size and will provide results early in the C-LTPP program. The Bayesian statistical methodology has the unique capability of being able to combine objective information like the C-LTPP data with subjective information in terms of expert judgment. This capability substantially expands the information base on which the predictive models can be based.

3.0 PROGRESS TO DATE

The analysis efforts within the context of C-LTPP were initiated in 1990. There are a number of accomplishments to date, including:

1) an overall C-LTPP analysis framework has been detailed,
2) predictive modelling has been identified as the key missing link,
3) Bayesian statistical methods have been established as the key to predictive modelling,
4) the Bayesian "math" has been completed,
5) functional Bayesian software is up and running,
6) the Bayesian methodology has been successfully demonstrated on existing data from several jurisdictions, including Newfoundland, Ontario and Saskatchewan.

Planned activities include the application of Bayesian methods to the C-LTPP data, making the Bayesian software user friendly, and providing the training and support necessary to make the Bayesian software available to sponsoring agencies.
ENDNOTES


3. Research currently under way by Mr. Curtis Berthelot, Graduate Student, Department of Civil Engineering, University of Saskatchewan, Canada.
Figure 1
C-LTPP Analysis Framework

Database Module

**Input**
- Referencing System
- Inventory
- Details of initial construction of segment
- Monitoring
  - Maintenance
  - Condition history
  - Traffic
  - Etc.

**Output**
- Computerized (Relational) Database
- Summary Information
  - Statistics
  - Maps
  - Other

Performance Prediction Module

**Input**
- Information
  - Database
  - C-LTPP
  - Other
  - Expert Judgement
  - Priors
  - Bayesian Statistical Methodology

**Output**
- Predictions
  - For road segment i and rehabilitation strategy r,
    predict pavement condition (severity and extent of distress type k) at time t.
  - Deterioration
  - Gains

Life Cycle Cost Module

**Input**
- For road segment i and rehabilitation strategy r,
  predict pavement condition (severity and extent of distress type k) at time t.
  - Agency cost model
  - User cost model

**Output**
- Life cycle agency costs for road segment i under rehabilitation strategy r
- Life cycle user costs for road segment i under rehabilitation strategy r
- Total life cycle costs for road segment i under rehabilitation strategy r

Prioritization Module

**Input**
- Life cycle costs for road segment i under rehabilitation strategies r1, r2, r3...

**Output**
- For road segment i, rank rehabilitation strategies r1, r2, r3... according to total life cycle costs (minimum to maximum)

New Technology Module

**Input**
- New Methods (management, design, construction, maintenance, etc.)
- New Products (asphalt binder, additives, etc.)

**Output**
- Cost-Effectiveness of New Methods/New Products

- Missing Element
Long Term Pavement Performance Trials
and Data Analysis in the United Kingdom

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Abstract
At the Gothenburg Conference in 1989 the possibility of longterm pavement performance studies (LTPP) in Europe was discussed. It was agreed that European LTPP pilot-trials would be carried out to identify any problems of complying with the Strategic Highway Research Program (SHRP) requirements for constructing and monitoring test sections and for transferring data. Test sections of bituminous overlays on heavily trafficked jointed concrete roads have been constructed in the UK to assess the feasibility of involvement in the SHRP LTPP programme. The UK test sections comply with the requirements of the SHRP General Pavement Studies 7B (GPS 7B) and Special Pavement Studies 6 (SPS 6). The objective of the UK project is to develop a design procedure for overlays on jointed concrete roads.

The condition of the test sections is being assessed from detailed visual condition surveys and from measurements made with the falling weight reflectometer and a range of high speed survey vehicles. The inventory and condition data are being stored on a copy of the SHRP Database in preparation for transfer to the main SHRP Database in Washington.

In parallel with this work, statistical techniques are being developed to analyse and interpret the road condition information from the General Pavement Studies (GPS) and Special Pavement Studies (SPS) in the USA for application in the UK. The pavement condition data being entered into the UK database is as yet insufficient to test the statistical analysis procedures being developed. Consequently the procedures are being evaluated using simulated pavement condition data.

In the paper details of the UK trials will be discussed and a preliminary evaluation of the statistical analysis procedures will be presented.
LONG TERM PAVEMENT PERFORMANCE TRIALS AND DATA ANALYSIS IN THE UNITED KINGDOM

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1. INTRODUCTION

In the United Kingdom more than 500 bituminous road test sections and about 250 concrete sections have been constructed during the last 30 years. Pavement design methodologies currently used in the UK are based on the performance of these test sections. In common with other countries studying long term pavement performance (LTPP), past trials have generally been monitored using techniques that are different from those specified in the Strategic Highway Research Program (SHRP) and therefore these historical studies are not directly usable in the SHRP LTPP programme.

At the SHRP conference in Gothenburg in 1989 it was agreed that European LTPP pilot trials would be carried out to complement and extend the General Pavement Studies (GPS) and Specific Pavement Studies (SPS) experiments in the USA and Canada. These trials would help to address questions about differences in materials, units of measurement, performance monitoring and data storage and transfer.

Test sections of bituminous overlays on heavily trafficked jointed concrete roads are being constructed in the UK which comply with the requirements of SHRP GPS-7B (future AC overlay of PCC pavement) and SPS-6 (rehabilitation of jointed PCC pavements). The objective of the UK trials is to develop a design procedure for overlays on jointed concrete roads. In parallel with this work, statistical techniques are being developed to analyse and interpret the road condition information from the GPS and SPS experiments in the USA and Europe for application in the UK. These will complement the data analyses being carried out under SHRP research contracts in the USA.

2. DESIGN OF ROAD TRIALS

The test sections are designed to provide data on the performance under heavy traffic of different thicknesses of hot rolled asphalt overlay on jointed reinforced and unreinforced concrete pavements. In addition to the general programme of concrete pavement maintenance and rehabilitation practiced in the UK, "crack and seat" and "sawcut and seal" techniques are being evaluated and the potential benefits of using geotextiles with bituminous overlays are also being assessed. The test sections in the UK form part of major rehabilitation schemes and therefore it is not expected to be able to include any sections which will not be overlaid.

The proposed design of the UK SPS-6 experiment is shown in the extended SHRP matrix in Figure 1. The UK is located in the wet, no-freeze climate zone and the standard SHRP matrix has been extended in this area to include additional test sections. It is intended that each shaded cell should contain at least one test section. The basic design of the GPS-7B experiment is shown in Figure 2. The performance of concrete pavements overlaid with between 75mm and 200mm of hot rolled asphalt will be studied.
### Fig. 1 UK - LTPP test sections SPS 6

<table>
<thead>
<tr>
<th>Climate</th>
<th>Wet freeze</th>
<th>Wet, no freeze</th>
<th>Dry freeze</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pavement type</td>
<td>JPCP</td>
<td>JRCP</td>
<td>JPCP</td>
</tr>
<tr>
<td>Initial condition</td>
<td>F</td>
<td>P</td>
<td>F</td>
</tr>
<tr>
<td>Maintenance</td>
<td>Overlay inches</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Routine maint.</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum restoration</td>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum restoration</td>
<td>4'</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crack/break and seat</td>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Key:
- Saw and seal overlay above joints
- F - Fair condition
- P - Poor condition

### Fig. 2 UK-LTPP test sections GPS 7B

<table>
<thead>
<tr>
<th>Climate</th>
<th>Wet no-freeze</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sub-grade type</td>
<td>Fine</td>
</tr>
<tr>
<td>Traffic rate</td>
<td>High</td>
</tr>
<tr>
<td>Pavement type</td>
<td>JPCP</td>
</tr>
<tr>
<td>Initial condition</td>
<td>Fair</td>
</tr>
<tr>
<td>Overlay thickness (inches)</td>
<td>3</td>
</tr>
</tbody>
</table>

Fig. 1 UK - LTPP test sections SPS 6

Fig. 2 UK-LTPP test sections GPS 7B
Construction of the UK trial sites for GPS and SPS experiments began in 1990. Two trial sites have been constructed and two more are planned. The first was constructed in May 1990 on the London bound carriageway of the M2 Motorway which links London with Dover in the south of England. The motorway has two lanes and a hard shoulder in each direction. Three sections of the trial site had needle punched geotextile bonded with bitumen to the concrete before overlaying. Ethylene vinyl acetate (EVA) modified binder was incorporated in the hot rolled asphalt applied on four sections. In general, the condition of the concrete was poor particularly at some joints where extensive spalling had occurred and at others where temporary repairs had been carried out. Many of these joints were renewed mainly on the basis of their visual appearance immediately before overlaying. Detailed design information about the trial site and the overlays is given in Table 1.

Table 1. Design and construction of trial on M2 motorway

<table>
<thead>
<tr>
<th>Section Number</th>
<th>Geotextile</th>
<th>Overlay Material</th>
<th>Overlay Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Yes</td>
<td>Hot Rolled Asphalt with EVA</td>
<td>75mm</td>
</tr>
<tr>
<td>2</td>
<td>Yes</td>
<td>Hot Rolled Asphalt with EVA</td>
<td>100mm</td>
</tr>
<tr>
<td>3</td>
<td>No</td>
<td>Hot Rolled Asphalt with EVA</td>
<td>100mm</td>
</tr>
<tr>
<td>4</td>
<td>No</td>
<td>Hot Rolled Asphalt</td>
<td>100mm</td>
</tr>
<tr>
<td>5</td>
<td>No</td>
<td>Hot Rolled Asphalt</td>
<td>200mm</td>
</tr>
<tr>
<td>6</td>
<td>Yes</td>
<td>Hot Rolled Asphalt with EVA</td>
<td>140mm</td>
</tr>
</tbody>
</table>

Original Construction:
- **Sub-base:** 200mm Flint Gravel
- **Concrete:** 250mm Jointed Reinforced
- **Spacing of joints:** 24 metres (80 feet)
- **Date opened to traffic:** 1963
- **Cumulative traffic in lane 1:** 32 million Equivalent Standard Axle Loads (ESALs) of 8,200 kg

Bituminous Overlay:
- **Date overlaid:** May 1990
- **Length of trial sections:** 150 metres
- **Annual traffic in lane 1:** 2.2 million ESALs

The second trial site was constructed in October 1990 on the A45 trunk road, a by pass at Bury St. Edmunds in Suffolk in eastern England. This is a dual carriageway road with two traffic lanes in each direction passing through an urban area. The concrete was generally in fair condition and very few repairs were carried out to the concrete before overlaying. The thickness of overlay was limited by drainage considerations in the urban area and it was decided to incorporate styrene butadiene styrene (SBS) as an additive in the hot rolled asphalt overlay for the main works. The trial site comprises three pavement sections, one situated in the main length with the other two adjacent. Details of the overlay applied are given in Table 2.
Table 2. Design and construction of trial on A45 trunk road

<table>
<thead>
<tr>
<th>Section Number</th>
<th>Overlay Material</th>
<th>Overlay Thickness</th>
<th>Saw-cut and Seal</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Hot Rolled Asphalt</td>
<td>100mm</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>Hot Rolled Asphalt</td>
<td>100mm</td>
<td>No</td>
</tr>
<tr>
<td>3</td>
<td>Hot Rolled Asphalt with SBS</td>
<td>100mm</td>
<td>No</td>
</tr>
</tbody>
</table>

Original Construction:
- Sub-base: 200mm Cement-bound Granular Material
- Concrete: 250mm Jointed Plain
- Spacing of joints: 5 metres (16 feet)
- Date opened to traffic: 1974
- Cumulative traffic in lane 1: 23 million ESALs

Bituminous Overlay:
- Date overlaid: October 1990
- Length of trial sections: 150 metres
- Annual traffic in lane 1: 2 million ESALs

Two further sites have been identified for trials to be constructed as part of major rehabilitation schemes in 1991/1992, one on a three lane motorway and the other on a dual carriageway trunk road. The GPS-7B trial site proposed on the trunk road consists of two sections of 200mm of hot rolled asphalt overlay on an unreinforced jointed concrete pavement. One section is planned to be on concrete in fair condition requiring minimal restoration and the other on concrete in poor condition requiring maximum restoration before overlay. The trial proposed on the motorway is an SPS-6 trial site, also on a jointed plain concrete pavement. It is planned to have six sections, three with crack and seat, and three control sections with overlays of various thicknesses of hot rolled asphalt.

3. MEASUREMENTS BEFORE OVERLAYING

3.1 Surface condition

A detailed visual survey was carried out at each trial site to record the general condition and specific defects within the concrete slabs and at the joints. In conjunction with other measurements these surveys were used to select the precise location of the test sections.

3.2 Slab movement

It is most important to establish the condition of joints in concrete pavements before overlaying in order to assess their ability to accommodate thermal movements and to transfer traffic loads between slabs. If adjacent concrete slabs are not free to slide along load transfer bars then high stresses can be generated within the slabs due to seasonal changes in temperature. This could result in mid-bay cracking and spalling at the joints. Thermal movements are larger at joints that move freely and this increases the potential for reflection cracking in overlays placed directly above such joints. In order to compare the long term performance of different test sections, particularly with reference to reflection cracking, the ability of individual slabs to contract and expand has to be measured.
In the UK thermal movements at joints are measured using the Demec gauge. Pairs of metal studs are inserted in the surface of the concrete at approximately 200mm spacing spanning a joint or crack and these act as a datum for the measurements. The separation of the studs is measured using the Demec gauge at different times during the year to cover a range of temperatures within the concrete. The thermal movements are expressed in terms of hundredths of a millimetre (0.01 mm) per deg C. For the trial site on the M2 motorway many of the joints were found to be locked but those on the trial site on A45 were all moving freely, expanding by about 0.06 mm per deg C change in temperature.

3.3 Deflections

Before remedial work was carried out on the concrete at the trial site on M2, a Dynatest 7000 Falling Weight Deflectometer (FWD) was used to measure deflections in the outer wheel path of lane 1 at both sides of joints, transverse cracks and at the geometric centre of the slabs in accordance with the SHRP LTPP manual for FWD testing (1989). The measurements were made deliberately during cold weather when the concrete had contracted and the joints and cracks were relatively wide; this represents the most severe situation for load transfer. The applied loads and the number and sequence of drops are shown in Table 3. The locations of the FWD loading plate and detectors relative to the joints are shown in Figure 3.

Table 3. Loading sequence for FWD

<table>
<thead>
<tr>
<th>Load Sequence</th>
<th>Applied Load (kPa)</th>
<th>Number of Drops</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>770</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>580</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>770</td>
<td>4</td>
</tr>
<tr>
<td>4</td>
<td>1020</td>
<td>4</td>
</tr>
</tbody>
</table>

Fig. 3 FWD measurement of load transfer at joints and cracks
The ratio of deflections measured at points equidistant from the loading plate gives a measure of the load transfer across the joint or crack. For perfect transfer the ratio should be unity and as the degree of transfer reduces then the ratio will also decrease which indicates the need for remedial work. Figure 4 shows how the ratios measured at the joints varied along the site on the M2 motorway. Results from other sites have shown that there is a general variation in the load transfer ratio of plus or minus 5 % across joints with good interlock. The ability of joints and cracks to spread traffic loads can be categorised as shown in Table 4.

Table 4. Load transfer categories

<table>
<thead>
<tr>
<th>Category</th>
<th>Deflection Ratio</th>
<th>Load Transfer Ability</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&gt; 0.9</td>
<td>Good</td>
</tr>
<tr>
<td>2</td>
<td>0.9 - 0.7</td>
<td>Partial</td>
</tr>
<tr>
<td>3</td>
<td>0.7 - 0.5</td>
<td>Poor</td>
</tr>
<tr>
<td>4</td>
<td>&lt; 0.5</td>
<td>Very Poor</td>
</tr>
</tbody>
</table>

Fig. 4. FWD Measurements at Concrete Joints (UK-LTPP GPS-7B: M2 Motorway)
Analysis of the FWD measurements at the joints showed that the load transfer ratios were not influenced by the level of applied load as shown in Figure 5. It was also found that the deflections generated by the 770 kPa load applied initially, as recommended by SHRP and shown in Table 3, were identical to those generated by the same load applied later in the sequence. These results have very important practical implications. By using only one 'standard' load (700kPa), the time required to take the measurements could be reduced by 75 per cent. The testing time could also be reduced further by carrying out the measurements with the FWD positioned on one side of the joint only. In the UK, measurements are usually made with the FWD plate positioned after the joint as shown at (a) in Figure 3. Operating such a testing pattern would enable the LTPP sites to be surveyed more frequently.

![Deflection Ratio for high or low load](image)

**Fig. 5.** Effect of FWD Load at Concrete Joints (UK-LTPP GPS-7B: M2 Motorway)

3.4 Longitudinal profile

The longitudinal profile and the surface texture of the concrete were measured using the TRRL High-speed Road Monitor (HRM). Specific defects including slab misalignment caused by subsidence, slab cracking and stepping at joints can be identified by these measurements as shown in the example in Figure 6.

3.5 Traffic

All sites selected for LTPP studies in the UK are heavily trafficked by cars and commercial vehicles. The general method of assessing traffic loading on the motorway and trunk road network in the UK is to carry out regular classified traffic counts at 3000 sites on a rotating basis. Each site is surveyed approximately once in five years. Continuous counts and measurements are also taken at strategic sites with weigh scales. The results from these surveys have enabled procedures to be developed for predicting the traffic loading at any
location from comprehensive classified traffic counts carried out nationally as described by Robinson (1988). The trial site on the M2 motorway had an estimated traffic loading of 2.2 million ESALs in 1990 and the trial site on the A45 trunk road had an estimated loading of 1.85 million ESALs.

![Diagram of slab movement](image)

**Fig. 6. Example of slab movement on an unreinforced concrete motorway**

4. MATERIALS TESTING

4.1 Sampling

During construction of the trials, samples of hot bituminous materials were taken from the hopper of the paving machine and tested for compliance with specifications in terms of aggregate grading and binder content. For all the test sections, the materials were within specification. Temperatures of the hot bituminous mixes were monitored on site, in and behind the paver, to ensure that laying and compaction were carried out to specification.

The bituminous materials and the concrete were sampled by coring to determine thickness of the layers and to carry out performance related laboratory tests. Cores of 150mm diameter, were taken in the late spring of 1991 from between the wheel tracks of lane 1 at intervals along the test sections in order to obtain a measure of any variability which might occur along the trial lengths. This procedure differs from that recommended by SHRP where the samples are taken from the end of the test section only thus inhibiting the assessment of variability.

4.2 Laboratory tests

In addition to extracting the 150mm diameter cores, 100mm diameter cores were also extracted at the trial site on the M2 motorway, from above the Demec studs at the joints to enable thermal joint movements to be monitored under the overlays. These cores provided additional information about layer thickness and were suitable for some of the laboratory tests. It was not possible to obtain samples from the granular sub-bases or from the underlying subgrades. However, some information on the type and thickness of the materials is available from records of the original construction. Bulk samples of aggregate, bitumen and the mixtures were taken and stored according to SHRP requirements for use in the asphalt and asphalt mixture testing programme in the USA. The tests being carried out on cores extracted from the trial sites are listed in Table 5.
Table 5. UK-LTPP Laboratory tests of materials

<table>
<thead>
<tr>
<th>Tests On Concrete</th>
<th>Tests On Asphalt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core examination and thickness</td>
<td>Core examination and thickness</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>Bulk and maximum specific gravity</td>
</tr>
<tr>
<td>Splitting tensile strength</td>
<td>Elastic stiffness</td>
</tr>
<tr>
<td>Static modulus of elasticity</td>
<td>Creep stiffness</td>
</tr>
<tr>
<td></td>
<td>Wheel-tracking rate</td>
</tr>
<tr>
<td></td>
<td>Type and classification of coarse and fine</td>
</tr>
<tr>
<td></td>
<td>aggregate</td>
</tr>
<tr>
<td></td>
<td>Bitumen content</td>
</tr>
<tr>
<td></td>
<td>Penetration of bitumen at 55F, 77F and 275F</td>
</tr>
<tr>
<td></td>
<td>Viscosity of bitumen at 77F, 140F and 275F</td>
</tr>
</tbody>
</table>

5. PERFORMANCE MONITORING

The in-service performance of the road trials is being assessed by annual measurements of road condition. It is planned to make all the measurements, except skid resistance, in the springtime (March-June). Skid resistance, measured in terms of the Sideways Force Coefficient using the Sideways-force Routine Investigation Machine (SCIRM) is expressed as a mean value measured during the summer months as described by Rogers and Gargett (1991). It is intended that some measurements of road condition will be repeated in the autumn (September-November) particularly after any deterioration in condition is observed. The long term condition measurements that will be carried out are as follows:

* Visual condition of the road surface
* Thermal movement at joints and cracks
* Deflections at joints and cracks
* Longitudinal surface profile
* Depth and shape of wheel track ruts
* Surface texture
* Skid resistance

In addition to the road condition measurements, traffic loading and the environmental conditions will be monitored although it is not proposed to install permanent weigh scales or weather stations at the trial sites. The traffic loading will be determined from regular classified traffic counts using the relationships developed from research studies by Robinson (1988) and environmental condition will be extracted from national weather records kept for regional weather stations spread throughout the UK.

Equipment developed at TRRL is being used to record some of the road condition parameters. The high-speed road monitor (HRM) described by Cooper (1985) will measure the longitudinal surface profile and surface texture. The transverse profilometer described by Potter and O’Connor (1990) will measure the depth and shape of wheel track rutting, and the surface friction will be measured using the SCIRM.
6. DATA TRANSFER TO NIMS DATABASE

Pavement data emanating from the trial sections being monitored by TRRL are to be transferred in a two stage process to the National Information Management System (NIMS) held in Washington. In the first stage, the data will be entered into a copy of the IMS held at the University of Birmingham. During this stage, a number of data validation checks will be performed as part of the quality assurance required to be done for all data from LTPP experiments. The second stage will be the transfer to NIMS in Washington, initially of inventory data and subsequently of pavement monitoring data on a regular basis.

A number of modifications to the IMS database have been necessary in order to accommodate data from the LTPP experiments being carried out in the UK. This is mainly due to three areas of significant deviation from LTPP specifications given in the Data Collection Guide (DCG) published by SHRP (1989). These are:

- Differences in material types used in pavement layers
- Unit measurement standards
- Equipment and test methods not included in LTPP specifications

The modifications made to the IMS fall into two groups; firstly those which require extensions or alterations to the standard data codes defined in the DCG, and secondly modifications required to incorporate data items not catered for within the structure of the IMS and hence new tables or amendments to existing tables are required. An example of the first group of modifications is given in Table 6. This shows pavement layer materials commonly used in the UK with the equivalent, or nominally similar, layer materials defined in the DCG. The table highlights some of the modifications which are necessary when the IMS is used outside North America. The UK specification for a Dense Bitumen Macadam (DBM) is different from that of a dense graded Hot Mix Asphalt Concrete (HMAC) defined in the DCG.

<table>
<thead>
<tr>
<th>Pavement Layer</th>
<th>UK Layer Material Type</th>
<th>SHRP Layer Material Type</th>
<th>DCG Table</th>
<th>DCG Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wearing Course</td>
<td>Rolled Asphalt</td>
<td>Sand Asphalt</td>
<td>A.5</td>
<td>3</td>
</tr>
<tr>
<td>Wearing Course</td>
<td>Bitumen Macadam</td>
<td>Hot Mix, Hot Laid Asphalt Concrete, Dense Graded</td>
<td>A.5</td>
<td>1</td>
</tr>
<tr>
<td>Base Course</td>
<td>Rolled Asphalt</td>
<td>Sand Asphalt</td>
<td>A.6</td>
<td>3</td>
</tr>
<tr>
<td>Base Course</td>
<td>Bitumen Macadam</td>
<td>Hot Mix, Hot Laid Asphalt Concrete, Open Graded</td>
<td>A.6</td>
<td>2</td>
</tr>
<tr>
<td>Road Base</td>
<td>Hot Rolled Asphalt</td>
<td>Sand Asphalt</td>
<td>A.6</td>
<td>46</td>
</tr>
<tr>
<td>Road Base</td>
<td>Dense Bitumen Macadam</td>
<td>Dense Graded, Hot Laid Asphalt ,Central Plant Mix</td>
<td>A.6</td>
<td>28</td>
</tr>
<tr>
<td>Road Base</td>
<td>Wet Mix</td>
<td>Crushed Stone, Gravel or Slag</td>
<td>A.6</td>
<td>23</td>
</tr>
<tr>
<td>Subbase</td>
<td>Gravel</td>
<td>Gravel (uncrushed)</td>
<td>A.6</td>
<td>22</td>
</tr>
<tr>
<td>Subbase</td>
<td>Crushed Rock</td>
<td>Crushed Stone, Gravel or Slag</td>
<td>A.6</td>
<td>23</td>
</tr>
<tr>
<td>Subbase</td>
<td>Fly Ash</td>
<td>Sand Shell Mixture</td>
<td>A.6</td>
<td>40</td>
</tr>
</tbody>
</table>
The resulting incompatibility with the IMS could be resolved in two ways. Firstly by extending the range of data codes defined in the DCG, for example to include bituminous mixes used outside North America. This however would result in a proliferation of material type codes with each country inserting its own non-standard code. A high degree of co-ordination would be required to resolve duplication in, for example, material type codes entered by all countries. The second, and preferred method, is for each country to use the standard DCG data codes and re-assign them to match data items with similar characteristics as illustrated in Table 6 for the UK. The few instances where additional codes are required should be limited to situations where there are insufficient numbers of DCG data codes to cover the range of data items used in the country of application.

The cases where modifications to the structures of IMS tables are required are limited. For UK applications, it is proposed not to modify pre-defined tables, but rather to create new UK specific tables instead. The objective for this is to limit the extent of incompatibility with the NIMS database in Washington. The data held in new tables will be used locally and not transferred to the NIMS. For example, pavement monitoring data given in DCG Table A.22 has to be extended to include all pavement condition variables measured using the HRM. In this case it is preferred to add a new table to the IMS to hold HRM data for use within the UK.

Currently, inventory data describing the characteristics of the LTPP trial sections in the UK are being collated prior to loading into the IMS at the University of Birmingham. The preliminary collation of inventory data gives a good example of the use of UK definitions for default DCG data codes. The minimum data set required in the DCG includes entries of codes for data items defined for North American conditions. For example, functional road classes, route signing, material type classifications, etc, have all been assigned equivalent UK classifications using the range of data codes in the DCG. In all cases, the SHRP ID (identification) numbers for trial sections in the UK have been assigned locally. It is expected that the entry of inventory data to the IMS will have been completed by the end of summer 1991 with pavement monitoring data to begin soon after. The first batch of data should be ready for transfer to the NIMS database in Washington by the end of 1991. This will largely comprise inventory data from trial sections constructed in 1990 and 1991. Pavement monitoring data will then be transferred thereafter on an annual basis.

7. PRELIMINARY ANALYSES PERFORMED ON UK SAMPLE DATA

Pavement monitoring data is intended, in the long term, to be used to derive performance relationships which will form the basis for new pavement design and maintenance methods. It is expected that data collection for this purpose will continue well into the next century. Ideally, pavement condition data needs to be accumulated for a number of years before analyses to determine pavement performance relationships can be conducted. In the meantime, it is required that statistical procedures for the analysis of LTPP data should be tried and tested before sufficient historical pavement condition data is accumulated during the LTPP studies. Consequently it has been necessary to conduct preliminary statistical analyses using existing data as well as simulated data.

Pavement condition data collected over the past thirty years from a number of trial sites built throughout the UK have been used to generate simulated pavement condition data in order to increase the volume of data available for the statistical analyses. The first task performed on the sample data was to load inventory and historical pavement condition data into the IMS at the University of Birmingham. This was accomplished using the modifications to the IMS described in Section 6 of this paper. In order to isolate the sample data from live UK data, pavement sections in the sample data set have been assigned a dummy state code of 99. This data will not be transferred to the NIMS database in Washington. The sample data included;
The sample data had to be pre-processed prior to import into the IMS at the University of Birmingham. This largely involved the preparation of data files which could then be imported into IMS using ORACLE SQL Loader commands. Many of the data items required in the IMS for pavement inventory were not available and therefore had to be simulated or deduced from other sources. For example, environment data (rainfall, temperature, sunshine and geological soil characteristics) were obtained from meteorological and geological records held elsewhere within the University.

Quality assurance checks on the sample data were limited to simple data validation because the pavement condition data had previously been measured and vetted according to standard UK procedures. Hence the validation was performed only to check that the data had not been corrupted during the transfer process. Typical data errors found were incorrect date entries and invalid pavement condition measurements (e.g. negative values). The quality assurance was also used to identify cases of missing data. A particular example is that of missing rehabilitation or maintenance data indicated by the sudden reduction in defect levels in one year. In such cases, blank rehabilitation records were inserted in the IMS database to indicate that some form of pavement repair or restoration had been carried out.

8. DEVELOPMENT OF STATISTICAL ANALYSIS PROCEDURES

The main objective for developing the statistical analysis procedures which will be applied to LTPP data is to have them proven and validated by the time sufficient data becomes available. A similar approach has been taken in the SHRP Data Analysis research contract (P-020) being conducted in the USA. The preliminary analyses have concentrated on the use of the statistical analysis software (SAS) to study the performance trends exhibited by the sample data. The primary objective for these analyses is to investigate alternative methods of deriving pavement performance relationships using data from the IMS.

The first stage of the analyses has been data familiarisation. To achieve this, a number of graphical plots of road condition have been carried out using CEPHALUS, a general purpose engineering graphics package with versatile curve fitting capability (Beaton, 1988). The objective was to identify independent variables and feasible transformations of these which are to be applied in the process of model building. Figure 7 illustrates the concept adopted in the data familiarisation stage. Figure 7(a) shows a line-curve joining all points in order to highlight the trend in rut depth progression with time. Figure 7(b) shows a similar plot of the cumulative traffic loading in ESALs with time. It may be seen that a high degree of correlation is indicated. The plot of the average annual temperature superimposed on rut depth progression shown in Figure 7(c) suggests that the two variables are not linearly correlated. Engineering judgement is required in this case to decide whether rut depth progression and average temperatures could be correlated in any way. If they are, then transformations of the two variables can be tested for possible correlation. For example, the annual change in rut depth progression might be expected to be correlated with average annual temperatures. Figure 7(d) suggests, however, that annual temperatures have little or no correlation with the annual change in rut depths and would not therefore be included in the performance model for predicting rut depth progression.
At a later stage in the development of predictive models, analysis of variance (ANOVA) techniques will be applied primarily to identify sets of independent variables, or their transformations, which have significant influence on the dependent variable. This will also assist with the assignment of the order in which independent variables are introduced in the process of model building. ANOVA will be used to study the effects of individual variables having eliminated the effects of other independent variables. Co-linearity between variables will be studied as the order in which correlated independent variables are introduced to the model will affect the sensitivity of the model.

In the final analysis, regression techniques will be used to determine the best fitting parameters of models which are in keeping with engineering understanding of the underlying problem of road deterioration. Regressions make implicit assumptions about the form of the relationships and this can be misleading. It is therefore important not to use regression analyses early in the development of performance models. An understanding of the structure of the problem, either from theoretical considerations of from observed characteristics, must be the basis for deriving the form of the models to be used. The function of regression analysis is then to fit parameters which best reflect the observed data.

The normal additive effects model illustrated in equation 1, is the starting point for regression analyses. This can be replaced by multiplicative effects models such as represented by equation 2 which cater for known interactions between independent variables.

\[
y = F(x_1) + F(x_2) + \ldots + F(x_n) \quad (1)
\]

\[
y = F(x_1) + F'(x_1)F(x_2) + \ldots \quad (2)
\]

where \( y \) = dependent variable (road condition)
\( x_1 \ldots n \) = independent variables

Other methods of model building, apart from various regression techniques, are currently being investigated. Of particular interest are probabilistic models which assume that random or natural variation is an inherent part of road deterioration. The problem currently faced with this approach is the lack of sufficient data to permit a study of the properties natural variation. The LTPP experiment, in general, is not structured to provide data in the depth required for such analyses. However, it may be possible, in future, to derive relationships of the form given in equation 3 below.

\[
y = F(x_1, x_2, \ldots, x_n) \pm P(x_1, x_2, \ldots, x_n) \quad (3)
\]

In the above equation, \( P(\ldots) \) denotes the effects of natural variation in the independent variables expressed in terms of probability. With this equation form, it would be possible, for example to estimate the upper limit of a pavement defect with a given probability of attaining this limit.

9. CONCLUSIONS

Road trials have been constructed and more are planned to study the long term performance of bituminous overlays on jointed concrete pavements. Measurements of the load transfer ratios at joints and cracks in concrete roads using the FWD are not affected by the magnitude of the applied load. Samples of materials have been extracted for laboratory performance related tests.
The statistical analysis work is at an early stage. The approach adopted relies on engineering knowledge to derive the forms of performance models prior to statistical analyses using the SAS package. This will lead to performance models which are largely based on current engineering knowledge and are statistically proven. Further analyses will investigate the causes of variation in road performance and attempt to model this using probabilistic models.

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11. ACKNOWLEDGEMENTS

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SHRP-NL: A Research Project Parallel to SHRP

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SHRP-NL: A Research Project Parallel to SHRP

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Abstract

The US Strategic Highway Research Program initiative was the major incentive for the Dutch Centre for Research and Contract Standardization in Civil and Traffic Engineering (CROW) in 1988 to set up a Steering Committee to investigate the need for a similar research effort in the Netherlands. Based on an inventory of the research needs performed by the Steering Committee under the Dutch state, provincial and municipal road authorities, a five year research program was drafted that focuses on the improvement of existing performance models, on remaining life prediction and on maintenance strategies. The research program on the one hand fulfills the specific needs of the Dutch road authorities as established in the inventory. On the other hand, it is very much in line with the research approach adopted for similar studies in the SHRP-USA program. This paper describes the SHRP-NL research program, the initial phase of the project and the linkage to SHRP-USA.

In essence, The Netherlands Strategic Highway Research Program SHRP-NL involves LTPP-studies on some 200 test sections throughout the Netherlands. Taking into account that the Netherlands has only one climatic zone (as opposed to the four zones dealt with in SHRP-USA) and that the SHRP-NL program only involves studies on asphalt concrete pavements, the number of test pavements corresponds well with the 1600 test sections included in SHRP-USA. This correlation follows directly from the use of the SHRP-USA system for experimental design in setting up SHRP-NL.

The major part of the test sections involved in SHRP-NL, are GPS-sections, which are monitored for their performance using Visual Condition Surveys and Falling Weight Deflectometer Tests. A limited number of SPS-sections is included to study maintenance effectiveness. The SHRP-NL LTPP-program is structured to be an independent experiment with a sufficiently large number of test sections to deal with the specific Dutch research needs. On the other hand, there is sufficient overlap with the LTPP-project of SHRP-USA to ensure a proper exchange of research findings.

The first phase of SHRP-NL involves the selection of the 200 test sections needed. From an initial set of 300 candidate sections submitted by the various road authorities, a first selection was made on the basis of the experimental design. Each of the cells in the three main experimental tiers was filled with at least two test sections, preferably selecting pavements covering the extremes within the particular cell. The sections thus selected had a length of typically 1-2 km. On these sections, LaCroix deflectograph measurements were performed to obtain homogeneous 300 m sections, which will be the final SHRP-NL test sections. The LaCroix deflectograph was used for these inventory measurements rather than FWD-equipment because of the much higher density in measurements obtained with the LaCroix. In the remainder of the project, Dynatest FWD-equipment will be used for deflection measurements.
SHRP-NL: A RESEARCH PROJECT PARALLEL TO SHRP

by

Govert T.H. Sweere

paper submitted for

INTERNATIONAL CONFERENCE

STRATEGIC HIGHWAY RESEARCH PROGRAM
AND TRAFFIC SAFETY ON TWO CONTINENTS

GOTHENBURG, SWEDEN
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Swedish Road and Traffic Research Institute

TRB
U.S. Transportation Research Board
1. INTRODUCTION

As was the case with the AASHO Road Test of the late nineteen fifties, the Strategic Highway Research Program carried out in the USA in the years 1988-1993 will set many of the standards in pavement design and management for the next decades to come. SHRP is much more versatile than was the AASHO Road Test, since it covers a larger range of pavement structures and -perhaps more importantly- the whole range of climatic conditions in the USA from Alaska to Hawaii. Furthermore, SHRP's data and results will be much more accessible than those of the AASHO Road Test thanks to the extensive use of computer technology throughout SHRP. Because of its versatility and the accessibility of data, SHRP's results are bound to be used not only in the USA but worldwide.

Realizing that its results were to be used worldwide, SHRP has from the start strongly encouraged international participation in its major research effort. Foreign nations have been invited to assign test sections in their own country and provide the performance data of these pavements to SHRP for incorporation in SHRP's data base and analysis. For SHRP, this results in a even more versatile data base and analysis. The major beneficiaries of international participation, however, are the foreign nations themselves. Submitting data from local pavements to SHRP for inclusion in data base and analysis will give them the opportunity to translate SHRP’s results to their local conditions.

For international participation in the LTPP program, SHRP distinguishes three possible approaches, denominated as Integral, Interface and Parallel. Integral participation means that conditions and test methods in the participating country are equal to those in the USA. The Canadian C-SHRP project is the prime example here: part of the 42 Canadian test sections form an integral part of the SHRP LTPP program. Interface participation in SHRP indicates that a given country submits a number of test sections to SHRP, often from a current national experiment of gathering performance data. Parallel participation in SHRP involves performing a full scale national SHRP-study and making sure that sufficient overlap exists between the two experiments to allow for proper exchange of data and research findings. Obviously, the latter form of participation is the most comprehensive one. Performing a national SHRP-study requires a large number of test sections to completely fill the experimental design factorials, whereas the other forms of participation may involve only test sections in selected cells of the factorials.
This paper describes the SHRP-NL project, which is an example of parallel participation in SHRP. The initiation, structure and first phase of the SHRP-NL project are discussed in detail. Further, the linkage to SHRP-USA is discussed.

2. PAVEMENT MANAGEMENT IN THE NETHERLANDS

The Netherlands is a small, densely populated country. Some 15 million people live on an area of 33,000 square kilometres, which is approximately the combined size of the states of Massachusetts, Connecticut and Rhode Island in the USA. For comparison, these states have a combined population of some 10 million people. In addition to the high population, the strong dependence of the Dutch economy on trade contributes to a very intensive use of the road network. The port of Rotterdam and Amsterdam's Schiphol Airport are main gateways to Europe, with the resulting extensive road based transportation of people and goods. The extensive use of the road network contributes to accelerated deterioration, while on the other hand the strong dependency of the economy on trade requires good pavements with a minimum of traffic hinder caused by maintenance work. Given these conditions, pavement management has been a prime topic of research and implementation in the Netherlands for a number of years.

Pavement management is performed in the Netherlands at four administrative levels. The main road network comparable to the interstate system in the USA is managed at the national level by the Rijkswaterstaat (the Public Works Department of the Ministry of Transport and Public Works). The twelve provinces in the Netherlands are the next level of road authority: they manage all major arteries not covered by Rijkswaterstaat. The third level are the municipalities, managing both urban roads and non-urban roads of local importance. The fourth level of pavement management is typical for the Netherlands: a number of "waterschappen" (Water Control Boards) is responsible for the management of rural roads.

3. INITIATION OF SHRP-NL

The Dutch Foundation CROW (Centre for Research and Contract Standardization in Civil and Traffic Engineering) performs a stimulating and coordinating role in pavement research in the Netherlands. All four levels of road authorities participate in the Foundation, while they also contribute to the funding of research projects performed under supervision of CROW.

The US Strategic Highway Research Program initiative and its quest for international participation were the main incentives for CROW in 1988 to set up a Steering Committee to investigate the need for participation of the Netherlands in SHRP. The Committee performed an inventory of research needs in the field of pavement management at all four administrative levels. Although
pavement management systems were developed and implemented in the Netherlands as early as 1980, all four levels of road authority clearly favoured participation in SHRP. Such a participation would first of all open the way for application of SHRP's results in the Netherlands, but moreover could produce an update of current management systems and a differentiation of current systems to cater for the specific needs of all four levels of road administration.

The overall conclusion of the inventory was twofold. Firstly, active participation in SHRP was strongly recommended. Secondly, the Steering Committee concluded that to best solve the pavement management problems identified by the inventory, a full scale, national Strategic Highway Research Program focusing on Long Term Pavement Performance would be required. Hence, the decision was made to participate in SHRP with a national research project of sufficient magnitude to independently solve Dutch problems in pavement management. This project is structured in a way very similar to the LTPP-study in SHRP-USA, thereby allowing for extensive exchange of data and research findings.

After reaching the above conclusion, the Steering Committee commissioned a joint-venture of three major Dutch pavement management consultants to draft the research plans for SHRP-NL. The resulting blueprint for SHRP-NL was published in 1990 (Grontmij et al., 1990).

4. STRUCTURE OF SHRP-NL

4.1 General

Of the four major topics dealt with in the SHRP-USA program (Asphalt, Concrete and Structures, Highway Operations and Long Term Pavement Performance), SHRP-NL only deals directly with Long Term Pavement Performance of flexible pavements. A comprehensive set of test sections is monitored for distress over a period of five years. Relevant materials testing is performed as part of this LTPP-project. Materials research such as development of new asphalt concrete mix design procedures is performed in the Netherlands by separate Working Groups of CROW (Hopman et al., 1991). Obviously, links have been established between these laboratory test programs and SHRP-NL’s LTPP data base.

4.2 Technical background

The SHRP-NL LTPP-project focuses on pavement management. Here, the pavement authority is faced with basically two questions: is rehabilitation required for a given pavement and -if so- what rehabilitation is required. Annual broad Visual Distress Surveys at the network level are the authority's main tool for establishing which pavements need rehabilitation, while detailed Visual Condition Surveys and Falling Weight Deflectometer tests are used for assessment of the pavement condition at the project level.
To further develop these pavement management tools, SHRP-NL will perform Long Term Pavement Performance studies on two main groups of test sections. The first group involves sections that have not had major rehabilitations to date, and will not have such rehabilitation in the next five years. On 144 of such sections, the development of pavement distress under the influence of traffic and climate will be monitored on an annual basis. The second group of test sections involve pavements that do get major rehabilitation in the initial phase of the SHRP-NL project. Here, the condition of the pavement prior to rehabilitation, directly after that and further on an annual basis will be monitored. Two types of maintenance are involved here: structural maintenance (asphalt concrete overlays) for pavements showing fatigue-type of distress and wearing course maintenance for pavements with distress modes such as ravelling and rutting. In all, this part of SHRP-NL involves studies on 120 test sections.

The pavement management systems currently being used incorporate models for the development of visual pavement distress such as cracking, rutting and ravelling. Having established the present state of distress of given pavement, the road authority can use the relevant distress model to predict how much longer the pavement will last until major rehabilitation measures are required. An initial set of such deterioration models was developed by the Working Group R1 "Pavement Management Systems" of CROW in the 1980's (Koning and Molenaar, 1987). Having been developed on municipal roads, this system is not fully applicable on the major arteries managed by, for instance, Rijkswaterstaat. This organisation, therefore, uses different performance models in its own Pavement Management System IVON.

From its yearly measurements on 144 test sections, SHRP-NL will update the current pavement distress models and differentiate them with respect to pavement category. Starting point in this update will be the current models themselves and the distress models developed by SHRP-USA in the GPS-1 (asphalt concrete on granular base) and GPS-2 (asphalt concrete on bound base) parts of the LTPP-project.

The 120 test sections involved in the pavement rehabilitation part of SHRP-NL are divided into two sub-groups. Structural rehabilitation (asphalt concrete overlays) is studied on 72 sections, while wearing course rehabilitation is studied on 48 sections. The latter group involves rehabilitation measures such as single and double surface treatments, thin stone mastic overlays and slurry seals. From its yearly performance measurements on these sections, SHRP-NL will compare the effectiveness of the various rehabilitation measures, given the condition of the pavement prior to maintenance. Again, the results of SHRP-USA will form one of the starting points in this analysis. The LTPP-data and analysis from the GPS-6 (AC-overlay on AC) part of the LTPP-program will be used in the structural rehabilitation study, while results from the Highway Operations program (SPS-3, preventive maintenance effectiveness) will be used in the study of wearing course maintenance.
5. **SHRP-NL TEST SECTIONS**

5.1 Experimental designs

The 144 test sections in SHRP-NL used for the development of visual distress models were selected using an experimental design very similar of that of SHRP-USA's projects GPS-1 and GPS-2. Key factors in selection of the test sections are type of subgrade, type of base, asphalt concrete thickness and traffic loading. Two types of subgrade are distinguished, being clay/peat and sand. For the base course, four factor-levels are distinguished: no granular base ("full depth asphalt") granular base, self-cementing granular base and bound base. Asphalt concrete thickness is incorporated in the experimental design at three levels: \( t < 14 \text{ cm} \), \( 14 \text{ cm} < t < 24 \text{ cm} \) and \( t > 24 \text{ cm} \). Traffic intensity is dependent on asphalt concrete thickness, ranging from less than 2,500 vehicles per day (total cross section) for the weakest constructions (asphalt thickness < 14 cm) to more than 60,000 vehicles per day for the strongest constructions (over 24 cm of asphalt concrete on bound base). As was done in SHRP-USA, traffic was defined in terms of rate (vehicles per day) rather than accumulated standard axle loads to provide a good distribution of both pavement age and traffic. Finally, in the experimental factorials obtained using the above key factors, separate columns are entered for normal dense asphaltic concrete wearing courses and for pervious asphalt wearing courses. This is deemed necessary since visual distress surveys on the two types of wearing course yield different results. Therefore, the data for the two types of wearing course cannot be mixed in the analysis and, hence, separate sets of data are required. In total, the above experimental design leads to 144 test sections.

The 120 test sections for the rehabilitation effectiveness study were selected using experimental designs similar to those used in the GPS-6 (AC overlay on AC) and SPS-3 (maintenance cost effectiveness) studies of SHRP-USA. For the overlay study, key factors are visual condition prior to overlay, surface curvature index SCI from deflection measurements, the stiffness of the pavement foundation (again obtained from deflection measurements) and depth of cracking. Factor boundaries are medium/poor for visual condition, low/medium/high for SCI, low/high for stiffness of pavement foundation and through-wearing-course-only/through-total-AC for depth of cracking. Again, separate columns are incorporated in the experimental design for dense asphalt concrete and previous asphalt concrete wearing courses. In all, this part of SHRP-NL involves 72 test sections.

The maintenance effectiveness study for wearing courses involves four widely used maintenance measures, being single and double surface treatments, stone mastic asphalt and slurry seals. For each treatment, SHRP-NL incorporates 12 test sections. Key factors in the experimental design are visual condition prior to maintenance (at levels medium and poor) and traffic intensity (at levels high, medium and low). In all, 48 test sections are involved in this part of SHRP-NL.
5.2 Selection of test sections

A total of 31 pavement authorities is involved in the SHRP-NL project through submitting test sections for the project. All four levels of road authority mentioned in chapter 2 take part in the project. For Rijkswaterstaat, all 12 districts participate, as do all 12 provinces. The major cities of Amsterdam, Rotterdam and The Hague have submitted test sections, as did the municipalities of Oss, Staphorst and Het Bildt. Finally, the Water Control Board of West-Friesland is involved.

The first step in selection of the test pavements is the submission of candidate sections to SHRP-NL by the road authority. Based on data from the authority’s files, a first selection is then made, in which the candidate section is allocated in the relevant cell of the experimental factorial. The next step in the selection process is the search for a homogeneous part in the candidate section. The candidate section submitted to SHRP-NL typically has a length of 800 - 1200 m. Out of this, the final SHRP-NL test section with a nominal length of 300 m is selected on the basis of homogeneity of construction using the Lacroix deflectograph (Hoyinck and Van Zwieten, 1991). The Lacroix deflectograph was selected for this part of SHRP-NL since it gives a much higher density of measurement at the same cost than does the Falling Weight Deflectometer. Once the final 300 m section have been identified, all further deflection testing will, for sake of conformity with SHRP-USA, be done using Dynatest 8000 Falling Weight Deflectometer equipment.

After selection of the homogeneous sections, the final SHRP-NL sections will be duly marked and identified with the Section Identification System set up by SHRP-USA. Then, coring and sampling will be carried out, followed by the first round of Visual Distress Surveys and FWD measurements.

6. SHRP-NL CURRENT STATUS

At the time of writing of this paper, the selection process for the SHRP-NL test section was still in progress. A first block of 165 test candidate sections have been identified, mainly from the provincial and municipal road authority level. Lacroix deflectograph measurements have been carried out and the first final sections have been identified. The second block of candidate sections from the Rijkswaterstaat level are in the process of submission to SHRP-NL.

7. RELATIONSHIP TO SHRP-USA

As noted throughout this paper, SHRP-NL has drawn heavily on SHRP-USA in setting up its experiment. The experimental designs used by SHRP-NL resemble closely those of the LTPP program of SHRP-USA. The whole principle of the LTPP-projects too is the same. Basically, Long Term Pavement
Performance data are gathered over a long period of time on a well designed set of test sections. The data gathered are to be used in a later phase of the project in the development of pavement distress models, which form the main tool for pavement authorities in allocation of their resources for pavement maintenance. For storage of data, SHRP-NL uses the Information Management System developed by SHRP (Jamsa et al, 1990).

The difference between SHRP-NL and SHRP-USA mainly lies at the detailed level. Obviously, units are a problem: SHRP-USA uses American Customary units in its data base and analysis, whereas SHRP-NL uses SI-units, as do all the other international participants in SHRP. Further, testing devices may differ. For instance, SHRP-USA uses the K.J. Law-profilometer whereas SHRP-NL uses the Canadian developed Automatic Road Analyser ARAN. For visual distress, the same modes of distress are identified, but the levels at which a given distress mode is considered to be, for instance, "severe" are different. For distress surveys, SHRP has developed its own system (SHRP,1990) whereas SHRP-NL uses the system that was implemented in the Netherlands in the 1980's (SCW, 1987). Other measurements may be almost identical: the procedure for FWD measurements used by SHRP-NL is a copy of that of SHRP-USA, the only difference being that the load levels are set at kN values rather than lbs.

In all, the data gathered by SHRP-USA and SHRP-NL will not be fully exchangable due to the difference in units, test methods and criteria. The overall results of the two projects will, however, certainly be very much in line, thanks to the close agreement in research goals, overall research approach and principles of testing. The differences at the detailed level of units, test devices and criteria cannot and should not be circumvented. Implementation of the research results requires adaptation to locally existing pavement management practice throughout the whole project. Deviation from the already implemented systems in the Netherlands would give SHRP-NL an even closer agreement with the USA data, but would in the end hamper the implementation of its results. Thanks to its size, SHRP-NL will be able to fully translate the USA research findings to local circumstances, without loosing the connection with current practice of pavement management in the Netherlands.

8 CONCLUSION

To SHRP's quest for international participation, the Netherlands has responded strongly by creating its own parallel SHRP-NL project. The most comprehensive form of participation was deemed necessary to fully profit from SHRP's research findings and, more importantly, solve problems currently encountered by pavement authorities in the Netherlands. These problems were shown in the inventory performed to be very similar to those addressed by SHRP-USA. Yet, the pavement management systems already implemented do not allow for a straightforward copying of the USA results in the Netherlands. Parallel participation in SHRP ensures sufficient local data and involvement to implement SHRP's results in current practice in the Netherlands.
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Structural Assessment, Performance and Economic Maintenance of Minor Roads

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STRUCTURAL ASSESSMENT, PERFORMANCE AND ECONOMIC MAINTENANCE OF MINOR ROADS

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- Abstract -

* This three year study was funded by the Science and Engineering Research Council and was undertaken in collaboration with the Hertfordshire County Council and the Transport and Road Research Laboratory. The objectives were twofold: to study the nature causes and rate of deterioration of minor roads and to identify warning levels for structural maintenance work to be effected on minor roads.

* A matrix approach was adopted to categorise the physical characteristics of the sites to be investigated. It has been hypothesised that the causes of deterioration were road width (overriding), construction, drainage, edge support and traffic loading. From these factors a matrix with 20 cells was produced. A search was made for roads showing some degree of deterioration which would likely yield changes in condition over the study period and a few with no deterioration to act as a control. The study was confined to an area of approximately 600 km² to the East, West and South of the Polytechnic. Ultimately 74 sites, each comprising a 100 metre length, were identified.

* At each site, road cores were taken to determine the construction and composition. The underlying CBR was established using the Dynamic Cone Penetrometer. Observations were made at 6 month intervals using both manual visual inspections and mechanical surveys. The former consisted of the CHART survey augmented by DECHART in which a detailed mosaic was built up of conditions relating to the edge, rutting cracking et al. The mechanical tests consisted of measurements by the High Speed Road Monitor (HRM) at 6 or 12 month intervals and the Deflectograph (2 sets of observations on half the sites) and finally the Falling Weight Deflectometer. The last mentioned was used to investigate the stiffness of the pavement in longitudinal and transverse directions in relation to changing condition.

* Deterioration rates were established for a number of conditions i.e. rutting and cracking for sites in each cell of the matrix. Detailed statistical analysis using regression and principal components techniques were used to establish the main causal factors of minor road deterioration. Though Non Destructive Testing methods used for assessing condition on major roads can be applied to the minor road network, the changes recorded at 6/12 month intervals were within the repeatability range.

* A further research grant has been obtained from SERC to continue the research with collaboration and financial support from four counties in various parts of the UK and a London Borough. The emphasis is changed to the development of cost effective maintenance strategies for minor roads based on whole life cost concepts and the deterioration rates established in the previous research programme and also to be obtained from the different regions in the UK.
1. OVERVIEW

1.1 This paper conveys the research being undertaken at Hatfield Polytechnic into the structural and economic maintenance of minor roads. Work commenced in April 1986 and was initially funded by a grant of £80k from the Science and Engineering Research Council (SERC) with the active collaboration of the Transport and Road Research Laboratory (TRRL) and Hertfordshire County Council (HCC). This work focused on the measurement of changing road conditions, successfully relating these to those factors believed a priori to induce wear; it was completed in early 1990.

1.2 The author subsequently obtained further funding through a grant from SERC £50k circa and industrial support £20k circa to pursue the research but with a change of emphasis to devising economic models for highway maintenance strategies based on the previous and current study. This work commenced in November 1990 and is being undertaken jointly with four county councils, one London Borough and in collaboration with WDM Ltd, the manufacturers of machines for highway condition assessment. (Fig. 1). Part I of this paper conveys the site selection, experimental methodology and results of the previous study whilst Part II sets the context and research thrusts of the current work and the expected outcome at its conclusion in late 1993/early 1994. A third part offers a brief re-appraisal of the deterioration rates, the reformulation of the causal matrix and some concluding remarks.

1.3 Whereas major road networks have been designed and are increasingly subject to further research, the minor road network by comparison has always been under researched and underfunded in most developed economies. The Strategic Highway Research Programme (SHRP) by definition heightens this fact. The TRRL the agent yet independent research arm of the UK's Department of Transport (DTp), is no longer a collaborative partner in this further research, though it is undertaking on a contract basis half yearly mechanical surveys using the High Speed Road Monitor (HRM) through its Technology Transfer Unit. WDM Ltd are sub contractors for the survey work using the deflectograph.
PART I - THE INITIAL STUDY

2. OBJECTIVES AND HYPOTHESES

2.1
The objectives of this research programme were twofold:

a) to study the nature, causes and rate of deterioration of minor roads;
b) to identify warning levels for maintenance work on minor roads.

As a result of several consultations with practising engineers, staff from Hertfordshire County Council and the TRRL, three hypotheses were formulated, which were the basis of the doctoral research programme by Kalombaris (1), namely that the:

i) principal contributory factors affecting the structural integrity of minor roads were road width, the traffic, drainage, edge support and construction.

ii) rate of change of road condition might be determined on minor roads utilising non-destructive test (NDT) methods already perfected for major roads;

iii) rate of change of road condition, by NDT methods and visual inspections, will give an early indication of the onset of critical conditions.

3. STUDY METHODOLOGY

3.1
The initial research programme effectively comprised four phases, the first being identification of the research sites and carrying out basic surveys. The second phase was the measuring of road condition at six-monthly intervals by mechanical and visual means and the third was the detailed statistical analysis of changes in condition. The culmination of the investigation was the determination of deterioration rates and their principal causes.

3.2
The factors believed to affect minor road deterioration were used to build up the causal matrix (Fig 2). An attempt was made to locate the 100m length sites to fit all the cells of the matrix but certain cells were not occupied because sites with the required combination of characteristics were not found. In all, 74 sites were chosen on the basis that most would yield significant changes in condition over the study period.

3.3
A coring programme was undertaken, providing information on the road construction; usually two cores in between the wheelpaths, on either side of the centreline. These were examined and where significant differences occurred, a third core was taken. Subgrade condition was investigated to provide information about the strength (bearing capacity) of the underlying soil concurrently with the coring using a Dynamic Cone Penetrometer (DCP).
3.4 Visual condition surveys were carried out at six month intervals based on the CHART assessment system (2) for structural maintenance needs developed at TRRL. An extension of CHART is DECHART (Detailed CHART) whereby defects were recorded in more details in pictorial form. The defects recorded were: wheeltrack cracking (single, multiple), transverse cracking, patching (sound, unsound), crazing (fine, coarse), fretting (slight, severe), potholes, rut depth, bleeding and edge condition.

3.5 At the onset of this study, it was hypothesised that Non Destructive Testing (NDT) measurement, already perfected for major roads, could be used to determine the rate of deterioration of minor roads. The HRM was used for surveying the surface condition at normal traffic speeds, using laser sensors to produce information on longitudinal profile, rutting and texture depth (macrotexture). Threshold values for profile have been established for major roads, but nothing similar existed for minor roads. The High Speed Road Monitor (HRM) surveyed all seventy four sites on five separate occasions. The Deflectograph was used to survey half the sites twice, over a one year period. The Falling Weight Deflectometer (FWD) was used to survey ten sites twice over a one year period at 10m intervals along the wheelpaths and also on two cross sections within each site, thus allowing variation in stiffness moduli of the pavement layers to be assessed.

3.6 Traffic flows and axle loading information were essential and Manual counts were carried out on three occasions in the morning peak period (07.30 - 09.30). Pneumatic tube counters were also laid out at 18 locations which effectively covered 30 sites to obtain more accurate daily flows. A portable weighbridge (Weighman) was borrowed from the TRRL and used to obtain the axle spectra at twenty six sites.

3.7 A software package (OPEN ACCESS II) was used to develop the database, into which all information was fed. The design of the database enabled all information to be stored in chronological "stacks" to calculate deterioration rates. The database was linked via a communications package (KERMIT) to a statistical software package (MINITAB) used for the statistical analysis and graphical output. Average deterioration rates were established for each cell of the matrix and multi step regression and principal components techniques respectively were used to generate deterioration equations containing the main factors causing wear and their weighting to overall deterioration.

4. RESULTS

4.1 Construction Characteristics

The results from the coring programme were very variable, the resulting cores being analysed for composition where enough material was obtained. The range of materials recovered was such
that no conclusive results could be obtained. The bearing capacity of the subgrade was investigated and values for CBR calculated from the DCP results (para 3.3). Very low calculated CBRs could be due to weathered subgrades, whereas very high calculated CBRs could be due to the DCP hitting large stones.

4.2
It was expected that on sites which had weak subgrades and were heavily trafficked, a substantial thickness of bound material would be found, to compensate for the detrimental effects of the above factors; the minor roads surveyed appeared not to be constructed on the basis of subgrade strength or traffic carried. However, Fig 3 indicates that with increasing traffic, minor roads broadly have a greater thickness of bound material.

Machine Surveys

4.3
The longitudinal profile of the sites using the HRM were expressed in profile deviations ($6_3$) from a 3m moving average. Jordan and Young (Ref 3) and Cooper (Ref 4) established that for major roads, a subjectively acceptable ride was observed on surfaces with $6_3$ values less than 1.2mm. In a final year degree honours project at Hatfield Polytechnic, Lee (5) obtained a comparable figure for minor roads of about 3mm. By this standard, the vast majority of the minor road lengths surveyed had, not surprisingly, an unacceptable ride given that the sites chosen for this study were at the worst end of the condition scale. Repeatability studies were undertaken at ten sites to provide a guide as to the variability to be expected from this source. It was established that there was a 95% probability that any measured values of $6_3$ exceeding the boundary given by the line: $y = 0.071(6_3) + 0.093$ where $6_3$ is the mean value over each of the test sites, are not attributable to repeatability errors. Rates of change at all sites was determined over the study period and at most sites it was small, (ie less than 0.30mm per year). The change in texture depth observed over the three years, was found to be statistically significant at only eight sites and of the order 0.5mm Sensor Measured Texture Depth (SMTD) per year.

4.4
During the Deflectograph surveys, in November 1987 and November 1988, the operating temperatures at 40mm below the road surface were 7-9°C. These were too low to standardise the measured deflections to standard 20°C deflections without introducing errors. The measured deflections were compared directly to assess change in structural conditions. Generally there was no significant change in deflection over the one year period. At the few locations where significant changes did occur, they correlated well with changes in the visual conditions. Furthermore, the absolute values of deflection were high by major road standards; averaging for all thirty five sites 0.53mm. The deflection surveys are to be repeated in November 1991 at those sites not lost (Fig 2).

4.5
The stiffness moduli of the bound and unbound layers were computed
from FWD measurements along and across the 10 sites surveyed. Again broadly speaking, there were no significant changes over the year that elapsed between the two sets of surveys. Where changes were observed, however, or weak layers were identified, significant changes in visual condition were also observed. Where the construction of the sites was very variable, it proved difficult to replicate their behaviour using a two layer model. In some cases the thickness of the bituminous layers were below the recommended minimum of 75mm for FWD stiffness analysis purposes. The two layer model however proved adequate for comparing relative performance of the test sections. FWD measurements will be repeated at those sites which have not been lost.

Traffic Flows and Axle Loadings

4.6 In line with the research hypothese (para 2.1), it was decided to use two parameters to assess the effect of traffic: total traffic in vehicles per weekday and the commercial traffic expressed in standard axles per weekday. The traffic flows ranged from 50 vehicles per day (v.p.d) to 9000 v.p.d in both directions. To assess the effect of commercial vehicles, the "standard axles" concept was utilised. On the sites where the Weighman operated (para 3.6) this was fairly straightforward, as the data logger stored all the axles that went over the mat in predetermined axle weight categories. The equivalence factors recommended in LR910 (6) were then used to correct these axle weights into standard axles for each category. According to Addis and Whitmarsh (7), the damaging effects of commercial vehicles on relatively weak pavements are underestimated.

4.7 On the sites where axle load spectra were not available, an approximation method was used as outlined by Currer and O'Connor (established by the manual counts) into standard axles by multiplying the former by an average vehicle damage factor recommended by the above authors. The average vehicle damage factor of 0.65 recommended for lightly trafficked roads in the year 1988 was used. The results ranged from virtually zero standard axles to 804 standard axles per day in both directions, virtually 400 standard axles/day uni-directional for the wider minor roads.

Deterioration Rates from Visual Condition Surveys

4.8 Using the values for the defect observed during the first CHART and DECHART survey (Phase I - October/November 1986) as datum, the changes observed during subsequent surveys were input into the database and rates of change of defects for all sites in each cell of the matrix, were established (Fig 4a and b). The major conclusion that can be drawn was that sites not effectively drained deteriorate faster in most cases than effectively drained ones. The trends in deterioration are linear for all practical purposes as indicated in the example of Fig 5. Care must be exercised in interpreting the results because of the sample sizes, even in the better covered cells of the matrix, (Fig 2) more test sites would
have increased the project cost prohibitively. It will be noted that construction (including subgrade) appears only as the 3rd axis in the matrix and has not been disaggregated; this is elaborated on later in the paper (para 6.1).

Statistical Analysis

4.9
To ensure the statistical analysis was realistic, engineering judgement was applied to identify those causal factors believed to contribute significantly to the measured defects (Fig 6). These relationships were used in the stepwise regression model and the principal components analysis. An example of a multiple regression output can be seen in Fig 7 and from which a typical predictive equation was established, eg: \[ Y = 8.710 - 6.20 X_1 \] where \( Y \) is change in edge defect in metres per 6 months and \( X_1 \) is edge support (0 unsupported, 1 supported), and 100\( r^2 = 54 \). This indicates that for the minor road lengths observed, an unsupported road would show on average a change of 8.7m of edge defect per 100m road length every 6 months, whilst on a road with a supported edge, a much smaller change of 2.1m of edge defect over the same road length would occur, provided no other causal factors were contributing. Introducing a secondary causal factor, the same predictive equation would change into: \[ Y = 16.24 - 9.7 X_1 + 0.007 X_2 \] where \( X_1 \) is the edge support (0 or 1) and \( X_2 \) is the daily commercial vehicular traffic, in equivalent standard axles per day. The associated coefficient of determination (100\( r^2 \)) would then increase to 65. Adding a third and fourth independent variable would only increase the coefficient of determination to 72 and 74 in turn.

4.10
From the stepwise regression, a series of predictive equations for deterioration rates per half year was drawn up (Fig 8). It is evident that traffic and the thickness of bound material have the most appreciable effect on several defects. The analysis was extended further by utilising the principal components analysis technique to assess the relative weighting of defects in establishing an overall index of deterioration. It was found that up to 71% of the total variance in the change in condition data was explained by a single principal component. Edge deterioration, potholing and rutting were found to affect this principal component more than cracking, fretting, crazing, profile and texture (Fig 9).

Warning Levels

4.11
Over the initial study period, 17 out of the original 74 sites were subject to some maintenance treatment. A correlation was attempted between the visual condition of these sites prior to maintenance and the maintenance recommendations made in the Local Authorities Associations' Codes of Good practice (8) but no relationship was identified. It was noted however, that sites were twice as likely to be maintained where they had poor drainage, lacked edge support and were narrow. With the advent of the second study this general statement no longer holds as will be referred to below.
Limitations of the initial study

4.12
The writer obtained in 1990 further funding to continue the research but with a significant change of emphasis and objectives. The shortcomings of the first investigation were that only a small proportion of the sites were in urban areas, that no adverse weather was experienced over the period 1986 to 1989 in relation to road conditions (there were gales affecting trees) and that no observations were made on the probable effect of water table position on pavement performance. The new study addresses the first and last mentioned in that in two authorities, Barnet and South Yorkshire (Sheffield), the sites selected are chiefly in the urban environment and hence edge supported and drained. In Sheffield piezometers are being installed at some sites to observe water tables and this will also be undertaken by the research team at Hatfield at some of the remaining Hertfordshire sites.

PART II THE CURRENT STUDY

5.1
At the commencement of the second study it was evident that of the original 17 sites lost to June 1989 (phase VI) a further 12 had been subject to maintenance treatments of some kind the next time surveys were carried out in October/November 1990 (phase VII). Further inspections in March of this year revealed the loss of more sites (restrictions on treatment other than for overt safety reasons were lifted at the conclusion of phase VI).

5.2
At the recent steering group meeting, the author recommended the abandonment of the four sites with PQ concrete road bases and three more sites where conditions were so bad that considerable visual recording difficulties were apparent. Of the original 74 sites in Hertfordshire only 33 now remain at the time of writing. Given the collaborative nature of the new investigation three authorities (South Yorkshire, Shropshire and Barnet) are undertaking the visual condition surveys using their own staff whilst the Hatfield team are monitoring the Hertfordshire and Buckinghamshire sites.

5.3
All teams are recording the DECHART defects to a set of definitions as agreed by the Steering Group chaired by Director of Transportation Hertfordshire and including a representative of all the collaborating partners. The study director (the writer) advanced the suggestion that two defects might be dropped as a result of measurement difficulties and poor 100r values (para 4.9) namely "Slight fretting" and "bleeding". It was however decided to retain these as indicators of surface condition and to add "areas of chipping loss" as another surface condition variable. In the case of the Buckinghamshire sites, these will be subject to skid resistance measurements using its Griptester. The author is also looking to introduce this aspect of sideways force coefficient into the Hertfordshire sites.
Aim and Objectives

5.5
The overall aim of the new study is:

to develop maintenance strategies for minor roads based on warning/intervention levels within existing systems thereby weighting the various pavement defects.

In pursuance of this aim, the specific objectives will be:

1) to undertake a review of warning and intervention levels for maintenance treatments and obtain unit casting for remedial work from a number of highway authorities

2) to compare deterioration rates obtained from the worst condition sites with those obtained from the annual random site observations from the National Road Maintenance Conditions Survey

3) to observe the effect of variation in water tables on pavement performance

4) to monitor sites in both Hertfordshire and the collaborating authorities to isolate possible regional variations in road deterioration through climatic variations.

Systems approach and proposed outcome

5.6
The proposed outcome of the investigation is best illustrated by means of a flow diagram (Fig 10) indicative of a system approach. First a "profile" of minor roads for all cells of the matrix (A to S) for each measured defect (1 to 12) will be obtained. Next a set of standards for intervention levels will be set mindful of structural, safety and service (ride) factors within a set time horizon (for whole life cost purposes). The costing will then be based on inputting deterioration rates established from the previous and current investigation and the unit costs for various treatments. It will be possible to undertake a sensitivity analysis based on "what if the time horizon is taken as x years rather than x + 5 years and intervention level for deformation (rutting) is taken as 20mm rather than 25mm" scenarios.

PART III - A REAPPRAISAL OF DETERIORATION RATES AND THE FORMULATION OF THE CAUSAL MATRIX

6.1
In concluding this paper further reference is made to the original matrix (Fig 2) and the lack of definition on pavement construction and subgrade support. In consultation with the author, the researcher conducting the present study carried out a preliminary reexamination of the rates of change in condition for the concluded study with explicit reference to the above parameters. The initial effect of this is to spread more evenly the original sites in the matrix from about 7 cells (Fig 11) at 10 per cell to 10 cells at 7 per cell. By way of example rates of change determined from graphs of condition for three defects are shown in Fig 12.
6.2
The matrix inputs the variables; traffic, bound material thickness, (the most significant variables isolated from the previous study) CBR and drainage, leaving out only the road width and edge support conditions. On the most cursory of inspections, about half of the rates appear logical i.e. "the right way round" when comparing the various pairings in the matrix, a fifth to a quarter give near identical rates and the remainder are clearly "the wrong way round". At this stage it is necessary to cross correlate with the number of sites within each of the cells.

6.3
By way of example and effecting comparisons in the horizontal dimension, for < 10% CBR and effective drainage, WTC (wheel track cracking) is 5.65 and 6.15 m/pa for < 1500 vpd, for > 150mm and < 150mm thickness of bound materials respectively; the sample sizes are 6 in both cells. Here the rates are the "right way" round. An example of "wrong way round" can be found for rutting where for > 10% CBR and "not effectively drained", rutting is less at 1.6mm/pa for a pavement of less than 150mm thickness than for one of greater than 150mm thickness at 2.63mm/pa where however the sample sizes are only 2 and 4 respectively.

Concluding Remarks

7.1
The fact that the minor roads under investigation have never been designed in the engineering sense militates against the establishment of logical deterioration rates, compounded by the small number of sites in the cells of the matrices however formulated. That said the initial research investigation has pointed to - upon a statistically sound basis - traffic loading and thickness of bound materials are to date the principal variables inducing deterioration in most of the defects measured. On the basis of the limited information available at the conclusion of the previous study the second objective: to identify warning and intervention levels proved inconclusive.

7.2
Naturally in pursuing the current study it is hoped to overcome some of the limitations arising from the original investigation utilising more condition data analysed by the methods, techniques and software used in the initial study. In the final analysis what comes out is the quality of the effort which is input, not least the painstaking recording of condition data both by manual (visual) inspections and machine surveys. Though a common basis of recording has been agreed, there will still be the need to interpret conditions and make engineering judgments on site. For this reason it is imperative that the same person does the recording in the respective authority areas. To this end the techniques have been transferred from the original research team to the collaborative partners by joint working on site and by the agreed definitions (para 5.3).
Acknowledgements

The author wishes to thank Mr K R Youngman and Mr J F Potter of the Herts C C and TRRL respectively for their help and advice in the initial study along with his colleague Mr Ian Cooper at Hatfield Polytechnic who gave considerable advice on statistics to the research team. Mr N G Knott, Director of Transportation, Hertfordshire C C chaired the steering group succeeding the late Mr Michael Hardy, County Surveyor, who was largely instrumental in pioneering this first indepth research on minor roads. Thanks are extended to the chief officers and their staff for the support to the current project and to Mr Ju-Kun Pan the current researcher. Dr Alkis Kalombaris successfully completed his PhD in February 1990 and most of the work presented reflects his work and diligence. Finally any views expressed are those of the writer and neither purport to represent the views of the steering group nor the collaborating partners.

References


1. Shropshire
2. South Yorkshire
3. Buckinghamshire
4. Hertfordshire
5. Barnet

Fig 1. Location of Collaborating Highway Authorities
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Fig 2 ( ) Sites remaining at June 1991

Causal matrix - Numbers indicate site identification number

Letters denote cell identification for analysis/categorisation purposes
Fig 3 Total traffic versus thickness of bound material
SINGLE AND MULTIPLE CRACKING
TRANSVERSE CRACKING
CRAZING

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Fig 4 (a) Deterioration rates (per annum) for each cell of the causal matrix
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Fig 4 (b) Deterioration rates (per annum) for each of the causal matrix
Fig 5  Some deterioration trends for Cell H
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<th>Support</th>
<th>Width</th>
<th>Thickness</th>
<th>Bound</th>
</tr>
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<tbody>
<tr>
<td>Edge 1,2,3</td>
<td>yes</td>
<td></td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td></td>
</tr>
<tr>
<td>Wheel Track Cracking</td>
<td>yes</td>
<td>yes</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>yes</td>
</tr>
<tr>
<td>Transverse Cracking</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>yes</td>
</tr>
<tr>
<td>Crazing</td>
<td>yes</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>yes</td>
</tr>
<tr>
<td>Fretting</td>
<td>yes</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>yes</td>
</tr>
<tr>
<td>Bleeding</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>yes</td>
</tr>
<tr>
<td>Potholes</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td></td>
<td></td>
<td>yes</td>
<td></td>
</tr>
<tr>
<td>Rutting</td>
<td>yes</td>
<td>yes</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>yes</td>
</tr>
<tr>
<td>Profile</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
<td></td>
<td></td>
<td></td>
<td>yes</td>
</tr>
<tr>
<td>Texture</td>
<td>yes</td>
<td></td>
<td></td>
<td>yes</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deflection</td>
<td>yes</td>
<td>yes</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>yes</td>
</tr>
</tbody>
</table>

Fig 6 Table relating defects to variables expected to contribute to their deterioration
### Stepwise Regression of Edge123 on 5 Predictors

#### Step 1
- **Constant**: 8.710
- **Support**: -6.2
- **T-Ratio**: 4.66
- **R-Sq**: 2.47
- **R-Sq Adj**: 53.98

**More? (Yes, No, Subcommand, or Help)**
- **Fenter** must be > or = **Fremove**
- **Fremove** set = 0.00

**Continue? Y**

#### Step 2 3 4 5
- **Constant**
  - Step 2: 16.2401
  - Step 3: 20.3408
  - Step 4: 17.4611
  - Step 5: 12.1128

- **Support**
  - Step 2: -9.7
  - Step 3: -11.3
  - Step 4: -6.8
  - Step 5: -7.5

- **T-Ratio**
  - Step 2: 2.88
  - Step 3: 2.31
  - Step 4: 1.72
  - Step 5: 1.16

- **C.V.Traff**
  - Step 2: 0.007
  - Step 3: 0.0064
  - Step 4: 0.0059
  - Step 5: 0.0009

- **T-Ratio**
  - Step 2: -1.91
  - Step 3: -1.78
  - Step 4: -1.12
  - Step 5: 0.67

- **Drain**
  - Step 2: -6.7
  - Step 3: -5.2
  - Step 4: -2.1

- **T-Ratio**
  - Step 2: 1.34
  - Step 3: 0.76
  - Step 4: 0.34

- **Width**
  - Step 2: -1.2
  - Step 3: -1.8

- **T-Ratio**
  - Step 2: -0.33
  - Step 3: 0.21

- **Traffic**
  - Step 2: 0.00076

- **T-Ratio**
  - Step 2: 0.18

- **R**
  - Step 2: 3.17
  - Step 3: 2.67
  - Step 4: 2.07
  - Step 5: 2.01

- **R-Sq**
  - Step 2: 65.51
  - Step 3: 72.34
  - Step 4: 74.63
  - Step 5: 75.77

---

*Fig 7 Stepwise Regression example for edge defect*
<table>
<thead>
<tr>
<th>DEFECT</th>
<th>EQUATION</th>
<th>$100r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Cracking</td>
<td>$Y = 2.17 - 0.0089 \text{ THICKBOU}$</td>
<td>54</td>
</tr>
<tr>
<td>Edge</td>
<td>$Y = 16.24 - 9.7 \text{ (SUPP)} + 0.007 \text{ CVTRAFF}$</td>
<td>65</td>
</tr>
<tr>
<td>Rutting</td>
<td>$Y = 3.72 + 0.05 \text{ CVTRAFF} - 0.07 \text{ THICKBOU}$</td>
<td>64</td>
</tr>
<tr>
<td>Potholing</td>
<td>$Y = 1.77 - 5.21 \text{ CBR} - 0.0053 \text{ THICKBOU}$</td>
<td>59</td>
</tr>
<tr>
<td>Texture</td>
<td>$Y = 10.31 - 0.082 \text{ THICKBOU} + 0.01 \text{ TRAFFIC}$</td>
<td>51</td>
</tr>
<tr>
<td>Wheel Track Cracking</td>
<td>$Y = 71.24 - 0.097 \text{ TRAFFIC} - 14.11 \text{ CBR}$</td>
<td>46</td>
</tr>
<tr>
<td>Crazing</td>
<td>$Y = 36.08 + 0.19 \text{ CVTRAFF} - 0.11 \text{ THICKBOU}$</td>
<td>40</td>
</tr>
<tr>
<td>Profile</td>
<td>$Y = 1.98 - 0.0097 \text{ THICKBOU} - 0.41 \text{ DRAIN}$</td>
<td>37</td>
</tr>
<tr>
<td>Fretting</td>
<td>$Y = 11.28 + 0.304 \text{ TRAFFIC} - 0.065 \text{ THICKBOU}$</td>
<td>28</td>
</tr>
<tr>
<td>Bleeding</td>
<td>$Y = 5.17 + 0.212 \text{ CVTRAFF} - 0.18 \text{ THICKBOU}$</td>
<td>24</td>
</tr>
</tbody>
</table>

Fig 8 Summary of predictive equations for measured defects
<table>
<thead>
<tr>
<th>VARIABLE</th>
<th>PC1</th>
<th>PC2</th>
</tr>
</thead>
<tbody>
<tr>
<td>EDGE 123</td>
<td>0.2275</td>
<td>0.0135</td>
</tr>
<tr>
<td>WTC</td>
<td>0.0825</td>
<td>0.0152</td>
</tr>
<tr>
<td>TRC</td>
<td>0.0019</td>
<td>0.2381</td>
</tr>
<tr>
<td>CRAZ</td>
<td>0.0971</td>
<td>0.0130</td>
</tr>
<tr>
<td>FRET</td>
<td>0.0017</td>
<td>0.2516</td>
</tr>
<tr>
<td>BLEED</td>
<td>0.0688</td>
<td>0.3229</td>
</tr>
<tr>
<td>POT</td>
<td>0.1527</td>
<td>0.0131</td>
</tr>
<tr>
<td>RUT</td>
<td>0.2155</td>
<td>0.0120</td>
</tr>
<tr>
<td>PROF</td>
<td>0.0702</td>
<td>0.0172</td>
</tr>
<tr>
<td>TEXT</td>
<td>0.0821</td>
<td>0.1034</td>
</tr>
<tr>
<td>% VARIANCE</td>
<td>70.89</td>
<td>7.83</td>
</tr>
</tbody>
</table>

Fig 9 Summary of Principal Components Analysis
FIG 10 A SYSTEMS APPROACH TO DEVELOPMENT OF MAINTENANCE STRATEGIES
### Fig 11: Causal matrix (figures denote site number)

<table>
<thead>
<tr>
<th>NOT EFF</th>
<th>EFF</th>
<th>NOT EFF</th>
<th>EFF</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;1500 VPD</td>
<td>&gt;150 mm</td>
<td>&lt;150 mm</td>
<td>&gt;150 mm</td>
</tr>
<tr>
<td>&lt;10%</td>
<td>51, 57, 73</td>
<td>67</td>
<td>7, 8</td>
</tr>
<tr>
<td>&gt;10%</td>
<td>30, 55, 59, 65</td>
<td>22, 42, 44, 52</td>
<td>14, 41</td>
</tr>
<tr>
<td>&gt;10%</td>
<td>31, 37, 40, 70</td>
<td>64, 69</td>
<td>10, 21, 32, 60</td>
</tr>
<tr>
<td>&lt;10%</td>
<td>3, 4, 6, 17, 23</td>
<td>24, 25, 39, 43</td>
<td>33, 47</td>
</tr>
</tbody>
</table>

### Fig 12: Deterioration rates for Edge, Wheeltrack Cracking and Rutting

<table>
<thead>
<tr>
<th>NOT EFF</th>
<th>EFF</th>
<th>NOT EFF</th>
<th>EFF</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;1500 VPD</td>
<td>&gt;150 mm</td>
<td>&lt;150 mm</td>
<td>&gt;150 mm</td>
</tr>
<tr>
<td>&lt;10%</td>
<td>EDGE = 1.70 m/yr</td>
<td>EDGE = 2.10 m/yr</td>
<td>EDGE = 2.50 m/yr</td>
</tr>
<tr>
<td>WTC = 5.10 m/yr</td>
<td>WTC = 5.60 m/yr</td>
<td>WTC = 3.80 m/yr</td>
<td>WTC = 5.00 m/yr</td>
</tr>
<tr>
<td>RUT = 3.17 mm/yr</td>
<td>RUT = 3.70 mm/yr</td>
<td>RUT = 4.00 mm/yr</td>
<td>RUT = 3.25 mm/yr</td>
</tr>
<tr>
<td>&gt;10%</td>
<td>EDGE = 1.02 m/yr</td>
<td>EDGE = 0.70 m/yr</td>
<td>EDGE = 1.15 m/yr</td>
</tr>
<tr>
<td>WTC = 5.65 m/yr</td>
<td>WTC = 6.15 m/yr</td>
<td>WTC = 4.35 m/yr</td>
<td>WTC = 3.34 m/yr</td>
</tr>
<tr>
<td>RUT = 3.18 mm/yr</td>
<td>RUT = 3.20 mm/yr</td>
<td>RUT = 2.35 mm/yr</td>
<td>RUT = 3.03 mm/yr</td>
</tr>
<tr>
<td>&gt;10%</td>
<td>EDGE = 1.73 m/yr</td>
<td>EDGE = 1.90 m/yr</td>
<td>EDGE = 1.75 m/yr</td>
</tr>
<tr>
<td>WTC = 5.90 m/yr</td>
<td>WTC = 5.60 m/yr</td>
<td>WTC = 3.87 m/yr</td>
<td>WTC = 3.72 m/yr</td>
</tr>
<tr>
<td>RUT = 2.63 mm/yr</td>
<td>RUT = 1.60 mm/yr</td>
<td>RUT = 2.50 mm/yr</td>
<td>RUT = 3.83 mm/yr</td>
</tr>
<tr>
<td>&gt;10%</td>
<td>EDGE = 1.16 m/yr</td>
<td>EDGE = 0.80 m/yr</td>
<td>EDGE = 1.15 m/yr</td>
</tr>
<tr>
<td>WTC = 10.30 m/yr</td>
<td>WTC = 7.27 m/yr</td>
<td>WTC = 1.05 m/yr</td>
<td>WTC = 5.30 m/yr</td>
</tr>
<tr>
<td>RUT = 3.36 mm/yr</td>
<td>RUT = 4.35 mm/yr</td>
<td>RUT = 2.95 mm/yr</td>
<td>RUT = 2.51 mm/yr</td>
</tr>
</tbody>
</table>
Treatment of Bearing Capacity Results

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and

Anica Petkovsek
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Treatment of Bearing Capacity Results

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and

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Abstract

In 1987 we started with a research project on 26 road sections located in 3 regions with different climate conditions (hard and moderate winters), with different road constructions and various materials in subbase. For all test sections we collected:
- inventory data - location, structure;
- laboratory tests of materials in road constructions and of that from subbase;
- monitoring data - distress and deflection measuring with deflectograph Lacroix;
- traffic data;
- environmental data hydrogeological data of the surroundings of test sections;
- maintenance and rehabilitation data.

Our aim was definition of seasonal, material and regional factor influencing bearing capacity results.

Some of the conclusions until now are:
- we didn't found good seasonal differences between bearing capacity results by time (main reason is that no thaw period appeared in project time);
- in situ and laboratory tests soils has shown an important decrease in dry density and bearing capacity of soil materials in road construction and an important decrease in quality of base materials;
- so we decided to buy different laboratory equipment and FWD to make dynamic investigations of materials and road constructions in next years.
1. STARTING ELEMENTS

Treatment of deflection measurement results, also by the Lacroix measuring instrument, is a complex process. It requires a precise performance of measurement runs and an extensive knowledge of potential impacts on these results, in order to be able to give a professional explanation of our findings that will result in a practical application of these results to road section rehabilitation problems on the entire road network.

Having performed a number of bearing capacity measurements using the Lacroix instrument, on the existing road network and having treated measurement results and followed up literature relating to this particular subject, we established that relevant instructions when to run measurements, treat and apply those results in our circumstances, did not meet our needs or were incomplete. We came to similar conclusions while examining and making use of the other standards and technical regulations. Considering that the obtained results - representative deflection values - do not reflect the actual condition of our pavements, we decided to determine, through this project assignment which we have already been working on for a few years, the laws of specific parameter impacts on bearing capacity of road structural systems in the Slovene area.

As already recommended by the procedures of measurement runs, we strongly emphasized measurement of regular programs carried out in the early spring - during the period of thaw. This period of time was decided in consultation with different contractors of civil works who advised us that we should avoid running measurements on possibly encountered frozen road body.

Springtime measurements were proceeded also after the thaw and lasted until the period of time when the arisen weather temperatures resulted in asphalt layers temperature exceeding 30 degrees Celsius and/or until a period of time estimated to be without rainfall for a longer period of time.

Measurements were continued in autumn and after longer period of rainfalls.

Having based our consideration on well-known facts as
concerns impact of varied weather conditions and seasons on the values of measured deflections, we endeavoured to identify the current seasonly factor as per individual impacts arising from

- region
- geologic- geotechnical and hydrogeological characteristic of the ground
- foundation soil quality and constitution
- quality of road structure earth materials and stone aggregates
- road structure cross section
- quality of asphalt concrete
- asphalt layer temperature
- traffic load

and affecting pavement bearing capacity.

2. PROJECT ASSIGNMENT PROGRAM

In accordance with project assignment goals, the program of this assignment includes the proposed investigations and activities which are aimed at bringing the desired results within a few years, starting from 1987. We wish to highlight the following program lines:

- determination of road sections to be subject of investigation
- touring of geological and hydrological conditions of the referenced road section environment
- identification of layer thickness and material component type constituting individual layers of road structure
- identification of foundation soil constitution
- bearing capacity measurement by the Lacroix.

The assignment is a research project under way, cofinanced by "Republiška uprava za ceste" (The Republican Agency for Roads) and "Raziskovalna skupnost Slovenije" (The Research Community of Slovenia).

2.1 Determination of Road Sections

The original group of 50 sections on 6 regional areas of Slovenia has been reduced to 26 sections in 3 regions. As a rule, the sections are 500 m in length. Considering that these sections have been divided, in regard to the results achieved by bearing capacity measurements, to shorter sections, we have accordingly got 57 sections, which are now undergoing investigations according to the program.
2.2 **Determination of Regions**

Climate characteristics as per individual regions are:

- Gorenjska (the Upper Carniola): harsh winters, mild summers
- Prekmurje (Transmuraland): continental climate with harsh winters
- Kočevsko (Lower Carniola): continental climate with harsh winters and hot summers.

2.3 **Selection of Profiles**

With regard to road structure cross section, the following types were selected:

- cut: deep, shallow
- embankment: high, low
- R.O.W. running in the formation level (ground).

2.4 **Selection of Asphalt Concrete Pavements**

The assignment includes only sections with flexible pavements having less than 13 cm of asphalt concrete layer thickness.

2.5 **Geological and Hydrological Conditions**

Slovenia has an extremely heterogeneous geological structure. The process of soil formation and origin affect the morphology of ground and thereby related certain specific behaviour of soils under adjoining hydrological conditions and dynamic loads.

It is a characteristic feature that certain types of soil in Slovenia, although placed under optimum moisture conditions and compacted to a maximum density, have a high capacity of water absorption, which is directly reflected on mechanical properties and a variable bearing capacity of a placed layer in different time periods, in particular with regard to rainfalls and thaw.

We were interested to know whether it was possible, by an expert examination of geological and hydrogeological conditions and practical experience of soil constitution and properties, to forecast possible effects of foundation soil or ground geology on pavement bearing capacity.

2.6 **Identification of Constituent Material Type and Individual Layer Thickness**

Soil sampling was carried out by drilling up to 2 m depth (subject to road section) under the surfacing. Ap-
plied was the system of 100 % core sampling with undisturbed samples squeezed out only in the laboratory. Regarding laboratory analyses, we laid emphasis on the material classification, finding of natural moisture content, determination of grain size parameters, natural density and CBR relations.

2.7 Bearing Capacity Measurements

The largest emphasis in the assignment is laid on deflection measurements and treatment of relevant results.

According to the program measurements are to be performed on each measuring section in at least five yearly cycles. Measurements are to be more frequent in thaw period. The cycle of measurements is to be run always after a longer period of rainfall, in order to establish rainfall impact on pavement bearing capacity.

3. RESULTS

Hitherto we have carried out bearing capacity measurements, defined geologic and hydrologic characteristics of the ground and, moreover, we have taken samples of materials from some test pavements and performed laboratory analyses.

3.1 Bearing Capacity Measurements

We carried out 16 measuring cycles, but in our tables and hitherto processing we have not allowed for the results from the years 1987 and 1988.

We were practically not able to make any conclusions from the results of deflection measurements up to the year 1990 because the values of average deflections do not show the expected movements. The reason for such results was explained by two facts.

First, the disposition of individual cycles was not very successful, which is further affected by hydrometeorologic conditions within these three years, which were not standard. There was, namely, no harsh winter with low temperatures being followed by thaw periods.

Additionally, there was essentially less rainfall than normal or it had only a local character. Consequently, less measurements than originally anticipated, were performed.

After measurements in 1991 we carried out the analysis of results of deflection average values measurements, according to the day in the year when measurements were
taken, regions, profiles and cycles, and tried to compare and analyse the same with data obtained from touring the site for geological and hydrogeological conditions and with regard to the results of laboratory researches of soil samples taken from road structure.

The initial comparisons and analyses of the hitherto results - although uncomplete - lead us up to a conclusion that with measurements of pavement bearing capacity it is the matter of a certain standard pattern of behaviour, which was in a way expected while preparing the study.

Anywhere in subbase and/or subgrade there are occurrences of clayey, clay-silty soils or brittle stone materials, such as clay, stones, shales, which are classified, according to AASHTO, into the groups A5, A6, a lower supporting capacity and in particular, a more pronounced dispersion (non-uniformity) of results of measured deflections, as compared with those sections having the subbase and/or subgrade of sand-gravel or solid crushed stone materials. These differences still increase on sections with unfavourable natural hydrological conditions and inadequate or unsufficiently maintained water discharge facilities.

Let us compare measurement results from two characteristic measuring sections no.3 and no.25.

<table>
<thead>
<tr>
<th>Sect. no.</th>
<th>Site Geology and Materials</th>
<th>Hydrogeol. conditions</th>
<th>R.O.W.</th>
<th>Water Discharge</th>
<th>Forecast of Possible Neg. Effects of Existing Ground</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Triad dolomite in the base course, covered by up to some metres thick coat of weathered silty clay. The weathered clay is highly deformable and sensitive to atmospheric conditions.</td>
<td>Good - no groundwater in influential depths.</td>
<td>Mixed, shallow building cut, very low earth-fill, ground.</td>
<td>Weak.</td>
<td>Possible negative effect of deformable clayey soils occurring in foundation ground and of weak water discharge.</td>
</tr>
</tbody>
</table>

TABLE 1: Geological and Hydrogeological Conditions - Forecast of Natural Ground Possible Negative Effects on Road Structure

As shown in Table 1, possible negative impacts of bad water discharge and clay-silty soils in the foundation ground, exerted on pavement bearing capacity, may be expected on Section no.3, following observations of soil investigation touring. On Section no.25, however, geological and hydrological condition of the ground is good,
CHART 1: Deflection average values of the last 3 years measuring cycles on section no. 3 for each of the homogeneous subsections, on embankment and with CL - CH in subbase and/or subgrade.

CHART 2: Deflection average values of the last 11 measuring cycles on section no. 25 for each of the homogeneous subsections, on ground and with GM, GW in subbase and/or subgrade.
so that negative impacts of environment on the road structure are not expected according to forecast. Hydro-meteorological circumstances on Section no.25, on the other hand, are less favourable than those on Section no.3.

Deflection average values of the last 11 measuring cycles carried out on the referenced two sections, are collected in Charts no.1 and 2, separately for each of the homogeneous subsections inside Section no.3 and/or 25. The results show that essentially higher values of deflections and in particular a considerably higher dispersion of all measured values, have been recorded on Section no.3. The structure and quality of materials constituting the pavement, are similar on both sections.

CHART 3: Deflection average values of last 11 measuring cycles on pavements with clay and silty soils in subbase and/ or subgrade.

The conclusions similar to the results of measurements of compared sections no.3 and no.25, are offered by the results of the latest 11 cycles of measured deflections on all referenced sections. In Charts no.3, 4 and 5 we collected mean values of measured deflections, yet separately with regard to the type of soil in the subbase and/ or subgrade or to ground constitution within the influential base course of the road structure. The rate of settlements is clearly different and above all the high dispersion of measured values on roads with clayey soils incorporated in the subbase and/ or subgrade. Surprising are certain values of settlements from
CHART 4: Deflection average values of the last 11 measuring cycles on pavements with clayey sand and clayey gravel in subbase and/ or subgrade.

CHART 5: Deflection average values of the last 11 measuring cycles on pavements with sandy gravel and crushed stone in subbase and/ or subgrade.
sections having crushed stone materials incorporated in
the subbase and/or subgrade. We will endeavour to find
out and finally clarify also this phenomenon, by further
analyses; there are certain indications that major in-
fluential parameters result from an unsufficient water
discharge and extremely weak grain size of stone mate-
rial.

Of course, a major deficiency of such presentation of
measurement results is in its allowance for ground con-
stitution as the only eliminating factor, while the
other very important factors such as water discharge,
road cross section and others, remain in the shadow and
are indirectly reflected in a high dispersion of collected
results.

CHART 6: Deflection average values of the last 3 years
measuring cycles on embankments and with CL -
CH in subbase and/or subgrade.

A fuller idea is therefore offered by the results of me-
asurements where possible influential factors are addi-
tionally limited. The Charts no.6, 7 and 8 show the col-
lected mean values of measured deflections, separately
in regard to soil constitution of the subbase and/or
subgrade, but only on road sections taking their course
in embankments. There the problem of water discharge or
unfavourable hydrogeological conditions is less
pronounced, that is why the results on Charts no.6, 7
and 8 show a clearer and more comprehensible picture of
CHART 7: Deflection average values of the 3 years measuring cycles on embankments and with GC, SC, CL/GC in subbase and/or subgrade.

CHART 8: Deflection average values of the last 3 years measuring cycles on embankments and with GM, GW in subbase and/or subgrade.
measured deflections dependence on subbase and/or subgrade.

4. CONCLUSIONS

The research assignment program has been developed in order to establish the importance of individual impacts on the results of bearing capacity measurements by means of the Lacroix instrument. Some work has already been done, we also showed some of the hitherto obtained results and some correlations between the observed parameters.

The basic conclusion of the hitherto study: the obtained bases to determine the rules for treating the results of deflection measurements depend on the day when measurement is taken in a year, on road structure cross section and on type of materials in subgrade.

Another conclusion of the study: owing to mostly varied intensity of yearly seasons (in particular winters) we have to expect variable results of bearing capacity measurements and will, therefore, establish the rule based on our own experience, for treating deflection measurement results.

Owing to deficient data on road structural systems we shall have to carry out additional sampling and make still missing investigations of road structure materials and/or materials in subgrade and from asphalt layers.

We found important decrease of subbase and/or subgrade materials quality.

Because of incapability to look over dynamic response of road constructions and materials under it we decided to buy different laboratory equipment and FWD to make dynamic investigations of material and road structure in next years.

5. LITERATURE


A Model of IRI for Jointed Plain Concrete Pavements

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J Gonzalez
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Abstract

The international roughness Index IRI is an essential input parameter for the evaluation of operational costs. In order to generate a predictive model of IRI a linking equation with the surface distress parameters is needed. Contributing to that purpose predictive models of slab cracking and joint faulting have already been developed by considering as a whole the actual behavior of Chilean PCC pavements. In the paper a correlation will be presented between the IRI and several indices of surface characteristics, mainly slab cracking, spalling and repairs determined through detailed visual surveying and absolute faultings measured at joints. The IRI is determined by continuous measurements made with an optical profilometer KJ-Law passing several times on 100-200 m long homogeneous sections of jointed concrete pavements selected to cover an ample-range of surface distress. To use the profilometer a complete measurement methodology had to be developed to take into account a number of environmental effects that are controlling the performance of the Chilean in-service PCC pavements, such as the predominant upward concavity of slabs due to the yearly cyclic moisture warping and to the daily cyclic temperature curling. The results presented and the IRI measurement methodology are within the cope of LTPP program.
A MODEL OF IRI FOR JOINTED PLAIN CONCRETE PAVEMENTS

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1. ABSTRACT

The International Roughness Index IRI is an essential input parameter for the evaluation of operational costs. In order to generate a predictive model of IRI a linking equation with the surface distress parameters is needed. In the paper a model is presented between the IRI and several indices of surface characteristics, mainly absolute faulting measured at joints, slab cracking and repairs determined through detailed visual surveying. The IRI is determined by continuous measurements made with an optical profilometer KJ-Law passing several times on homogeneous sections of JPCP selected to cover a range of surface distress. To use the profilometer a complete measurement methodology had to be developed to take into account a number of environmental effects that are controlling the performance of the Chilean in-service PCC pavements, such as the predominant upward concavity of slabs due to the yearly cyclic moisture warping and to the daily cyclic temperature curling.

2. INTRODUCTION

The surface roughness is a characteristic of the road affecting the safety and comfort of the passengers and drivers, as well as the operational costs of vehicles. To objectively quantify roughness the World Bank has defined the International Roughness Index IRI (1) as the ratio between the total displacement of a standard car body suspension (in m) and the travelled distance (in Km) speeding at 80 Km per hour (quarter car hypothesis). The IRI is an evaluation parameter increasingly in use among pavement managers, but with applications reduced only to asphalt pavements and unpaved roads. Therefore, specific applications to concrete pavements relating the IRI to the several distresses that are typical in these kinds of surface are demanded. On the road, the distress can be quantified by means of visual and instrumental surveying procedures recording every defect or variable being incident in the surface roughness (2).

To achieve the characterization of a road surface through the IRI parameter, a possibility is to measure it with a bump integrator mounted on a standard vehicle, or compute its value from a measured profile (1). In this paper some IRI values resulting from calculations on actual profiles measured with an optical inertial profilometer K.J. Law 690 DNC are presented. Measurements were made
in 100 - 200 m long homogeneous sections of PCC pavements, selected to cover an ample range of surface distress.

The present work is part of a research conducted during the last 7 years on the Chile's jointed concrete pavement network and contracted by the National Highway Administration with the ultimate objective of implementing a pavement evaluation system. To that purpose models of distress and costs, had to be developed and/or calibrated for the actual conditions of the road network.

3. COMPONENTS OF IRI

The surface of a newly constructed pavement, notwithstanding all precautions cared during the finishing process, contains several imperfections building an initial $I_{RI0}$ not less than 1.0 (m/Km). As the pavement deteriorates by the cumulative action of traffic and climate, increasing irregularities are developed as a result of wearing out, joints faulting, cracks and joints spalling, as well as punchouts produced by settled slab fragments. Aside of these normal distresses, deficient maintenance operations of patching and joints resealing are adding bumps that increase the IRI value in generally very important proportions. To the case of the Chilean jointed concrete pavements the following composition of IRI has been identified:

$$I_{RT} = I_{RI0} + I_{FAULT} + I_{SPALL} + I_{PUNCH} + I_{SEAL} + I_{PATCH} \ldots \text{Eq.1}$$

It is worth noting that the slab cracking, usually considered the most relevant structural distress, by itself does not constitute a functional distress unless is accompanied by spalling and faulting at the crack edges and/or edge punchouts. To illustrate that statement Figure 1 shows the IRI value measured during the last six years along 7 Km of a jointed PCC pavement, including an instrumented test section (TS-8). These typical results are good example of the high repetitiveness being possible to attain with both the Maysmeter and K.J. Law Profilometer; the small variations among different dates would be essentially the experimental error rather than produced by physical causes. Looking closer at TS-8, the average IRI in a full kilometer is presented in Figure 2 as function of age, together with the percentage of cracked slabs in the same section. The results are showing no significant evolution of IRI, in spite of the progressive cracking that is affecting an increasing number of slabs in the same period; therefore, that value of IRI is still in the primary phase represented by the initial $I_{RI0}$. 
Figure 1. State of the roughness index IRI during the last 6 years along a 7 km pavement section.

Figure 2. Evolution of slab cracking and IRI at a 300 m long test section pavement.
Another component of the IRI value, being significant in Chile, is the effect of slab edges uplifting as a result of the upward warping and curling originated by moisture and temperature gradients (3,4). Considering the complexities introduced by these effects, measurements with the profilometer were performed during the afternoon hours, trying to seek favorable thermal conditions not affecting the road profile, as shown by the result presented in Figure 3.

![Figure 3. Influence of the daily cyclic thermal gradients on IRI.](image)

4. MODELLING THE IRI COMPONENTS

In the Chilean concrete paved network, the initial roughness has improved as more strict prescriptions of construction quality were introduced. As an example, prior to around 1982 when no smoothness requirements were being specified a value of $\text{IRI}_0 = 2.5 \text{ m/Km}$ is considered representative. Later on, the introduction of the Hi-Lo rolling edge to control the acceptance of construction quality, resulted in a sharp reduction of the initial roughness to about $\text{IRI}_0 = 1.5 \text{ m/Km}$. In the future a criteria based on IRI is to be implemented for quality reception of the newly constructed pavements.

To compute the IRI value a mathematical model permitting the simulation of a quarter car can be used. The road profile can be real or simulated to highlight different defects one by one or on specific groups.
To model the joint faulting effects on the IRI, several sections of pavements showing mainly this kind of distress were selected. Measurements were performed at each transverse joint with an ad-hoc apparatus (± 0.2 mm accurate), to produce an average faulting value representative of each homogeneous pavement section. In a parallel action, a theoretical model of IRI from simulated profiles (Figure 4) was developed with the aim of producing a theoretical IRI value for each range of joint faulting. With this methodological approach the actual faulting data permit the prediction of a representative IRI, which is compared in Figure 5 with the actual IRI value obtained through the optical profilometer. The resulting fit may be expressed by the equation:

$$IRI_{FAULT} = 0.0176 \times \text{Fault} \times n_f^{0.83} \quad \text{Eq.2}$$

Units:
- IRI (m/Km),
- Fault (mm),
- $n_f$: the total number of faulted joints within the analyzed pavement section.
When transverse crack faulting is also present, aside of the joint faulting, the same equation can be used to predict the IRI component provided \( n_f \) is the total number of faulted cracks and joints in the section.

With respect to resealing a similar theoretical methodology was used through the modelling of the joint and crack bumps in terms of the average height \( h_R \) (mm) of the resealing and the total number of resealed cracks and joints \( n_R \) within the analyzed pavement section. The resealing height is measured with respect to the approach slab. The resulting fit shown in Figure 6 has a mathematical expression of the form:

\[
IRI_{\text{SEAL}} = 0.0083 \, h_R \cdot n_R^{0.92} \quad \ldots \ldots \ldots \quad \text{Eq.3}
\]
Similar approach for patching and punchouts was attempted, but considering the great variability in length of the actual defects an empirical approach was used instead. On the basis of precise slab-to-slab measurements of joint faulting and resealing height on several pavement sections, $\text{IRI}_{\text{FAULT}}$ and $\text{IRI}_{\text{SEAL}}$ values were deduced by means of the models just described. Having reliable overall $\text{IRI}_T$ measured simultaneously in the same sections, the $\text{IRI}_T$ components due to patching and punchout were obtained by simple subtraction of the other known components: \( \text{IRI}_{\text{PATCH} + \text{PUNCH}} = \text{IRI}_T - \text{IRI}_0 - \text{IRI}_{\text{FAULT}} - \text{IRI}_{\text{SEAL}} \). The resulting numbers were then compared with the corresponding summation, in absolute value, of the patching $\Delta_{\text{PATCH}}$ and punchout $\Delta_{\text{PUNCH}}$ roughness measured in the same sections to get the model expressed in the following terms:

\[
\text{IRI}_{\text{PATCH} + \text{PUNCH}} = 5 \times \left[ \frac{\sqrt{\sum \Delta_{\text{PATCH}}} + \sqrt{\sum \Delta_{\text{PUNCH}}}}{n} \right] \quad \text{Eq. 4}
\]

being $n$ the number of slabs included in the pavement section.

Summing up all components of surface distress included in this analysis (Equation 1), a final satisfactory result is attained as shown in Figure 7, having in mind the numerous sources of error that are implicit in the measurements and simulations.
Figure 7. Comparison between IRI predicted and measured on a pavement showing various types of surface irregularity.

It is important to point out that in this analysis the spalling of cracks and joints is not explicitly included, neither the surface irregularities produced by wearing out, because the pavements under consideration are still too young to develop such distresses; in either case, the analysis methodology for its treatment can be foreseen to be essentially empirical. Surface irregularities produced by geotechnical instabilities of the subgrade were also excluded.

5. CONCLUSIONS

A theoretical-empirical methodology to evaluate the IRI parameter in jointed concrete pavements was developed to predict with satisfactory accuracy the IRI associated with various types of surface roughness, such as transverse joints faulting, bumps from defective resealing and from patching and punchouts, which can be quantified by means of objective visual and instrumental surveying procedures. An initial value IRI₀ reflecting the quality of surface finishing on newly constructed pavements has been included as another single component. From these results the evolutions of IRI in time is considered predictable through the incorporation of distress models for each significant variable affecting the surface roughness.
6. REFERENCES


The High-Speed Road Deflection Tester

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The High-Speed Road Deflection Tester

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Introduction

In the early 1970s, a search for an objective road surveying method that could replace the subjective methods in use at that time began at the Swedish Road and Traffic Research Institute (VTI), following a request by the Swedish Road Administration (VV). It was thought that an ideal measuring system should:

- cause minimal interference to surrounding traffic;
- be durable, reliable, and require minimal service;
- have the ability to collect information about all road characteristics used in the decision-making process regarding road maintenance; and
- be capable of measuring several (and preferably all) relevant road characteristics simultaneously.

In 1981, the first Laser Road Surface Tester (Laser RST) was built at VTI, and it very nearly fulfilled all of these requirements. The Laser RST is a laser-based, computer-automated, non-contact road profilometer system with the ability to simultaneously collect and fully process data about the underlying road surface. Today, it is used to measure roads in Sweden (about 65,000 km annually) and in fourteen other countries throughout the world.

Research and development aimed at maintaining the Laser RST's state-of-the-art status is presently being carried out by OPQ Systems AB (Linköping, Sweden) and the Laser Road Surface Research Group at VTI. The goal is to create a fully automated system that will be capable of inspecting all aspects of the surface and sub-surface structures of a road; such a system does not currently exist. OPQ Systems and the research group at VTI are working with two of missing parameters of a fully automated road analysis system: (1) an automated road-surface distress analyzer for the detection and analysis of cracks and (2) a high-speed non-contact road deflection tester for the measurement of bearing capacity. This paper is concerned with the latter parameter.

History

The purposes of road maintenance are to preserve invested capital and provide the road user with an acceptable road standard. Bearing capacity is very important with regard to the preservation of invested capital, while vibration comfort of vehicle occupants is one of the earliest acceptance criteria considered by VV. The use of vibration comfort as a criterion for decisions about the need for road maintenance has a long international history, with decisions based largely on visual/sensitive information. To replace this subjective information with objective measurement data, it was necessary to separate the components of the road surface into categories or variables that were measurable and to find ways of measuring these variables.

One of the first attempts at VTI to correlate objective measurements with human subjective ratings was carried out in 1973. In this experiment, vibration comfort ratings were correlated with unevenness measures obtained from different types of meters (Magnusson & Arnberg, 1976). The results of this experiment convinced the researchers that measurement devices could be used to predict vibration comfort sensations and thus replace at least one part of subjective road surveys that were carried out routinely every three years.
A natural consequence of this was to consider the possibility of collecting several road surface parameters in only one run. One step in that direction was taken in 1975 with the development of the so-called BV11J, a friction meter BV11 with instrumentation for the simultaneous measurement of friction and unevenness. A brief consideration was then given to the idea of using a single tow vehicle, instrumented for the collection of measurement data from BV11J and a Dutch rut meter, which in two runs could give information about friction, unevenness, and rut depth. This idea was, however, abandoned in favor of the development of a multi-functional mechanical device, the Saab Road Surface Tester (Saab RST).

The Saab RST was based on the Saab Friction Tester, which was essentially a Saab 900 with a friction meter BV11 incorporated in the rear axle. It was developed jointly by Saab Scania and VTI in close cooperation with VV and was capable of measuring friction, unevenness, cross profile, crossfall, and curve radius. Two Saab RST's were built, and they were used in a road survey carried out by VV in 1980 (Arnberg et al., 1979; Arnberg, 1981; Arnberg & Sjögren, 1983a and 1983b).

The latest link in this chain of road surface measuring devices was the development of the Laser RST, which began in 1980 and is presently continuing. In its original version, it was capable of measuring road unevenness and cross profile, while the ability to measure friction when towing a BV11 was foreseen (Arnberg, 1983). Later, the ability to measure crossfall and macro- and megatexture and to detect cracks were added to the system (Arnberg, 1985 and 1986). The current version of Laser RST is also capable of measuring horizontal and vertical curve radii (Arnberg et al., in press).

As mentioned above, research and development of the Laser RST is presently continuing. Some of the projects under development, in addition to the high-speed deflection tester, include (1) a method of accurately calculating cut and fill volumes of road work based on Laser RST measurements; (2) the use of the “Navigation Satellite Timing and Ranging Global Positioning System” (Navstar GPS) for various applications such as navigation, mapping, surveying, etc.; and (3) an improved method to detect, classify and rate the severity of road surface cracking using a combination of video and laser techniques.

Knowledge about bearing capacity is very important with regard to road maintenance. The combined information about road unevenness, cross profile, crossfall, and cracking gives some indication of the condition of the road, but information about road deflection under load is also needed for more detailed information about the bearing capacity of the road. The purpose of this report is to describe the working principle and the measurement results obtained to date with the new high-speed deflection tester currently being investigated at VTI.

## Measurement Method

To find sections of road with low bearing capacity, stationary devices (e.g., the Falling Weight Deflectometer) and slow-moving instruments (e.g., the Benkelman Beam and the Lacroix Deflectograph) have been used for many years. However, because of the very low measurement speeds, only spot measurements have normally been carried out with the obvious risk of missing the weaker parts of the road. The purpose of the method under development at VTI is to make continuous road-deflection measure-
ments at higher speeds possible, thus establishing a more reliable base for pavement management work. It will also use a loading method more akin to the actual load the road is exposed to from heavy vehicles.

The measurement method involves a heavy vehicle, a lorry or a bus, with a cross profilometer mounted in front of or behind a lightly loaded front axle and outside the deflection basin caused by this axle. There is a second profilometer placed immediately behind the heavily loaded rear axle, which allows the measurement of the maximum deflection under the load. The front profilometer measures the cross profile of a section of road in an unloaded condition while the rear profilometer measures the same cross section when loaded. The difference between these profiles gives the deflection of the road surface. The cross profilometers are basically the same as those used by the Laser RST, but more laser units are distributed along the cross profilometer and in such a way as to make the recording of the shape of the deflection basin possible.

Figure 1 shows the experimental vehicle used for the initial testing of the method. The extension of the cross profilometer on the left side of the vehicle permits the outermost laser to measure a spot 1.54 m to the left of the left rear wheel. This point is defined as the zero point of the cross profiles. On the right side of the vehicle, a laser angled 45° vertically out from the vehicle is used, giving a measurement width of 4.4 m.

When the vehicle is measuring, it is driven along the road at normal traffic speeds, and the distance between the different laser units and the road surface is sampled at a rate of 16 kHz. A mean distance value for each laser unit is calculated every 100 - 150 mm, thus filtering out the macrotexture effect on the readings, and a mean profile based on these mean values is determined for each cross profilometer.

Measurement results are presented in terms of “difference profiles,” i.e., the difference between the profiles recorded by the two cross profilometers. The difference profile is calculated in this way:
1. the two cross profiles recorded at the same cross section of the road are found;
2. the cross profiles are arranged so that their end points coincide; and
3. the mean profile obtained by the front profilometer is subtracted from the mean profile obtained by the rear profilometer.

In addition to measuring the bearing capacity, the Laser RDT will also measure the temperature of the road surface and the thickness of the different layers of the road structure using non-contact methods. Finally, the Laser RDT may also be used for surveying, giving a quick appreciation of the bearing capacity, an indication of if and where more detailed deflection studies are warranted, and deflection information for statistical purposes.

It is anticipated that at some point in the future, the Laser RDT and the Laser RST will be combined in one measurement vehicle, making it possible to measure road deflection and a number of surface variables simultaneously.

Results

The results are presented in four parts: (1) the repeatability of the measurements, (2) the influence of axle load, (3) the influence of measurement speed, and (4) a comparison of the Laser RDT and a Falling Weight Deflectometer (FWD).

Repeatability. Figure 2 shows the results of two measurements at 5 km/h and constant axle loads carried out at Mantorp Park, a racing circuit used for drag-, truck-, car- and road racing. The difference profiles shown are the mean over a 400-m section, and the marked points on the profiles represent the lateral position of the lasers. The left rear wheel of the lorry is, as mentioned above, situated at 1.54 m while the right wheel as at 2.95 m from the zero point. As can be seen, the difference profiles end at 3.5 m instead of 4.4 as it should have been. The reason for this is that the angled laser on the right side did not work properly, so it sometimes had to be omitted in the calculation of the difference profile. Nevertheless, the figure shows very good repeatability of the measurement.

![Figure 2. Repeatability test. Measurement speed 5 km/h, rear axle load 110 kN.](image-url)
Influence of the Axle Loads. Figure 3 shows the results of repeated measurements using two different axle-load combinations. The upper curves are from measurements with a rear-axle load of 79 kN and front-axle load of 32 kN, while the bottom curves represent a rear-axle load of 94 kN and front-axle load of 30 kN. As can be seen, the increased axle load results in increased deflection, as would be expected.

![Graph showing influence of axle loads](image1)

**Figure 3.** Repeated measurements at two different axle-load combinations. The first figure is the rear-axle load and the second the front-axle load.

Influence of Measurement Speed. Figure 4 shows the influence of speed on the deflection. The well-known fact that the stiffness of the bituminous materials increases with increasing velocity of load application is reflected in the result in that increasing measurement speed gives decreasing deflections. In Figure 4, the outermost laser on the right side has been omitted for the reason described above, but it should not have any qualitative influence on the result.

![Graph showing influence of measurement speed](image2)

**Figure 4.** Result of measurements at three different speeds.
Comparison of the Laser RDT and the Falling Weight Deflectometer (FWD). To compare the Laser RDT with the Falling Weight Deflectometer (FWD), 13 road sections with different surface conditions and bearing capacity were identified. The sections measured were 100 m in length and rather homogeneous. Measurements with both devices were carried out at approximately the same time to avoid any difference in ambient conditions such as temperature and moisture.

Measurements with the FWD were carried out every 10 m along a 100-m section. This was followed by a second series of measurements, again at 10-m intervals, but this time displaced 5 m from the first measurement. A total of 21 points along the road section was measured. At each measuring point the FWD did 2 - 5 drops, depending on the difference between the first drops. The deflection value obtained at the last drop at each measuring point was used in the calculation of the validity of the FWD. The load was approximately 50 kN, and the deflection was measured by six sensors placed from 0 to 1.5 m from the center with 0.3 m as the step value. The sensors were placed along the road in the left wheel track, in contrast to the Laser RDT, which measures across the road.

The Laser RDT measured each section three times at 5 km/h and two times at 30 km/h. In the validity calculation (see below), the average of the three at 5 km/h was used. The axle loads were 94 kN on the rear axle and 30 kN on the front axle. In order to increase the deflection, the rear axle carried only single types, and the distance between them was 2.05 m. Tire pressure was 850 kPa.

Based on these measurements, the reliability coefficient was calculated for the Laser RDT and for the FWD. Figure 5 shows the results for the Laser RDT, and the values shown are the mean deflection for each test section. The reliability coefficient is 0.88, and the intercept of the regression line is very close to zero. Figure 6 shows the reliability of the FWD, which appears to be excellent (reliability coefficient 0.997). However, it should be noted that this reliability is calculated from the mean of 21 measured points along each 100-m section, while in normal production measurements one or perhaps two points would be measured. It should thus be observed that the reliability when measuring at random on a fixed section (e.g., 100 m in length) will probably be rather poor on sections with varying bearing capacity, especially compared to measuring devices producing a measure based on measuring the entire section.

Figure 7 shows a comparison of the Laser RDT and the FWD. The agreement appears to be limited (correlation coefficient 0.52). However, it should be noted that when comparing these two completely different methods, the relationship between the two measures probably depends on the properties of the road. It is known that Road Section 4 differs from the other measured sections in that it has a very thick wearing course as a result of repeated overlays. Road Section 3 is on a very narrow and dwindling road where measuring is very difficult, and at the evaluation of the second measurement it was observed that the vehicle was driven partly with the outermost laser outside of the road edge. If those two sections are deleted, the correlation coefficient increases to 0.82 (Figure 8).

Operating a vehicle with the laser carrying extension shown in Figure 1 will of course cause problems in normal traffic, especially on narrow and dwindling roads, and necessitate the use of special traffic safety measures such as a follower vehicle to warn other road users about the oncoming wide vehicle. To investigate the possibility of measuring without any extensions, a second evaluation of the Laser RDT measurements was carried out. The measurement used was the deflection under the rear
Figure 5. Reliability of the Laser RDT.

Figure 6. Reliability of the FWD.
Figure 7. Comparison between maximum deflections as measured by the Laser RDT and the FWD.

Figure 8. Comparison of maximum deflections as measured by the Laser RDT and the FWD. Corrected for outliers as described in the text.
wheels in reference to a line connecting the two nearest lasers on each side of the wheel. This measurement was compared to the deflection measured 300 mm from the center of the spot struck by the weight of the FWD. Figure 9 shows that Road Section 4 is still an "outlier" while Road Section 3 is now very close to the regression line. This illustrates that the problem with the narrow road has been solved by this type of evaluation. The correlation coefficient is 0.68. If Road Section 4 is again removed, the correlation coefficient increases to 0.88 (Figure 10).

[Graph showing deflection vs. deflection]

**Figure 9.** Comparison of deflections within a 300-mm radius around the center of the load as measured by the Laser RDT and the FWD.

## Discussion

The results look very promising so far, although quite a bit of work remains before this method can be used in real production measurements. Especially interesting is the observation that deflection measures based only on the deflection within 300 mm on each side of the tire show good correlation with the FWD.

Work to be done includes the construction of a new measurement vehicle that is a combination of the Laser RDT and Laser RST. In all probability, this will be a 2.5-m wide bus measuring at least 1 m out from the tires using angled lasers. The deflection will be calculated on-line in the bus together with all other road surface variables currently measured by the Laser RST. These measurement data will then be used to describe not only the surface condition as is done now by the Laser RST but also how the bearing capacity varies along the road.

A prognosis for the evaluation of the road condition can be made by combining Laser RDT results, like the difference in the deflection under the right and left
wheels in reference to a line connecting the two nearest lasers on each side of the wheel. This measurement was compared to the deflection measured 300 mm from the center of the spot struck by the weight of the FWD. Figure 9 shows that Road Section 4 is still an “outlier” while Road Section 3 is now very close to the regression line. This illustrates that the problem with the narrow road has been solved by this type of evaluation. The correlation coefficient is 0.68. If Road Section 4 is again removed, the correlation coefficient increases to 0.88 (Figure 10).

![Figure 9. Comparison of deflections within a 300-mm radius around the center of the load as measured by the Laser RDT and the FWD.](image)

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Work to be done includes the construction of a new measurement vehicle that is a combination of the Laser RDT and Laser RST. In all probability, this will be a 2.5-m wide bus (measuring at least 1 m out from the tires using angled lasers). The deflection will be calculated on-line in the bus together with all other road surface variables currently measured by the Laser RST. These measurement data will then be used to describe not only the surface condition as is done now by the Laser RST but also how the bearing capacity varies along the road.

A prognosis for the evaluation of the road condition can be made by combining Laser RDT results, like the difference in the deflection under the right and left
Figure 10. Comparison of deflections within a 300-mm radius around the center of the load as measured by the Laser RDT and the FWD. Corrected for outliers as described in the text.

wheels, with results like rut depth in the right and left tracks and how the rut depths and unevenness varies along the road. Other measured parameters such as crossfall, texture, and RMS values for different wavelength ranges could also be used as indicators of possible pavement problems.

Road surface temperature influences the magnitude of the deflection caused by a certain load. Some initial measurements have been carried out with the Laser RDT, but there have been no systematic studies of this to date.

Four different potential uses of measurement data from the Laser RDT have been identified:

1. to give an overall evaluation about the bearing capacity of the roadnet;
2. to direct more precise measurements to identified critical areas;
3. to collect information about the real road deflection caused by heavy vehicles at different speeds and in different seasons, providing a better basis for decisions regarding regional and seasonal regulations about speed and payload; and
4. to study actual road deflections caused by different types of vehicle, suspensions and tires.

The Laser RDT is still in its infancy, but the results of the testing to date indicate its strong potential.
References


PAVUE: A Real-Time Pavement Distress Analyzer

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PAVUE: A Real-Time Pavement Distress Analyzer

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Preface

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1 Introduction

The Laser Road Surface Tester (Laser RST) is a laser-based, computer-automated non-contact profilometer system with the ability to collect and simultaneously process data about the underlying road surface at 90 km/h (Arnberg et al., in press-a). The first Laser RST was built in Sweden in 1981 and is now used to measure roads in Sweden and in 14 other countries throughout the world.

Research and development aimed at maintaining the Laser RST's state-of-the-art capabilities is presently being carried out by OPQ Systems AB and the Laser Road Surface Research Group at the Swedish Road and Traffic Institute (VTI). The goal is to create a fully automated system that will be capable of measuring all aspects of the surface and subsurface structures of a road. Such a system does not currently exist. OPQ Systems and the VTI group are working with two of the missing parameters of such a fully automated system: (1) a high-speed non-contact road deflection meter and (2) an automated road surface distress analyzer. This paper is concerned with the second function.

The Laser RST vehicle already measures a large number of road variables including rut depth, roughness, surface texture, elapsed distance, crossfall, curvature, etc. The measurement system consists of 11 laser rangefinders mounted on the front bumper of a van, special accelerometers, inclinometers, speed transducers, rate gyros, and a large number of specialized signal processing computers to analyze the information coming from the pavement at a 32-kHz data rate. The system samples the distance to the pavement and forms a cross-profile of the road surface every 2 mm at its top survey speed of 90 km/h. The Laser RST van is built at VTI (Statens väg- och trafikinstitut) and at OPQ Systems AB in Linköping, Sweden. All the specialized electronics hardware and software have been developed by OPQ Systems.

While the direct measurement of depth using laser rangefinders is ideal for most road variables, this is not true for measuring cracks. With only a limited number of “sampling” spots (each only 1 mm in diameter), even 11 lasers can miss a significant number of cracks. While the longitudinal sampling rate is very high (32 kHz), the horizontal sampling is low. Too many cracks are missed, and patterns of cracking important to road engineers (such as “D-cracks” and “Y-cracks”) cannot be measured at all.

In order to improve the crack measurement performance of the Laser RST van, OPQ Systems AB is currently developing a high-speed video data acquisition and analysis (PAVUE) system to be used alongside the laser system. However, the addition of the PAVUE system cannot constrain the current laser system in any way, so must be able to acquire and fully process video data at road survey speeds of up to 90 km/h. Such high data rates require unique cameras, lighting, and processing approaches, along with specialized processing hardware architecture. PAVUE is a hardware and software system for the high-speed inspection of surfaces, in particular, road pavement surfaces. The final PAVUE design will be capable of sustained real-time continuous inspection of surfaces at velocities of up to 90 km/h. A highly parallel processing system, it will use pipelined processing hardware and one-pass scanline-based processing algorithms.

2 The Need

More economical, timely, and reliable feedback is needed about the performance of the materials and construction methods used in road building. This paper addresses the area of high-speed surface inspection using machine vision system components, and in particular its application to pavement defect characterization. This section gives an introduction to some of the problems involved in pavement inspection, current ways of implementing pavement inspection, the shortcomings of these methods, and why MV would be a good solution to the problems of pavement inspection. Also, previous attempts at using MV for pavement inspection are reviewed.

The purpose of visual inspection of pavement is to check for defects in the pavement surface and subsurface structures that may influence the serviceability of the road. The system of roads is one of the major factors in the economic growth of a country, and a significant percentage of the country’s national budget is usually allocated for road construction and maintenance. In recent years, maintenance and
rehabilitation of existing roads have taken priority over new construction, thus making information about the condition of road surfaces extremely important (Haas et al., 1990).

2.1 Pavement Management Systems
The maintenance of roads must be put on a firm scientific basis if limited resources are to be efficiently utilized. This is accomplished through the use of a pavement management system or PMS (Haas & Hudson, 1978). The PMS concept is currently being adopted around the world by road administration authorities who are trying to find more scientific methods to apply to the problem of road maintenance (Yeaman & Lee, 1979). Road-surface data has become an integral part of the decision-making process that road administrators utilize to make the most cost-effective decisions concerning pavement rehabilitation. Two of the most important outputs of a PMS are (1) the establishment of a serviceability index, representing pavement quality, and (2) a prediction of future pavement performance, represented by the relationship between time (and/or traffic) and the serviceability index (Hudson et al., 1979).

The pavement quality index is highly dependent on four factors: roughness, rutting, skid resistance, and surface distress. Roughness is basically the spectral (frequency and amplitude) content of the longitudinal profile of the road and how it reacts with the mechanical dynamics of a vehicle’s suspension and tires. Rutting is the lateral (cross-pavement) profile of the road and shows typical wear patterns called ruts (one for each wheel track). Skid resistance is the degree of change in the frictional properties of the road surface in the presence of water. It is related to two other properties, pavement surface texture (macro- and micro-texture) and the surface’s geometry (which determines the degree of water drainage). The last factor, surface distress, refers to cracking patterns in the surface, in terms of frequency, total surface area, and the length, width and depth of the crack (if large enough, the “crack” is a pothole).

The measurement of these pavement quality factors should be accurate, objective, and efficiently acquired. This implies some degree of automation in their measurement. Of these, roughness and rutting are relatively easy to automate, skid resistance is somewhat less so (except through measurement of texture and geometry), and surface distress is relatively difficult to automate. The first three factors are global features, in that they tend to be a characteristic of relatively long stretches of pavement, thus allowing their measurement in a statistical sense. The final factor, surface distress, is highly localized and therefore much more difficult to measure.

2.2 Road Surface Measurement
The need for a systematic means of acquiring and processing information about road-surface conditions is recognized as a critical component of any comprehensive pavement management system. Significant advances have been made in the development of non-destructive testing equipment to measure roughness, rutting, and even skid resistance (Hudson et al., 1987). With respect to pavement distresses, however, these authors concluded that

...this component of pavement automation is currently least automated and most needs the application of innovative approaches in order to reduce the subjectivity, and increase the reliability and cost-effectiveness of distress data collection.

Unfortunately, much of the road-condition data presently used as input into PMS and road databanks is acquired manually—a slow, costly, error-prone, and labor-intensive process of subjective visual inspection or individual measurement of road parameters (NCHRP, 1981). Consistent ratings require a considerable amount of training and particular care in cross-validation by more experienced staff.

The evaluators “walk the pavement” (a dangerous process) and record the defect types, extent of distress, and severity levels. The data is usually written down, or a “map” of the cracking patterns is drawn, although newer and more accurate methods use hand-held computerized dataloggers (which reduces transcription errors). Because of the time and expense involved, the pavement examined is always a
sample section of the entire pavement to be rated. Distresses appearing in the observed (sampled) sections are used to predict the overall state of the pavement.

The consistency of subjective ratings (even amongst highly experienced highway engineers) was tested in Sweden several years ago, with results indicating a low correlation between observers, and even among the same observer’s ratings over time (Magnusson & Arnberg, 1976). A U.S. Federal Highway Administration (FHWA) report also documents the limitations of manual visual inspection (NCHRP, 1981). Visual surveys tend to be extremely subjective, and almost always lead to inconsistencies in distress detail over space and across evaluations. In addition, they require considerable time and resources for data collection. Given current constraints on resources, visual methodologies are limited to very small sample sizes, which also (from a statistical viewpoint) lowers both accuracy and reliability. Reliability also suffers from record-keeping and transcription errors, and infrequent updating of the data. The primary advantage of manual visual inspection techniques, however, is the verification of the actual location, frequency, and type of distress. Under proper conditions the human inspector can easily recognize the presence of distresses and properly scale the size, extent, and severity of a distress.

Finally, the diagnostic value of pavement surface ratings is critically dependent on tracking changes over time and correlating such changes with other road variables such as construction type, road usage patterns, etc. This in turn implies a timely and frequent appraisal of road surface conditions, something currently too expensive and time-consuming to be done by manual methods. Hence, more automated methods for pavement inspection are critically needed.

To collect the necessary pavement quality data efficiently, a number of different automated measurement devices are used. Several different automated pavement measurement systems are in use worldwide. They vary in their use of passive sensing (photologging) versus active sensing (lasers or ultrasonics). The problem is that none is completely adequate for the measurement of pavement cracking.

### 2.3 Automated Measurement Systems

In determining the rut depth of the pavement driven over, the Laser RST system samples the distance from the van platform to 11 points on the pavement at a maximum rate of 32,000 samples/sec from each point. The 11 laser range-finders (laser RFs) are mounted on the front bumper (2.6 m in width) with two lasers angled outward to allow coverage of up to 3.2 m road width. An averaging process combines 128 sequential measurements from each laser RF into a single mean depth value at each of the 11 points. This averaging process improves the accuracy of global measures of pavement characteristics, such as rut depth, by reducing measurement noise variance. Global characteristics such as rut depth are assumed to vary relatively little in a longitudinal direction, so this statistical averaging process is very effective in improving reliability and accuracy. Rut-depth measurements are considered relatively static in a lateral direction also. A cross profile from the 11 lasers is produced for each averaged sample, and from this the maximum rut depth is calculated.

The resolution required for the measurement of cracks and macrotexture is much tighter. Macrotexture influences many aspects of road performance. Surface macrotexture can be considered either a local feature (such as spalling) or a global feature (general pavement texture). As such, it must be sampled at a high enough rate to capture its smallest features and yet also be summarized for output as a global pavement characteristic.

Cracks are seldom characteristic of long stretches of pavement but are instead manifestations of short-term, “local” changes in the pavement surface. The Laser RST measures cracks at 4 points across the front of the van, at the full rate of 32,000 samples/sec. At the maximum survey speed of 90 km/h, the van is traveling at 25,000 mm/sec, measuring the distance to the pavement every 0.8 mm (0.03 in). At its highest speed, this system can accurately resolve a 2-mm feature. With slower survey-speeds, this is improved in proportion to the speed reduction.

A crack is detected as a sudden increase in depth. The width of the crack is measured as the time it takes for the depth discontinuity to reverse itself. The laser RFs are connected in pairs for crack measurement, and a crack recorded within a specified distance by both cameras is considered as a single
transverse crack. The Laser RST’s crack measurements give an indication of the overall severity of road cracking and the average width and depth of the cracks.

### 2.4 The Problem

The Laser RST is a state-of-the-art technological solution to an extremely difficult problem: the timely, economic, yet accurate and repeatable measurement of thousands of kilometers of road surface. Currently, there is no better system for road-surface testing than the Laser RST. This road measurement system is:

- highly flexible and adaptable
- easy to operate, requiring less training than other systems
- easy to maintain
- reliable

The Laser RST also:

- provides uniform and accurate data collection of many variables simultaneously
- does not interfere with existing traffic
- has flexible data output formats

It thus replaces low-quality, subjective, labor-intensive work with a high-quality, objective, economic measurement process.

Still, a major shortcoming in the Laser RST system is the lack of lateral resolution for the detection of local features. This is most clearly shown in Figure 1. The crack labeled (A) is not detected at all while the single meandering crack (B) could give rise to spurious multiple crack counts. In addition to these crack counting errors, individual crack widths cannot be measured accurately with the current system. The width is usually defined as being perpendicular to the major axis of alignment of the crack. With the current system, only the widths of fully transverse cracks are measured accurately (crack C). Cracks at oblique angles (D) give width readings that are too large (in proportion to the crack’s angle of inclination, Figure 2).

The problem is the lack of lateral resolution in the measurement. While more than adequate for global features that tend to have little change over relatively large stretches of pavement, the Laser RST’s use of four sensing points laterally across the road lane is not adequate for measurement of more local features in which closely adjacent spatial information is needed to establish such characteristics of a local feature as its connectivity, orientation, etc.

As noted above, the problem is a lack of lateral resolution. At 32,000 samples/sec, longitudinal resolution is more than adequate. One obvious and feasible solution would be to use information from all 11 laser RFs to determine cracking patterns. This would nearly triple the lateral resolution while requiring minimal changes to the Laser RST system (simply the addition of more processing modules). The question is whether this be enough sensed points to properly discriminate between the various cracking patterns shown in Figure 1. With 11 lasers, the lateral resolution would be approximately 0.3 m.
2.5 Count Buffon's Needle

This situation is reminiscent of a classical problem in Monte Carlo simulation, called Buffon's Needle Problem. This is one of the earliest known examples of the use of probability games for the purpose of ascertaining physical parameters. It was posed by Georges Louis Leclerc, Comte de Buffon (1733). Buffon used an empirical process (counting the number of times a dropped needle touches a line drawn on the ground) to calculate the value of \( \pi \). Since we now know the value of \( \pi \) to any desired degree of accuracy, this solution is only of historical interest. However, the paradigm can be reversed and applied to the crack detection problem of the Laser RST. That is, knowing \( \pi \), it should be possible to predict the expected number of line crossings for a specified “needle.” If Buffon’s lines drawn on the ground are considered analogous to the laser sensor tracks, and the randomly dropped “needle” of length \( L \) is conceptualized as the random appearance of a crack of length \( L \), then the event frequency with which a crack will “touch” a laser sensor’s path can be characterized as the probability of detecting that crack.

Buffon's Needle problem is formulated as follows. If a needle of length \( L \) units is tossed randomly upon a floor composed of parallel lines \( T \) of separation \( d \) units (with \( d > L \)), what is the probability that the needle will cross (or touch) one of the lines (of conceptually zero width)? In the present application, the parallel lines \( T \) are the laser RF sensing tracks (Figure 1), with separation \( d = 0.3 \) m, and the needle is really a crack of length \( L \). Assume that the resting position of the needle/crack can be uniquely described by a pair of random variables:

\[
Y = \text{the distance from the crack's midpoint to the nearest sensor track } T \text{ and} \\
\theta = \text{the acute angle between the sensor track } T \text{ and the crack}
\]

By assuming that:

- \( Y \) and \( \theta \) are independently distributed random variables
- \( Y \) has a rectangular distribution over the region between 0 and \( d/2 \), and
- \( \theta \) also has a rectangular distribution, yet over the region between 0 and \( \pi/2 \)

then in accordance with Figure 3, a toss of the needle (the probabilistic event of the appearance of a crack on the road surface) corresponds to the specification of a set of coordinates \((\theta, Y)\) within the rectangle of length \( \pi/2 \) and height \( d/2 \), and of area \( \pi d/4 \). Only those coordinates falling beneath or upon the curve

\[
Y = \left( \frac{L}{2} \right) \sin \theta
\]

correspond to cracks that cross or touch one of the sensor tracks (and is thus detected). This is the equation for a sine curve of amplitude \( L/2 \). Since the coordinates \((\theta, Y)\) are uniformly distributed over the rectangular region of area \( \pi d/4 \), the probability \( p \) of the crack’s touching or crossing a sensor track (the probability of detection) is given by the proportion of this area beneath the sine curve (Figure 3) from \( \theta \) to \( \pi/2 \):

\[
p = \int_{\theta=0}^{\pi/2} \left( \frac{L}{2} \right) \sin \theta \ d \theta / (\pi d/4)
\]

\[
= \frac{2L}{\pi d}
\]

That is, the proportion of the total rectangular area contained by the sine curve. Buffon suggested that, by tossing as randomly as possible a needle \( n \) times upon the floor, one could compute \( f \) the relative frequency of occurrence of the needle’s contacting a line. For \( n \) sufficiently large (a large number of trials), one could anticipate that:

\[
f \approx p = \frac{2L}{\pi d}
\]

Alternatively, since \( L \) and \( d \) are fixed lengths, Buffon noticed that one could estimate the constant \( \pi \) by means of the inverse expression:

\[
\hat{\pi} = \frac{2L}{fd}
\]
Figure 3: Buffon's Needle.

Once the ratio \( f \) of crossings to tossings had been experimentally determined. In the present application, since the value of \( \pi \) is known, the frequency of occurrence of the needle's contacting a line can be computed directly, without empirical trial.

Assuming the normal measurement conditions (using four laser RFs for crack detection), the sensor separation distance is then \( d = 0.9 \) m. For a minimum 50% detection probability, the crack length \( L \) must be greater than:

\[
0.5 = \frac{2L}{0.9 \pi}
\]

\[
L = \frac{\pi dp}{2}
\]

Hence, any crack less than 0.71 m (28 inches) in length will not be detected the majority of the time (greater than 50% of the times it appears).

Consider now the case of using all 11 lasers. The probability of detecting a crack the same size as the sensor separation distance \( (l = d = 0.3 \) m) would be \( p = 0.63 \), i.e., such a crack size would be detected 63% of the time. The crack length giving a 50/50 chance of detection in this case would be 0.23 m (9 inches).

From the above analysis, it would appear that while increasing the number of crack sensors to 11 would be a definite improvement, it would still not provide the required discriminatory power for accurate crack detection, much less proper crack measurement.
It is necessary to note a very important caveat, that the above analysis assumes the presence of only one crack (event) at a time. It also does not address the case where \( L > d \), so that a single crack can span several sensor tracks. In most real-world situations there are both long and short cracks, and more than one crack can occur at a time. In these cases, the probability of at least one laser detecting a crack increases. This perhaps explains why the Laser RST is still relatively successful at characterizing the degree of cracking present. It looks at cracking in a statistical sense, instead of counting each crack—some cracks will be missed but others will contact multiple lasers or be contacted multiple times by a single laser (as in the meandering crack of Figure 1). It assumes that it all “averages out,” but that the average is nevertheless indicative of the overall level of cracking present.

This again brings up the earlier discussion of global versus local pavement characteristics. Cracking is characterized in the Laser RST in the global sense; individual, localized cracking features such as cracking patterns cannot be measured. With the current Laser RST, there is still the difficult problem of distinguishing a “crack event” as being a single crack spanning multiple sensor tracks versus multiple individual cracks. With the present system, it is possible only to distinguish a single crack spanning several sensor tracks when it is fully transverse (horizontal). If sufficiently oblique, it is not possible to distinguish it from multiple transverse cracks.

Finally, the above model based on Buffon’s Needle assumes that cracks have negligible width, that the crack has a single uniform orientation (no bends or branches), and that all orientations are equiprobable. All of these assumptions are false in the real-world.

Of course, the spot size (resolution) of ultrasonic systems means that ranging systems based on ultrasonic transducers cannot characterize cracking at all—the “lines” drawn on the pavement by an ultrasonic transducer are so thick that only the largest cracks are detectable as such.

3 Machine Vision as a Solution

The present need of the Laser RST system is to more accurately characterize cracking in road surfaces. The primary problem is one of increasing the lateral sensor resolvability while providing the ability to examine spatial connectivity in a longitudinal direction. A 2D sensor with the necessary horizontal and vertical resolution currently exists in the form of a photodiode array found in CCD video cameras. The proper processing for establishing connectivity and making measurements currently exists in the form of commercial machine vision systems. With a machine vision sensor, the number of sensor tracks is increased from 4 to 2048 or more. Also, current machine vision systems have the ability to extract important shape parameters from the sensed cracks, such as angle of orientation, area, width, “roundness,” etc. The current laser RF system would still be needed for measurement of the global features, such as roughness, rut depth, etc., features that a machine vision system could not measure. Moreover, the local depth information available from the laser sensors would be useful in discriminating those distress features lacking depth, such as filled cracks. In fact, a very important task would be to properly integrate these two sensor systems to maximize system performance.

Machine vision inspection is a technology that has come of age; it is now able to handle a wide variety of inspection application needs. This technology is being used in a large number of very diverse applications in a wide variety of industries. Image data dominates, in terms of information content, the outputs of man’s sensory receptors. Automation of the image analysis and interpretation processes, aimed at replacing the human operator, poses a very fascinating and extremely challenging problem.

While current commercial machine vision systems are very reliable and capable, it is important to examine their use within the constraints of the Laser RST application. The advantages of the use of machine vision for processing of local feature road surface information include:

- accurate detection of cracking and similar local road distresses
- measurement of distress parameters, such as area, bounding rectangle, percentage of total surface area, etc.
• ability to acquire and store visual images of the road surface for later retrieval and review, making an image database of road conditions possible
• ability to detect (in conjunction with the laser RFs) cracks that have already been patched, outputting reports on the degree of maintenance already conducted on a road unit
• detection of local macrotexture features, such as spalling

The machine vision system would need to work closely with the current laser RFs because the vision system operates on scene luminance (contrast), rather than range (distance). The two systems are complementary in this sense. The laser RF is an active sensor system that measures distance directly, while the video system uses a passive light detector array, and so needs an external source of energy (lighting). It does not measure distance but instead measures changes in brightness across the spatial plane imaged. Since cracks (filled with debris, or perhaps patched) generally differ in contrast from the background pavement, they will be detected and measured. Information from the laser RFs would be used to discriminate if the cracks were filled (no cracking detected by laser RFs) or not (cracks also detected by laser RFs). The laser RFs would also be necessary to help the machine vision system reject painted markings on the pavement (these would have a different contrast like cracks but would have higher contrast than cracks and no depth changes). It is necessary that both contrast and distance sensing be included in the RST system. The limitations of machine vision systems must also be kept in mind, including:

• requirements for extremely high computation rates
• specialized lighting needs
• high capital costs

Of the above, the most critical limitation is processing speed. Anyone who has worked with images knows how to quickly overwhelm most computer systems. A single image, represented as a 512 x 512 array of picture elements (pixels) with 8 bits brightness information (256 levels of gray), takes more than a quarter-megabyte of storage. And few applications use only one of these images. The system can spend most of its time just shuffling pictures between main memory and secondary storage devices. Total data flow-rates for image-processing systems are on the order of 15,000,000 bytes/sec (considering only data transfers, without computation). This is beyond the capability of even the latest generation of microprocessors. While such “ultrafast” systems do exist commercially, this is pushing the limits of most computing equipment, so selection must be done very carefully.

3.1 Previous Attempts

Initial applications of machine vision technology to road transportation systems have primarily involved automated traffic monitoring. There have been several attempts to perform traffic analysis using MV, particularly where dense traffic has the potential for congested or hazardous conditions (List et al., 1989). Other uses have included traffic flow management, vehicle-based collision avoidance, and rear-view “mirrors” for buses. More publicized attempts have included the automated acquisition of license plate numbers.

Research into automated pavement distress analysis has concentrated primarily on the feasibility of automated technologies for pavement distress analysis. These studies have investigated and compared hardware and software needs, data collection procedures, merits such as cost and accuracy, and limitations of automated technologies for data collection. The identification of potential uses of machine vision for pavement inspection was initially investigated by Wigan & Cullinan, 1984. One of the motivations for their work was that while the costs of machine video systems were declining, the costs of manual pavement inspection were steadily increasing.

There are at least three different ways in which digital processing of pictures of road surfaces could be used in determining road surface ratings (Wigan & Cullinan, 1987). One approach would be to compute a single, overall characteristic visual texture of the surface that would be sensitive to cracks, patching, and other surface defects. This rating could then be added to subjective visual rating factors for specific types of cracks; over a period of time both could be jointly assessed for their value in terms of correlations.
between different manual raters, and with road performance factors over time. This more statistical approach to cracking is similar to what is currently used by the Laser RST.

A second, more complex and demanding approach would be to use machine vision technology to identify the presence of cracks, patches, and other surface features automatically, and to count the number of cracks present (irrespective of crack type). Finally, a third technique would be to extend this simpler crack counting procedure to a complete crack recognition and pattern characterization. Such an approach would allow the analysis of various crack types and their dimensions, such as total length, total cracked area, etc.

Of these three, Wigan and his colleagues attempted only the first by using a general texture-based approach to generating a simple road-surface rating. They have examined surface texture using the response of specially designed convolution masks to classify the general texture properties of the road surface. These masks are variations of the Laws texture masks. Their experiments tested the texture discrimination performance of this mask set against a set of artificially structured textures (disks, checkerboard, vertical and horizontal bars), artificially stochastic textures (noise patterns), and several natural textures, including grass, raffia, sand, and several road surface samples. Application of MV to crack categorization was discussed by not attempted.

Mendelsohn (1984) reviewed the applicability of various technologies to automated pavement crack detection. Perhaps one of the first systems developed specifically for the analysis of surface distress data was the ADDA (Automated Pavement Distress Data Acquisition and Evaluation) system (Haas et al., 1985). ADDA was developed by the University of Waterloo under the sponsorship of the Ontario Ministry of Transportation. The system utilized a video camera mounted on a vehicle traveling at a maximum speed of 30 km/h. The width of the field of view of the system at its midpoint is 14 ft at a camera angle of 35° from the horizontal. The resolution of the system is 512 × 512 with 128 gray levels, yielding a resolution of about 0.32 in. Resolution is further reduced during processing by intermediate storage on a VCR and by blurring in the image due to the forward motion of the vehicle. The images were analyzed using horizontal and vertical edge masks, counts of each in each subimage, and then a comparison of counts across subimages to classify cracking types (longitudinal, transverse, and alligator). Performance was generally poor, with a great many other types of image objects all being classified as alligator cracks.

Mahler (1985) discussed the design of an automated pavement crack measurement system for use in a vehicle. The Automated Crack Monitor (ACM) was designed to measure cracks in the roadway surface and was developed by KLD Associates Inc., under contract to the U.S. FHWA. The design used two video cameras, which provided a field of view of 3 ft by 2 ft and a resolution of 0.05 in (1.25 mm). A high-intensity gas-discharge strobe light was used to illuminate the pavement surface. This light source froze the images while traveling at a speed of up to 40 mph. The imaging limit was 6 fps, giving a maximum sampling rate of one 3 × 2 ft section every 20 ft (about 10% sampling). The coverage rate could be increased by decreasing the speed of the vehicle.

Cox et al. (1986) reported on a sophisticated system designed to analyze images of developing crack areas and other surface features. They then addressed the problem of combining the various crack measurements into a simple cracking index.

Some work has been done on the off-line analysis of the 35-mm film generated by vehicles such as the GERPHO. Caroff et al. (1989) discuss such a system, called the MACADAM system for Method for Automatic Classification and Analysis of Distresses for Aided Maintenance. The GERPHO (Groupe d’Examen Routier par Photographie) system is similar to the Japanese ROADRECON in that it employs a survey vehicle to take continuous 35-mm photographs of the pavement surface. It was developed by the Laboratoire Central des Ponts et Chaussees (LCPC) of France (Ministere des ’Equipment) and has been used extensively in France since 1972.

The GERPHO vehicle uses a slit camera delivering a continuous film of the pavement surface at a scale of 1/200. Normally, the film is analyzed manually by trained technicians using a standard manual pavement distress rating system—judgements of distress are solely by trained observer, same as would be done in the field. The primary difference is in the method of entry of the distress data into a computer database. In the standard GERPHO system this is done off-line back at the office instead of out in the field.
field. Also, the data is entered directly from the film using a manually operated pointer to outline the surface distress, instead of having the observer record the distress for later entry. Observing conditions are less hazardous than in the field, and somewhat less fatiguing, but the basic problems of subjectivity and boredom are still present. Somewhat improved is the transcription of the data. Many mistakes in simply writing down or entering the data occur in the field. These are alleviated by the computer-aided data entry when the film is analyzed in the office. Indeed, the computer does a great deal of the measurement (such as determining the crack dimensions).

Caroff et al. are currently developing a system that automates the above process by digitizing the film images and using MV algorithms to analyze these for surface defects. Because the film can be digitized at very low speeds, real-time processing is constrained only by economic considerations. As of February 1989, this system was still in development, although a limited prototype has been built and tested.

The MACADAM system is designed to distinguish between three different distress categories: longitudinal cracks, transverse cracks, and alligator cracking. A Perkin-Elmer micro-densitometer is used for the scanning and digitization of the film, and the data is fed into a SUN-3 workstation. The majority of the processing is then done by software (although a more hardware-based system using a VICOM VME-buss MV system is under development). Furthermore, the algorithms selected are not optimized for processing efficiency. Although, as expected, the system is very slow (on the order of 12 seconds per image), it clearly demonstrates the feasibility of the basic MV approach to pavement analysis. Interestingly, Caroff et al. use an expert-system based approach to higher-level image analysis.

Fukuhara et al. (1989) discuss the design of another high-speed MV analysis system for off-line analysis of surface distresses in 35-mm film photologs of the pavement surface. Their system is designed for the output of the Komatsu Road-Man. The longitudinal measurement accuracy is dependent on the speed of the vehicle; that is, a vehicle speed faster than 10 Km/h will "outrun" the processing, and wider-spaced samples will be taken. The system uses a structured light approach. A single 4000-pixel element linescan camera is mounted on the front of the vehicle bumper, pointing forward and down. The camera must use a photomultiplier because the separation from the pavement is great enough that returning light is greatly attenuated and must be amplified before entering the camera. Also, the light source, a swept IR laser beam, is relatively weak and is the same one used for rutting analysis. The scanned laser beam spans a four-meter wide section of pavement, hence the resolution is on the order of 2 mm for the 4000-element camera.

As in the French system, the initial development work has centered on off-line analysis of data. The video from the camera is stored on magnetic tape in the vehicle, using a special high-density (100 Mbpi) VCR. The post-processing system is unusual, being software-based and employing no specialized image processing hardware. Because of the inherent slowness of this approach, it is obviously necessary to employ massively parallel processors. Over five-hundred 32-bit processors (MC68020) are used, in addition to seven specialized parallel processors (INMOS Transputers). This brute-force approach to image processing is not without its problems—in such a massively parallel system, it is difficult to manage all these processors without incurring excessive overhead from the increased system complexity.

Analysis of the cracking is primarily by streak-featurehistograms, a simplistic form of the radon transform. The image is divided into multiple 32 x 32 subimages (referred to by Fukuhara et al. as "slits") by a single MC68020 microprocessor, called the "survey processor". The gray values are accumulated (streaked) across rows to form a vertical streak histogram, and likewise down the columns to form a horizontal streak histogram. The two resultant gray-level histograms are then a single feature pair \(h_v, h_r\). The 32 x 32 subimage is then rotated by an angle \(\theta\) and the histograms calculated again. This yields the final feature data for each subimage, \((\theta, h_v, h_r)\), each such feature data vector being calculated by a single MC68020 processor.

The data from each of the subimages must then be combined together; this is the task of the Transputers. They determine whether a crack spans several subimages, determining connectivity, linearity, etc., based on the relative arrangements of the individual crack subcomponents. Isolated components are removed, and collinear and proximal components are merged.
As we have seen, surface distress surveys are critical to modern pavement maintenance systems, yet are still conducted for the most part by visual inspection. With the advent of video technology, a new generation of distress survey vehicles has been developed in which images of the pavement surface are stored on 35-mm film, video tape, or video disk. This allows the inspection of pavement surfaces to be performed indoors, therefore saving some time and money, but primarily improving safety of the inspecting personnel compared to in-field inspection. The next logical step after that is the automatic analysis of the collected distress data.

An American company, PCES (Sparks, Nevada), has been attempting to develop a fully automated pavement analysis system for several years now. A recent report detailed the status of their Pavement Distress Imager or PDI system (Butler, 1989). The company is now in the process of being liquidated. Like the Komatsu system, it is software-based, which extracts a speed penalty, but unlike the Komatsu system, it is designed to provide analysis “on the fly” while in a van. Given these constraints the system performance is very poor. Although claimed to be real-time, the processing cycle is 250 msec, which is not fast enough for full processing at even moderate vehicle speeds. The pavement surface is only sampled (every third section) and is not continuously analyzed.

The PDI uses two 512-pixel linescan CCD cameras to image two 4-ft by 4-ft areas of the pavement. An unusually large amount of illumination (15,000 watts) is provided by banks of incandescent lighting, and an on-board diesel electric generator. The luminaires and cameras are mounted in a shroud on the underside and toward the rear of a step van. Two banks of luminaires, front and back, 13 inches off the pavement, are mounted at a 35° angle to the pavement to maximize the contrast between the pavement surface and the shadows created by pavement cracking. This bidirectional lighting with perpendicular camera position is argued to improve the sharpness of the crack images and to minimize spurious shadows.

The camera signal is digitized and corrected for dark current reference (normally not needed, except perhaps for the high levels of heat generated by the lighting). Next, nonuniformities in illumination are compensated for by using a light-field technique. A histogram is then computed and a threshold selected for image binarization. Each 4-ft by 4-ft binary image is then divided into 1-ft² subimages, and a simple pixel count is taken. The number of pixels above threshold in each such square is taken to be an indication of the amount of distress present (but not the type or frequency of cracking). This simple pixel count is the primary data output from the system; no further analysis is done, except through operator intervention.

Only the most crude measurement of overall cracking (in a statistical sense) is output. A single measure of cracking (actually, any kind of distress) is created per square foot of pavement surface. The system cannot, for example, distinguish between longitudinal and alligator cracking. The system requires that an operator watch three monitors displaying images of the surface underneath and enter “on the fly” an event code to help interpret the output (events such as indications of patching, changes in pavement type, bleeding, skid marks, etc., plus the distress categories such as alligator, lateral cracking, etc.).

Researchers at the Texas Transportation Institute (Texas A&M, College Station, Texas) have begun to address some of the fundamental issues in automated pavement distress analysis. Chan et al. (1989) examine the problems involved in using simple thresholding techniques for distress identification. The paper then discusses the use of Kittler’s thresholding techniques for use in a scanline-based pavement distress analyzer. This research has only now begun, but the ultimate goal is an on-line analysis system. It is expected that a full on-line analysis system will be mounted on a vehicle (perhaps an ARAN) within three years.

3.2 Problem Definition
The basic inspection requirements that influenced the selection and design of the MV solution are:

- size, type, and frequency of the defects have to be determined; size refers to the percent area involving each defect type; frequency refers to the number of instances of each defect type encountered
- the decision that a defect type is present is made by applying a set of context-dependent rules (initially written for human inspection) to the detected and measured defects
The system has to provide high measurement and recognition accuracy and to perform at high speed in order to be considered for actual highway use.

The above requirements translate into the following ones relevant to the MV system design:

- high resolution for small surface defects
- a large amount of pixel data because of the comparatively large dimensions of the surface
- variable illumination because of the requirement to operate out-of-doors and any time of day or night
- variable surface contrast because of the variety of defects and surface characteristics
- special hardware architectures because of the very high-speed inspection requirements
- accuracy and robustness of defect detection and classification in the face of highly variable defect characteristics
- flexibility with respect to changes in surface structure, defect patterns to be inspected for, etc. (to match user requirements)

In terms of speed requirements, the on-line inspection task has to be performed very fast. This is necessary to avoid slowing down the laser-based portion of the total road inspection system. This is also necessary to make the system cost-effective. A slower system would require that video data be logged to disk or magnetic tape for later off-line analysis. This is extremely expensive in terms of storage media, handling costs, etc., but still requires a special architecture to analyze the tapes. However, the requirement for real-time analysis makes the whole problem of finding a suitable combination of sensing, algorithms, architectures, optics, lighting, etc., an extremely demanding task.

The current system, which gives a characterization of the pavement surface in terms of frequency and severity of surface defects accumulated over a section of road (say, every 20 m), is a much more efficient system. What is lost, however, is the raw data, which can be important to certain users, particularly research groups who wish to track detailed changes in the pavement over time. In an attempt to satisfy this need, the PAVUE system will include a photologging option, which will store selected defect patterns when present in an image and/or when a certain defect severity threshold is exceeded; these video images will then be sent to off-line storage. This intelligent selective videologging is obviously more efficient in storage space and time than continuous videologging, as long as the selection criteria can be specified accurately enough.

### 3.3 What is Expected from a Crack Measuring System

A somewhat informal survey of surface distress classification systems in use in 48 state transportation departments in the U.S. was conducted by PCES (1986). This survey found that the 41 states contacted gather some form of pavement surface condition data, all using manual methods. These varied from walking and rating segments sampled from the overall road network (13 states) to driving at a slow or moderate speed and rating a full section of road (12 states), to driving at normal speed and slowing to rate a sample segment (10 states). Nine states used raw surface-distress data, and nine others combined several surface-distress measures into a single surface-distress index, which was later combined with other indices in a PMS or graphical display system. Nineteen of the states surveyed used surface-distress data along with other information such as roughness, friction, traffic loads, etc., to form a single serviceability index for each section of road.

A more recent survey was undertaken to find out what type of data should be provided by a pavement distress analyzer (Arnberg et al., 1990). The above review of current manual pavement distress survey classification systems for several countries and for several states in the U.S. showed some areas of general agreement concerning measures of the type, amount, and severity of distress. However, there were also some areas of ambiguity, confusion, and even contradiction among these various distress measurement "standards." The survey was sent to road engineers in several countries (including Denmark, Great Britain, Australia, Saudi Arabia, and Sweden) and to several state DOTs in the U.S. It first asked respondents to rate the importance of cracking data with respect to other road survey measures such as roughness, texture,
rut depth, etc. Other questions asked about the uses of cracking information in discriminating between the effects of climate, traffic load, construction practices, etc. Respondents rated how important it was to have knowledge of the various dimensions of cracking (length, orientation, area, position, etc.). Five different road surface zones were defined (left edge, left wheelpath, midroad, right wheelpath, right edge) in an attempt to prioritize the importance of the location of cracks. Questions were also asked about the full lane width which should be surveyed and the minimum acceptable crack width. An attempt was made to examine the relative importance of various crack categories, using drawings of different patterns to avoid problems with contradictory and obscure terminology.

4 PAVUE I

Initially, video systems have involved primarily a process of videotaping the pavement at relatively high survey speeds, returning the tapes to the office/lab, and having a trained observer watch the tapes and conduct a manual assessment of the cracking. This is an improvement over the field collection of cracking data, since it is much safer and more comfortable for the rater. The videotape/manual assessment approach would not appear to be either more or less reliable or accurate than a field survey, but there have been no studies examining this. The videotape approach may be less reliable and perhaps even less accurate. The added comfort can induce more boredom and hence reduce reliability. Most raters will say that watching a video tape of pavement is even more boring than field-observation. On the other hand, there are less distractions, the tape can be stopped for breaks, and the playback speed can be varied (somewhat) to try to improve the reliability and accuracy of the rating. Accuracy may suffer, however, since the rater is limited to only a single view of the pavement.

The primary advantage of off-line manual analysis would seem to be its savings in cost, but this is primarily in terms of safety costs, since labor costs (for raters) are approximately the same in both the videotape and field observation systems. Unfortunately, even this savings may be offset by the increased costs of the extra equipment involved in collecting and displaying the videotapes. A final point worth noting is that this type of off-line manual videotape crack survey system is relatively speed independent, since the human observer used can in most cases discriminate and compensate for most types of image overlap.

The next major technological advancement would be to have a computer analyze the videotapes, removing the primary source of unreliability in the above system, the human rater. The system would be much more reliable—the analysis program run over several repetitions of the same pavement videotape will yield highly correlated results. However, there can be a significant loss in accuracy. Human perception is an exquisite image processing system, still better than anything that humans can build. But as noted it is extremely unreliable, and prone to boredom, fatigue, and biases. An MV analysis system will in general be less accurate than a human rater and would make more errors (although being more reliable, it will make the same errors each time).

In addition to an increase in reliability, such a machine-vision-based videotape analysis system would be more economical, replacing the costly human raters, and would operate at much higher analysis speeds. It would be capable of inspecting more kilometers of pavement, and hence network-wide statistics generated would be improved. But the primary problem is still the lack of basic accuracy in the machine-analysis process. The root causes of the poor accuracy are:

- speed dependent effects (multiple crack counting)—it is difficult for a machine to detect the degree of image overlap and compensate; the use of specially-constructed variable-speed VCRs would be too expensive
- low image resolution—standard VHS-type VCRs have very limited bandwidth (240 pixels horizontal), meaning that the image is blurred; this lowers the effective resolution of the system
- poor algorithms—image processing for inspection is still an ad hoc field, more art than science; proper selection of algorithms is difficult
In addition, there is a tradeoff between speed and accuracy—as the computer analysis is made more accurate, it also tends to operate more slowly. The result is that even for minimally acceptable levels of accuracy, computer analysis can be very slow. The more accurate the algorithm, the more complex the implementation and the slower the processing. For efficiency, most of the image processing algorithms must be encoded in hardware, not software.

The PAVUE I system attempts to improve the accuracy of the basic computer analysis of a videotape via use of:

- 5 channels of video (5 video cameras and 5 S-VHS VCRs), giving an effective resolution of over 2000 pixels across a 5-meter wide path; S-VHS recorders have a horizontal bandwidth of 400 pixels
- special circuitry to allow speed independence of the processing, while using standard fixed-framerate video cameras and VCRs (either PAL or NTSC); pavement is analyzed as a continuous "ribbon" of data and not defined by frame boundaries.
- standard commercially-available video cameras, shuttered at 1/10,000 sec for minimal motion blur up to 90 km/h
- off-line analysis done on a specially-designed image processing computer (approximately 20 custom VME cards); the ultimate goal is a processing speed of 90 km/h
- capability of integrating video- and laser-based pavement information for higher system accuracy

In addition, the basic PAVUE system design provides the following significant advantages:

- modularity—the pattern recognizer is separately configurable for a particular customer’s crack classification system
- add-on to existing measurement systems with a minimum of modification
- unique proprietary, highly adaptive off-line analysis algorithms, which automatically compensate for different pavements and lighting conditions
- compact storage of results; output statistics and a crack map
- image database capability; random access to any user-specified section of pavement on the videotape

4.1 System Overview

A machine vision system is an obvious solution to the problem of high-speed surface inspection. And yet, the task constraints severely test the current state of the art with respect to both machine vision hardware and software components. For the target application area of highway pavement inspection, the problem constraints are formidable indeed:

- ambient lighting is inadequate and uneven, requiring external lighting
- cameras may need to be relatively close to the surface being inspected (0.5m)
- the distance from the camera to the surface is not constant but varies with vehicle motion
- the surface goes by at a maximum rate of up to 90 km/h (80 ft/s, 55 mph) and this speed also varies (though usually known)
- shuttering or other special techniques must be used to reduce motion blur
- surface texture varies greatly, over both space and time
- surface contrast varies greatly also
- a continuous ribbon of pavement (3-5m wide) must be measured, with distress patterns extending across multiple image frames
- it is a rugged environment (vibration, dust, rain, etc.)

As a solution, machine vision presents many challenges that have not been addressed previously. High speed puts a premium on proper hardware and software design for high throughput. The algorithms must support parallel-processing and extensive pipeline processing. All processing must be of a sequential scanline nature so as to not disrupt the pipeline. Those scan-line processes operating in parallel on multiple
parts of the image face a new problem—combining information across image boundaries. A “stitch-n-go” methodology allows the logical connection of multiple subimages across subimage boundaries (from multiple cameras) into a single coherent image. This also holds true sequentially across images over time, because the pavement is a continuous object spanning multiple images over time. Since the pavement surface forms a continuous ribbon of material, it is necessary to stitch together multiple images of a surface over time to form a single continuous analysis.

In addition, the presence of appreciable levels of surface texture complicate the defect analysis, as does the nature of the defects themselves. The defects can have any shape, size, and location, making simple template matching (the algorithm of choice for nearly all high-speed inspection systems to date) no longer feasible. There are simply too many variations in the defect patterns to generate any manageable set of templates.

Instead, a full connected components (blob) analysis must be done, with subsequent extraction of shape features for defect classification. However, the majority of such algorithms require global, random access to the image. This is not possible in the highly pipelined architecture necessitated by the processing speed constraints of the task. For this application a one-pass scan-line based analysis must be done, requiring that new algorithms be developed.

4.2 Camera System

The video camera used is a standard broadcast-TV type of camera, providing low-cost, availability, and ease of repair/replacement. It outputs a PAL (or NTSC) video signal and is therefore a fixed frame-rate camera, generating video images at a fixed number of frames per second (fps), 25 fps for PAL (or 30 fps for NTSC).

The camera uses square pixels to minimize distortion and inaccurate dimensional measurements. The use of rectangular pixels is not recommended for two reasons. First, a true square in the real-world would be measured as a rectangle; moreover, an object would change in dimensions depending on whether it was vertically or horizontally oriented. Secondly, all standard image processing operations (such as convolution) assume that the image is defined over a square grid. If it is not, errors result and are compounded by every succeeding processing step.

The camera must have some type of shuttering system to survey at even moderate speeds. Otherwise, motion blur will render the images useless. The camera is monochrome—the monochrome-equivalent signal from a color camera is inferior to that from a true monochrome camera because of chroma phase interference, fringing, etc. The use of a color signal is a waste of hardware (and significantly adds to the costs) when color information is not important, and in pavement distress analysis, color is definitely not important. Furthermore, the presence of color information in the video signal degrades the monochrome information. Finally, the use of a color camera reduces light sensitivity significantly, requiring 2-5 times more light.

All shutters significantly reduce the amount of light available for sensing. Mechanical shutters are simply too unreliable and not robust enough. Electronic shutters have their own problems. Most electronic shutters reduce the basic light sensitivity of the camera (apart from the reduced light integration time). The most important effect is that the vertical resolution is halved. The reason for this is that a standard NTSC or PAL camera outputs its image (frame) in two separate parts called fields. One field consists of all the odd-numbered lines followed by a field with the even-numbered lines. With many cameras, it is only possible to shutter only one field at a time and so only half the vertical lines are present in the frame.

The camera must be selected so that both fields are shuttered simultaneously, and then read-out sequentially to form the full-resolution frame.

The video interface will be five S-VHS recorders feeding signals into a VME camera rack with five camera boards (Figures 4 and 5). Each camera card contains a buffer interface to the camera (digital and analog signal termination and buffering), an analog video amplification stage (gain of 10, but programmable), an offset-correction circuit (summing amplifier driven by a D/A with offset correction RAM), a gain correction circuit (multiplying D/A with gain correction RAM), an A/D converter, and a FIFO digital
buffer providing an interface to the PAVUE pipeline. Timing circuits unique to each camera are also on each camera board. There is a single output from each camera board, the PAVUE pipeline interface. Certain control inputs to the camera board (the row address, the data values for the gain and offset correction RAMs) come over the pipeline interface, while the camera timing signals common to all clocks come from a single camera timing board, perhaps over the VME P2 connector. The camera timing board also resides in the camera rack and provides the digital timing signals for all the cameras (pixel clocking, shutter control, etc.); it is also controlled over the main system pipeline.

The PAVUE pipeline is unique in that it is not simply for the high-speed transfer of image data (as in common with most other image-processing pipelines), but it is also directional and can carry control and command information for modifying the parameters of the devices attached to the pipeline. Hence, it is possible to accept the data from the cameras at a high rate using the pipeline, while it is also possible to control their functions over the pipeline. Unlike other pipelines (which are simply high-speed parallel interfaces from one card to the next), the PAVUE pipeline nodes are “intelligent” and can be programmed to respond in unique ways to the data circulating in the pipeline. This provides a high degree of flexibility, adaptability, and reprogrammability which is not available in most other image-processing pipelines (such as Datacube’s MAXBUS or Imaging Technologies’ PixelBus).

The initial PAVUE prototype system is essentially identical to the design detailed above, with the exception of having only a single video channel for collection. The pavement data was collected using only a single camera mounted on a boom at the front of the vehicle and a single S-VHS recorder. All five zones (as in the full system) were collected, however, using multiple collection passes. To generate the video data for the five zones, five passes were made over each road section. With the final system, only a single pass over the pavement will collect data for all 5 zones simultaneously.

Ambient light and shadows could be a major problem. Placing the cameras on the front of the vehicle and near the lasers places a significant constraint on the choice of lighting. The primary reason is that the lasers operate on infra-red (IR) light, and if the light source for the PAVUE system puts out too much IR light itself, it will lower the effectiveness of the laser system or even prevent it from working at all. Since the laser system works at a relatively fixed frequency (850nm wavelength), it would be possible to filter this frequency out of the PAVUE lighting and so reduce interference, but this may not be practical. Normal incandescent bulbs output a large amount of their power at this wavelength, and the filter would need to be large and expensive, in addition to reducing the total light output from the lamps. The alternative is to use a light source with minimal IR output. In general, the constraints on the system lighting are:

- range of control—the light output levels will vary over the short-term (warm-up, environmental temperatures, air-velocity, etc.) and over the long-term (aging effects); hence, it is necessary to be able to sense and control the level of lighting in order to keep it as constant as possible
- spectral content—the spectral distribution of the light source should match the sensitivity curve of the sensor, while providing minimal interference with other processes
- flicker—because of the high shutter speeds involved, the light source should show no flicker, or at least have a flicker frequency much higher than the shutter frequency
- efficiency—the light source should be as efficient as possible (provide the most light watt output per electrical watt input) since the source will have to rely on vehicle power
Figure 5: Block diagram of the PAVUE I system.
The level of lighting required will be determined by the efficiency of the selected source, the shutter speeds, the lens f-number, the basic sensitivity of the cameras used, and the reflectivity of the pavements encountered. Light shielding also needs to be addressed.

With a single camera, a wide-angle lens must be used to get the greater FOV, so there is more “vignetting” which is a drop-off in the intensity of the image from the center to the edges of the image (i.e., the image edges are darker than the center by a factor of $\cos^4 \theta$). The use of multiple cameras reduces this vignetting effect.

The lenses chosen have a motorized iris (aperture), which can be used for relatively large, slow changes in overall lighting and/or pavement reflectivity. Hence, if the van moves from a very light pavement to a very dark pavement, it would be necessary to open the iris and allow more light in. The iris system would be set such that the aperture is open the widest for the darkest conditions to be expected, using a signal from the supervisory computer (over the pipeline) to control the irises.

### 4.3 Speed Independence

The majority of existing videologging systems are not speed independent and therefore not suitable for surface distress analysis. The problem is that the frame rate of the cameras and recording system is fixed and does not vary with the speed of the vehicle. That is, 30 images/sec are received whether the vehicle is going very slow or very fast. What this means is that if the van is going too fast, the image rate will be too slow and some of the pavement will be missed, while if it goes too slow, the images will overlap and the same crack will be analyzed (and counted) multiple times.

For example, the horizontal FOV of the ARIA system (MHM Associates) is stated as being 9 ft, and the vertical FOV is estimated to be 4 ft (Mohajeri & Manning, 1991). That is, each image captures a 4-ft long section of the pavement, and this is done 30 times a second. Hence, 1 second of taping covers 120 ft of pavement. However, the system is not speed independent, so there is overlapping (and hence extraneous) data unless the van travels at a fixed speed. That is, the images are contiguous (one right after another, with no gaps and no overlapping) only if the van travels forward a distance of 120 ft in 1 second. This corresponds to a vehicle speed of 82 mph (130 km/hr). If the van travels faster than this (unlikely), then there will be gaps between the images. Not all the pavement will be captured and stored, and the pavement surface can only be said to be sampled. In general, though, the measured statistics would still be fairly accurate, since the sampling rate would be fairly high. More importantly, though, if the van goes slower than 82 mph, then the images will overlap. The same crack can then appear in several images and will therefore be counted multiple times. Any statistics collected would be grossly inaccurate.

Since most of the time the van will be traveling significantly less than 82 mph, it is extremely important that the image collection be made synchronous with the van’s speed. This is the reason for the use of 35-mm photographic film. The camera speed can be changed readily to synchronize with the vehicle speed. This is much more difficult to do with a video recorder. OPQ Systems AB has developed a very low-cost proprietary method of collecting speed-independent images using standard cameras and video recorders.

### 4.4 The Video Recorder

In many of the existing videologging systems, the video signal from the camera is mixed with a second video signal containing a forward view and text, and the combined video signal is stored on standard VHS tapes. The images collected on the VHS tapes are returned to the office for off-line processing.

Although the number of pixels in the camera may be high (say, $640 \times 480$), the effective camera resolution is reduced by intermediary storage on videotape. The most important limitation is in the storage media used, the VHS tape. The VHS recording system can only store images to a maximum of 240 lines horizontal resolution (at SP speeds). Vertical resolution is fixed by the NTSC video standard at approximately 480 lines, but horizontal resolution is significantly reduced. This is why a video tape recording seems so
much "fuzzier" than the original video source. The standard consumer-quality VCR is also prone to a
great many problems when interfaced to a MV system, such as sync jitter.

Use of an S-VHS recorder improves the horizontal resolution from 240 to 400 lines but also requires
an S-VHS camera as a source. There are very few stand-alone S-VHS cameras available—nearly all S-
VHS cameras are built into camcorders and hence are not suitable for this application. The alternative
would be to use the standard monochrome RS-170/CCIR output from the camera and let the S-VHS re-
coder separate the signals internally, but some resolution may be lost this way. The PAVUE system
design uses a special stand-alone S-VHS camera.

It is sometimes mentioned that the VCR recorder used is a "4-head" VCR, but this is totally irrelevant
for the application at hand. The extra heads are used only to improve the signal for special-effects during
playback such as freeze-frame. What would be more important to know is whether a flying erase head is
used; this would impact greatly on the amount of noise recorded on the tape (the SNR).

4.5 Algorithms
The biggest problem area is in the segmentation (binarization) of the gray-scale image (Caroff et al., 1989;
Li et al., 1991). Fixed thresholding is extremely idiosyncratic and unstable—a small change in the
threshold setting causes large changes in the resulting image, and it is not at all obvious how to automat-
ically determine the proper threshold value. Furthermore, the optimal threshold value is not fixed but
varies from image to image (and even from one side of an image to the other side), because the image
contrast is itself not constant. It changes with lighting levels and with pavement reflectivity (which is
known to vary from 10-40% in reflectance).

A large number of image processing algorithms have been designed for analyzing video images. This
is the heart of the problem: there are literally thousands of algorithms available, but many are not suitable
for one reason or another (such as being too slow). We have had to survey (and continue to survey) what
is available, and in many instances, have had to develop our own algorithms when none was available in
the literature. Algorithm selection for image processing system design is still very much an art. The system
tends to be highly nonlinear, so simply reversing the order in which two processes are applied can dra-
tastically change the outcome (the analyzed output). Furthermore, each algorithm tends to have a "magic
number" or two, parameter values that must be set (such as gain, window size, etc.), and varying these
can also change the outcome. Furthermore, varying the parameters of one processing step can change the
effectiveness of the settings of a different processing stage (again, because of the nonlinearities involved).
We are looking at on the order of twenty such processing steps, all interacting with one another in highly
nonlinear and very unpredictable ways. Algorithm selection is therefore a very formidable obstacle.

Concurrent with algorithm selection is the development of the hardware to compute the algorithms in
real-time at highway speeds. All the algorithms have been simulated in software but were selected so that
they could be easily converted into hardware. Because of the processing speed requirements, all but the
final few processing steps will need to be done in hardware and not in software. Design of these hardware
circuits has been going on in parallel with the selection and testing of the software simulation of the al-
gorithms. As the software simulation is stabilized (the specific algorithms for each stage, the ordering of
the stages, and the parameter values for each stage), the hardware design is finalized and the boards con-
structed.

Running using only software simulation of the hardware algorithms, the image processing computer
can analyze the images stored on S-VHS video tape in the lab. It is also possible to connect the computer
directly to the cameras in the Laser RST van and analyze the pavement images directly. However, because
the system would be using primarily software (even though boosted by those special-purpose hardware
cards already finished), the system would operate only at relatively slow traffic speeds.

The prototype system was tested on several defective and nondefective pavement samples from images
collected in the field. This tested the effectiveness of the vision processing system. The accuracy of recog-
nition was estimated with false negative, false positive, and misclassification ratios, using signal detection
measures. The full PAVUE system will require a total of 20-25 boards, using a total of 8-10 different board types.

The initial system design called for the interfacing of the cameras to a simple PAVUE system which would not do any processing but would acquire the images and store them on S-VHS video tape for later off-line automatic processing in the office. The initial cards for this have been designed and tested. The pipeline is controlled by a VME interface (VMEIF) card, which connects the PAVUE system to a VME system (the PAVUE boards use only the VME connectors for power; all signals must pass through the VMEIF card). This card has been finished and works properly and in fact was used to construct the first (working) interface to the cameras. A frame-buffer card, FBUFF, is also completed and is used to store a complete image. This can be then transferred to the VME system for display (through the VMEIF), or it can feed into another OPQ-designed card, the VIDEOUT card, supporting a display monitor and having an interface for providing an SVHS-compatible signal for video recording of pipeline results. A packet-switching board, the PACKSW, is necessary to control the flow of signals within multiple loops of the pipeline. The remaining boards in the system include:

- LUT board: does simple arithmetic and logical operations by look-up table; the LUT board is a very general-purpose board, used like an ALU, for simple operations on one or two images
- GFILTR board: for doing gray-scale filtering of the image, before thresholding to a binary image; capable of doing very large kernel convolutions
- THRESH board: for adaptive thresholding (converting the gray-level image into a binary image; also does the shading correction)
- BINPROC board: a single board containing 18-stages of binary processing (erosion, dilation, thinning, large median filters, etc.)
- TMS31 board: using the TMS32031 chip, it is a general-purpose floating-point signal processing board for implementing mid- and upper-level image processing functions (such as polygon encoding and reduction, and feature extraction)
- BLOBDEC board: for implementing blob decimation, since the algorithm for blob encoding can be easily overloaded by too many small "noise" blobs in the image; removes small blobs from the image before the polygon encoding and feature extraction.

The above boards will be used to implement several different types of image processing steps, so each board will serve several functions. The PAVUE processing pipeline consists of the following stages:

- contrast enhancement (gray-scale stretching and adjusting) like a software gain control (LUT boards)
- median filter (BINPROC board)
- gray-scale filter (GFILTR board)
- adaptive thresholding (THRESH board)
- binary cleaning (BINPROC board)
- blob decimation (BLOBDEC board)
- run-length encoding (TMS31 board)
- polygon encoding (TMS31 board)
- polygon reduction (TMS31 board)
- feature extraction (TMS31 board)

In addition, several of the above functions require the use of FBUFF and PACKSW boards. The size of the PAVUE rack will be large, approximately 15-25 boards in two VME racks, but these will be reduced in the future as multiple functions are consolidated onto a single board. It is estimated that the final PAVUE system could be built using 10-15 boards (a single VME rack).

The VMEIF card is installed in the main Laser RST system VME rack (which consists of a large number of signal processing cards, the SPC2 boards). It controls the PAVUE rack via the PAVUE pipeline connectors (front panel ribbon cables). The final image data from the pipeline (ending at the feature extraction card) is sent back to the main system rack via a pipeline cable fitted to a piggy-back pipeline interface.
card sitting on an SPC2 card in the main rack. The pattern recognition algorithms are implemented on this SPC2 card in the main system. The pattern recognizer varies from customer to customer, and it is simpler to re-program the SPC2 card.

The main system SPC2 system rack will also function as a command “relay” for starting and stopping the image processing system. The UNIX-based vehicle supervisory computer controls the laser system via the SPC2 rack. Commands from the supervisory computer are relayed through the VMEIF card in the SPC2 rack to the PAVUE system. The results of the pattern analysis are from the PR SPC2 card to the supervisory computer for datalogging and display. The SPC2 rack will also provide the PAVUE rack with a wheel-pulse signal to properly synchronize the cameras to the motion of the van.

4.6 Pattern Classifier

The final stage of the image processing system is the pattern recognizer. This is a piece of software (not hardware) that takes the features of pavement defects found in the images and decides what type of defect they are. This obviously depends on what type of defect classification system is used. A great many such systems are in use throughout the world. We have done a thorough analysis of the type of manual pavement ratings distress systems in use, and from this, we have developed an initial pattern analyzer. These and the results of the Arnberg et al. (1990) survey have been used to finalize the design of a “generic” pattern recognizer. In addition, Arnberg et al. (in press-b) have reviewed the distress manuals for Sweden. Six crack types were used and specified in a handbook for road maintenance engineers. From this, objective criteria for discriminating among six cracking categories were extracted and developed into a discriminant tree. The main objective measures for discriminating among the crack types were (1) position, (2) direction/orientation, and (3) surface area involved. A pattern recognizer has also been developed from this work and is the one used in the demonstration video and in the figures shown later in this paper.

There are many methods available for identifying and characterizing surface distresses. Almost all of them use features based on the following characteristics of cracks (Haas et al., 1990):

- type—the crack pattern or overall shape, usually given a name
- severity—the average size of the cracks or crack pattern dimensions
- density or extent—the average frequency of occurrence of such cracking patterns per length of pavement

One of the better known methods of rating pavement distresses is that developed by the Ontario Ministry of Transportation and Communications (MTC). Initially, this system used 27 distinct distress types with 5 levels of severity and 5 levels of density. This has now been consolidated into 15 distress types (Hajek et al., 1986). Using factor analysis, Hajek and Haas (1987) showed that these 15 distresses form 5 basic independent groups, which explain about 60% of the larger category set.

Other well-known methods are described in the American Public Works Association’s PAVER manuals (APWA, 1984), and in the U.S. National Cooperative Highway Research Program Report (Darter et al., 1985). Recently, a pavement condition rating manual was developed for use by Roads and Transportation Association of Canada (Anderson, 1986). Excellent surface distress survey manuals for the countries of Australia and France are also available.

The PCES survey mentioned earlier also examined the frequency with which certain surface distress categories were measured. For flexible (asphalt) pavements, the following percentage of DOT’s surveyed were found to measure the distress categories shown in Table 1.

In addition to these categories for flexible pavement, 65% of the respondents also surveyed rigid (concrete) pavements, which tends to have somewhat different surface distress characteristics. For example, rigid pavement does not suffer from alligator or block cracking; its primary distresses are faulting (66%) and joint spalling (60%). There is some overlap, as in transverse cracking (42%) and longitudinal cracking (39%). The current PAVUE system is concerned primarily with flexible pavements, but it can be easily modified for those distress categories specific to rigid pavements.
Table 1: Percentage of surface distress types measured.

<table>
<thead>
<tr>
<th>Distress Type</th>
<th>% of DOT's Collecting</th>
</tr>
</thead>
<tbody>
<tr>
<td>alligator cracking</td>
<td>86%</td>
</tr>
<tr>
<td>patching</td>
<td>84%</td>
</tr>
<tr>
<td>longitudinal cracks</td>
<td>75%</td>
</tr>
<tr>
<td>transverse cracks</td>
<td>75%</td>
</tr>
<tr>
<td>ravelling</td>
<td>69%</td>
</tr>
<tr>
<td>block cracking</td>
<td>66%</td>
</tr>
<tr>
<td>bleeding</td>
<td>51%</td>
</tr>
<tr>
<td>edge cracking</td>
<td>30%</td>
</tr>
<tr>
<td>potholes</td>
<td>30%</td>
</tr>
<tr>
<td>corrugation</td>
<td>24%</td>
</tr>
<tr>
<td>polishing</td>
<td>9%</td>
</tr>
</tbody>
</table>

After examining the manual-survey procedure guides for the several countries mentioned earlier, the PCES survey, and the more recent survey conducted by Arnberg et al. (1990), it was concluded that these crack types are of primary importance in the majority of the surface distress evaluation systems currently in use:

- longitudinal cracks, cracks running roughly parallel to the road edges; those near the center of the road are called mid-line cracks while those near the edges are called edge cracks
- transverse cracks, cracks running roughly perpendicular to the direction of travel
- diagonal cracks, single cracks (no major branchings) at other angles
- alligator cracking, a network of cracks forming multi-sided (polygonal) blocks; block size ranges from 5-50 cm; generally may occur anywhere, although some distress surveys distinguish between alligator cracking in the wheel paths (alligator A) and elsewhere (alligator B)
- potholes, small roughly circular areas of pavement distress; if associated with alligator cracking (i.e., loss of blocks), they are not a potholes but increase the severity rating of the alligator crack
- map or block cracking; similar to alligator cracking but with large polygon/block size

The measurable characteristics that distinguish the categories are complex, but they will determine the design of the pattern classifier.

The measure of severity, or size of the cracks, varies with the crack type. In most cases, crack length, width, and perhaps total area are combined to give a measure of severity. The severity measure can be given in averages or totals of the above raw measures, or it can be associated with one of three severity categories (slight, moderate, or severe). For those cracks forming more complex patterns, such as alligator cracking, the characteristics of the individual crack components are less important than measures associated with the pattern itself. Hence, severity of alligator cracking is primarily a function of the number of crossing cracks, which in turn defines the alligator block size.

From the procedures manuals and the surveys noted above, the following would seem to be the most useful severity categories associated with each of the above cracking categories:

- longitudinal cracks: length, width, total area combined into 3 categories of severity (slight, moderate, severe)
- transverse cracks: length, width, total area combined into 3 categories of severity
- alligator cracking: block size, total area, loss of blocks (potholing)
- potholes: area, diameter
- map or block cracking: block size (area), extent
The features extracted and included in the feature list should allow the proper measurement of the above severity data.

The density or extent of cracking in most of the manual rating systems is given as a percentage of the pavement surface affected. We will define two general categories of extent and then measure frequency directly.

- Local, less than 30% of pavement surface affected; distress localized to specific areas only.
- General, more than 30% of pavement surface affected; distress pattern is evenly distributed over the entire length of pavement section.

The surface distress data can be used as raw data for input into a PMS system, or it can be combined into a single surface distress index (SDI) which is then input into the PMS (Anderson, 1986). To calculate the SDI, an index is first generated for each crack type (0-10), which combines the severity and extent (weighted 40/60). Each crack index is then weighted (from 0-10) depending on its importance (for example, alligator cracking is usually perceived as more important than potholes). The final index is normalized to a range of 0-10.

The third major use of distress data is to combine the SDI with other indices to produce a single index for the pavement quality as a whole, the Pavement Quality Index (PQI). The other indices used usually include a measure of subjective roughness (such as the IRI or the Ride Comfort Index, RCI) and one relating to the substructure of the road, such as the Structural Adequacy Index (SAI) derived from deflection testing. An example of such a PQI index is the one used by Alberta province (Cheetham & Karan, 1982):

\[ PQI = 1.161 + 0.060 \text{RCI} \times SDI + 0.526 \text{RCI} \times \log_{10} SAI \]

This composite index may then be used by itself in a graphical presentation system or be combined with other data (such as traffic loading) as inputs to a PMS.

The classification step is complicated because defect discrimination is not feasible by using only conventional shape-based features, such as area, perimeter, moments, etc., of segmented objects. Conventional classifier design based on statistical training with a large number of defect samples is not feasible because of the variable nature of the defects requiring a huge number of defect samples as "teaching" images. In addition, there were certain types of defect specifications that conventional statistical classifiers cannot handle well, such as local shape- or position-based (context-dependent) defects. Lastly, the system must cope with the problem of context-dependent defects, in which it is necessary to determine the location of the defect in order to establish the relevance of the flaw; that is, the region of the pavement determines the importance of the presence of defects and their role in acceptance/rejection. The presence of context-sensitive defect specifications and other complications means that a multilevel decision-tree classifier must instead be used.

The output of the PAVUE pipeline will continually make the following information available to the SPC2 pattern recognizer program. As each blob is deemed complete (i.e., the end of a long vertical crack has been reached), the feature analysis board makes the blob-tree available, the five elliptical parameters of each component blob making up the blob-tree, plus some extra shape features. The length, location, area, width, and general shape of each blob will be available. From this information we can characterize the road with respect to:

- Crack location (outside, between, or in the wheelpaths)
- Extent of cracking; percentage of area distressed (no cracks, moderate, severe)
- Severity of cracking (area of cracks, width vs. length)

Percentage of road area with distresses will be output, localized into five pavement zones (the same as those defined in the questionnaire study). Initially, the only crack types discriminated will be longitudinal and lateral cracks (giving simple counts and a breakdown by pavement zones) and alligator cracking (percentage of pavement area involved in each pavement zone).

The initial pavement image goes through several stages of enhancement, followed by adaptive thresholding to a binary image. A connectivity module then processes the binary image, labelling each "blob" with a unique identifying number. The branches of a single blob are individually labelled and a
"branch flag" set, indicating that it is a part of a larger blob. A feature extractor then analyzes each blob as to its general shape, area, perimeter, orientation, location (x,y coordinates of its center of mass), and whether (and where) the blob intersects the image boundaries. This set of features and the branching information is then output from the pipeline processor, for further processing by the supervisory computer.

As each blob is deemed complete, the feature-analysis board makes the elliptical parameters of the concavity tree of each blob available. That is, any defect (a black area in the image, called a blob) is analyzed according to its subcomponents. A shapeless blob, which may have various concavities, is broken down into its minimal convex blob components. Each convex blob is fit with an ellipse, and the five elliptical parameters are then the shape descriptors for that subcomponent. The full blob is a tree-like structure of these shape descriptors and how they interconnect with each other. With these parameters, the area of the blob, the location (x,y) of its center of mass (centroid), its dominant orientation (angle), the total perimeter, its width, and the length of the centerline down the blob can be computed easily. Furthermore, it is easy to combine this information together to form the shape of the total blob from its “sub-blobs.” Analysis of the tree structure can discriminate between the various classes of cracks (such as Y-cracks, D-cracks, alligator cracks, etc.).

The classes of interest are defined by the customer. For example, a Y-crack would consist of three sub-blobs (each would have to be longer than some minimum) with the three connecting at the same level in the tree (at the same junction point); their orientations with respect to one another are approximately 130° (again, some thresholds on the range of acceptable angles must be set). An alligator crack would show a tree with many levels and cyclic loops (the enclosed blocks of the alligator crack form blobs that touch end to end in a closed loop); the areas and shapes of the blocks could be computed from the lengths and orientations of the sub-blobs forming the boundaries of the block. Alligator cracking would be discriminated from block cracking from the average size of these blocks.

The statistical approach to pattern classification involves the creation of a set of measures, called a feature vector, and the selection of a discriminant function of those vectors that divide the feature space into the desired categories. Each measurement type is called a feature, and the n measures taken on an object to be classified yield a point in n-dimensional feature space. Various algorithms have been explored for dividing the feature space into regions, each representing a class. The unknown entity is assigned to a class corresponding to the region of the feature space at which the point defined by its feature vector is found.

The other major pattern classification scheme, the structural approach, is based on detecting the presence or absence of some feature(s). Because of this, it is often referred to as binary classification. Extending the above example, you may know that all cats have retractable claws but dogs do not. Thus, perfect discrimination is possible using only one binary feature. Of course, it may be more difficult to measure this feature than weight or age. However, if it can be measured, it would provide the best classifier. This also illustrates another very important point, that no amount of sophistication in the decision process can make up for poor selection of features. In optical character recognition, by counting the number of end-points of a character (a single-vector termination), the upper case letters of the alphabet can be classified into five groups. Within each group, the letters can be further grouped according to the number of T-junctions (H has two). If the measures on the image indicate that the letter has two end points, then the best that you can conclude is that it must be one of “ACGLMNQRSUVWZ,” but if you also know that it has two T-junctions, then it must be the letter “A.”

This type of classification using multiple measurements is more easily visualized as a decision tree. Each node in the tree corresponds to a feature measurement, and there is a branch for each value of the measurement. Decision trees are used in many PR applications, although they are usually more complex that this simple OCR example. With the structural classifier, classification can be seen to be a multi-step process. Each step uses a classification rule to divide the objects into increasingly smaller groups until full classification is achieved. This approach also lends itself to an economical method of sequential classification by making only the measurements that are essential to classifying the object in question.

Each object to be classified trickles down through a series of node classifiers, which successively narrows the list of possible classes for the object until it is finally classified on the basis of logical checks.
on the object features (shape descriptors). Optimally, each node is a binary statistical classification. For example, node 1 typically checks the object’s area to see if it is large enough to be a true distress or if it is only a noise object. Node 2 could then check if the object is a line distress (crack) or an area imperfection (pit, pothole, patch, etc).

Fukuhara et al. (1989) use several such structural rules in their recognizer for pavement distresses. They combine crack segments on the basis of proximity and co-linearity, and remove small isolated segments. However, this addresses primarily the problem of connectivity determination. The specifics of how to categorize the cracks (Y-cracks, alligator cracks, etc.) is not discussed.

Using a structured “feature tree” can speed up the pattern recognition process by arranging the match comparisons so that the highest probability mismatch occurs first. Likewise, efficiency can be improved by not computing those features until needed by the feature tree. In this case, the classification process, guided by the feature tree, is used to control the feature extraction process. If extracting the first feature clearly classifies an object, other features do not have to be computed. If, however, the first feature does not completely distinguish an object, a second is computed and compared, and a third if needed, and so forth.

This classifier may be regarded as too simple, since it does not use a combination of features but rather tests each one separately. However, this overall structural framework is justified for uncorrelated features. In general, this condition is satisfied for most categorical features, and the methodology emphasizes their use over continuous features. When applicable, this approach offers some advantages. Besides efficiency, one of its important characteristics is extensibility, i.e., it is easy to modify, add, or delete new defect classes without disturbing the others. It is also easier to debug.

4.7 Information Fusion
An important aspect of the PAVUE system's increased detection accuracy is the integration of information from both the laser and video systems. The complexity of this process lies in the need to incorporate information about both cracks and their depths. Machine vision systems are not very effective at measuring depth. The MV system needs to work closely with the current laser RFs.

The vision system operates on scene luminance rather than on range (distance); the two systems are complementary in this sense. The laser RF is an active sensor system that measures distance directly. The MV system uses a passive light detector array with an external source of energy (lighting) and must assume that differences in contrast are due to differences in depth. It does not measure distance directly but instead measures changes in brightness across the spatial plane imaged. Since cracks (filled with debris or perhaps patched) generally have differing contrast from the background pavement, they will be detected and measured. Information from the laser RFs is needed to discriminate if the cracks are filled (no cracking detected by laser RFs) or not (cracks also detected by laser RFs). The laser RFs are also necessary to help the MV system reject dark markings on the pavement, such as tread marks and bleeding (these would have a different contrast like cracks but would have higher contrast than cracks and no depth changes).

The information from the lasers and the PAVUE system can be combined at differing levels of detail, the more detailed the level of fusion the more complicated the processing involved. The lowest level of information fusion is with respect to the overall 1-meter measurement segment (a statistical combination process). At the highest level of detail, the presence of cracking at a particular pixel is compared to an estimated pavement depth (from the lasers) at that pixel.

4.8 Selective Videologging
The current PAVUE system, which gives a characterization of the pavement surface in terms of frequency and severity of surface defects accumulated over a section of road (say, every 20 m) is obviously very efficient in terms of storage space. Even a relatively detailed “crack map” is output. What is lost, however, is the raw unprocessed video data, which can be important to certain users, particularly research groups who wish to track changes in the pavement over time. To try to satisfy this need, the full PAVUE system
includes a videologging option, which means that video images of pre-selected defect patterns and severity levels are stored off-line. This intelligent selective videologging should be much more efficient than continuous videologging, as long as the selection criteria can be specified accurately enough.

Current methods of video logging require that the entire measurement surface be recorded for later analysis in the lab. This simply postpones the inevitable, tedious, error-prone and costly manual analysis until some time later. There is still quite a bit of data to wade through before the interesting road sections are located for analysis. Such simple continuous video logging of the pavement surface is available if desired. Such systems are useful, but they suffer from low resolution. A higher resolution system would present problems in terms of storage size and bandwidth. It is difficult to do continuous logging of high-resolution data.

A much more efficient technique would be to pre-screen the images being video-logged in order to ignore the large number of sections with little interesting detail; only those sections of potential interest for further detailed analysis by road engineers would be stored. Data storage requirements would be cut drastically, and the tedious tasks of manual image screening would be vastly simplified, yielding better results.

The real-time crack analysis system can be designed to automatically record sections of road that meet some user-defined criteria. In this way, only sections of interest for later analysis are saved.

### 4.9 Current Status

This section will report on the progress to date in the construction of the PAVUE I system. Feasibility tests have been completed using multiple passes of a single-channel video system to yield the five videos making up the five zones of a lane of pavement.

Algorithm design is complete but is currently simulated in software, awaiting the final testing of the hardware processing boards. Five hardware cards have been developed and successfully tested; three more need to be completed for full real-time processing. Analysis of the above videotapes using the software-based algorithms have been found to be accurate and reliable under varying pavement conditions, but they are very slow.

Figure 6 shows the result of processing on a longitudinal crack. Depending on which zone it is in and on how continuous the crack is, it may be classified as a mid-line crack or an edge crack.

The above video data was collected at around 30 km/h. Manual crack maps were prepared for all measured sections, and these were compared with the cracks found by the computer analysis program. Figure 7 shows the manual crack map for the road section corresponding to the above longitudinal crack figure.

An edge crack is defined as any somewhat discontinuous longitudinal crack found in the outer road zone. An example of processing on an edge crack is shown in Figure 8.

Figure 9 shows the processing applied to a transverse crack; this data was collected at approximately 90 km/h. A transverse crack can extend across more than one zone.

Figure 10 is an example of processing on a section of road with alligator cracking. If the alligator cracking is primarily in the wheelpaths (zones 2 and 4), these are considered “bearing” or “load” cracks (Alligator A). If they extend into other zones, this cracking pattern is classified as Alligator B.

An example of the final data statistics for a 10-meter section of road pavement (ten 1-meter segments) is shown in Table 2. In this table, the entries show the severity of distress (area) for each 1-meter segment in each of the five zones. The numbers in parentheses are the classification numbers of the crack type of any distress exceeding a preset severity level (0.5). The sum of the distress areas within and across zones is also shown. The last column gives the average severity across zones and the dominant crack type for each segment. Likewise, the next to the last row gives the totals for the entire 10-meter section of pavement, analyzed by zone. The last row is the extent, the percent sections containing a detected cracking type. The first zone had only one 1-meter segment with cracking (type 3 cracking, a transverse crack); hence, the extent of cracking for zone 1 of this pavement section is 10% (1 out of 10). The final summary
statistics across all zones and segments are shown in the lower right corner of the table. The severity for this section is the total area distressed, the crack type given this section is the crack category with the largest area involvement, and the extent for the section is the percent of segments having that crack classification. Since 9 out of 50 segments show crack category 5 (longitudinal crack), the extent for this section is 18%.

Figure 11 shows a more advanced stage of processing than has been used in the above examples. It shows how the individual "blobs" in an image can be fit with a reduced order polygon (saving storage space and improving subsequent processing times. Each such blob is then fully measured and characterized with respect to its dimensions, orientation, perimeter, etc.

In addition to software implementation and testing of the analysis algorithms, a simple but very functional version of the random-access videotape image database system has been successfully implemented. The system operator simply types in the desired section and zone number and the videotape is automatically positioned by the PAVUE system computer to the beginning (segment 1) of that zone.
Handritning av sprickor

Plats: SKL Datum: 9/08/05
Typ av sprickor:
Fogssprickor, belastningssprickor

Figure 7: Manual crack map.
Figure 8: Example of processing on an edge crack.
Figure 9: Transverse cracking example.
Figure 10: Alligator cracking.
Table 2: Example of output statistics.

<table>
<thead>
<tr>
<th>Section 302</th>
<th>Zone 1</th>
<th>Zone 2</th>
<th>Zone 3</th>
<th>Zone 4</th>
<th>Zone 5</th>
<th>All Zones</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.28</td>
<td>0.03</td>
<td></td>
<td>0.15</td>
<td>6.7 (5)</td>
<td>1.4 (5)</td>
</tr>
<tr>
<td>2</td>
<td>0.06</td>
<td>0.09</td>
<td></td>
<td>0.11</td>
<td>4.1 (5)</td>
<td>0.88 (5)</td>
</tr>
<tr>
<td>3</td>
<td>0.14</td>
<td></td>
<td></td>
<td>0.17</td>
<td>3.4 (5)</td>
<td>0.75 (5)</td>
</tr>
<tr>
<td>4</td>
<td>0.03</td>
<td></td>
<td></td>
<td>0.07</td>
<td>3.3 (5)</td>
<td>0.68 (5)</td>
</tr>
<tr>
<td>5</td>
<td>0.12</td>
<td></td>
<td></td>
<td>0.06</td>
<td>4.1 (5)</td>
<td>0.86 (5)</td>
</tr>
<tr>
<td>6</td>
<td>0.15</td>
<td></td>
<td></td>
<td>0.08</td>
<td>3.5 (5)</td>
<td>0.75 (5)</td>
</tr>
<tr>
<td>7</td>
<td>0.06</td>
<td>0.04</td>
<td></td>
<td>0.05</td>
<td>3.6 (5)</td>
<td>0.75 (5)</td>
</tr>
<tr>
<td>8</td>
<td>0.01</td>
<td>0.03</td>
<td></td>
<td>0.09</td>
<td>3.6 (5)</td>
<td>0.75 (5)</td>
</tr>
<tr>
<td>9</td>
<td>0.03</td>
<td>0.08</td>
<td></td>
<td>0.09</td>
<td>3.5 (5)</td>
<td>0.74 (5)</td>
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<td>10</td>
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<td>6.3 (3)</td>
<td></td>
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<td>8.9 (2)</td>
<td>7.0 (3)</td>
</tr>
<tr>
<td>1-10</td>
<td>2.0 (3)</td>
<td>0.7 (3)</td>
<td></td>
<td>0.11</td>
<td>4.5 (5)</td>
<td>1.5 (5)</td>
</tr>
<tr>
<td>Extent</td>
<td>10%</td>
<td>10%</td>
<td>0</td>
<td>0</td>
<td>90%</td>
<td>18%</td>
</tr>
</tbody>
</table>

Figure 11: Polygon fit to image blobs and extraction of significant features.
5 PAVUE II

The PAVUE II is the ultimate expression of the basic PAVUE design. It is essentially the PAVUE I system but dispenses with the need for an intermediate videotape storage step (Figure 12).

The use of field-based analysis of the data would also allow instant feedback to the vehicle operators about the condition of the pavement and the integrity of the data being collected. The vehicle operators would monitor the generated statistics to see if they were consistent with what they were seeing.

Another advantage with the PAVUE system is that because the intermediate VCR storage is no longer used, the cameras no longer need to be broadcast TV standard. Higher-performance cameras like those used in industrial machine-vision systems may be used instead, such as a linescan type of camera. This type of camera reads only a single line of the imaged area, but if the imaged area moves (as in pavement inspection) and the lines are "snapped" in synchronization with this movement, the lines will make up a continuous ribbon of images. Furthermore, because of the way these cameras are constructed, they have a very efficient and very fast electronic shuttering which does not reduce the light (beyond that of the reduced integration time) or the resolution.

The problem with linescan cameras is that they are one-dimensional imagers that rely on external motion to provide the second dimension to the image. Without that motion, there is only a single line of video, which cannot be directly viewed except on an oscilloscope. The use of multiple cameras also complicates this: the registration of the images between cameras must be precise. That is, the point on the pavement where one scanline ends should coincide with the point where the scanline of the next camera begins. This requires precise alignment of the cameras, a fact that is inconsistent with vibration and field replacement.

The solution to this would be to use a 2-dimension (2D) normal array camera that would allow easy calibration by a relatively naive user yet could also somehow be switched to work in a linescan mode. It could operate like a regular camera (reading a 2D array of image lines) or it could readout a single user-specified line at a time. Field verification of the PAVUE system integrity by the van operator was therefore a simple matter of viewing the images coming off the cameras when in their 2D array mode. While the van was stationary, the operator could see at a glance any problems with lights, lenses, or the cameras themselves. Moreover, the cameras could then be made to scan only a single line (1D mode) across the pavement by selecting the appropriate scanline to be read out on each camera individually. If a camera had shifted, the alignment could be returned by simply using a different scanline (video row number) from that camera.

Unfortunately, only one such camera is commercially available, and testing showed it to be too noisy at the higher frame rates. Since then, a new prototype camera satisfying the requirements has been developed by a specialty company in Sweden. It is unique in that all video sensing and conversion is done within a single chip, which also contains sophisticated image processing circuitry. Testing using this "smart camera" will begin later this year.

The PAVUE II system will support a "smart videologging" function, selectively storing to a high-capacity WORM disk those frames of video containing information specified (to the pattern recognizer) as being of interest to the customer. This avoids the storage of billions of bits of uninteresting pavement stretches. Of course, it is necessary to be able to specify what is interesting and have a system smart enough to recognize it in the video and log it to disk. Storage of full crack maps would still be available, of course. These videologged images would become part of an extensive pictorial database system, a picture archive and storage system (PACS) similar to that now used for medical images.

The current status of the PAVUE II system is obviously very dependent on the progress made on the PAVUE I image processing hardware. In addition, it is necessary to develop the unique set of cameras, lighting, and optics. Several different lighting schemes and camera systems have so far been prototyped and tested, but a final selection has not been made yet. The system will use either (1) a single linescan camera and light-shroud, mounted near the top rear of the Laser RST van (with associated lights mounted at bumper level), or (2) multiple "smart" linescan cameras mounted at approximately bumper level. There are advantages and disadvantages to both of these approaches, and further testing is needed to find the
Figure 12: Block diagram of the PAVUE II system.
optimal configuration. The hardware for interfacing the cameras to the PAVUE pipeline is finished and tested and awaiting camera selection.

6 Summary

Today, the Laser RST uses lasers to measure cracks. This method has led to several difficulties:

- measurement is influenced by the megatexture and rough texture of the surface. The laser-based measurement system attempts to correct for this. The result of this correction is that very small cracks on roads with a rough surface are not presented.
- cracks filled with debris or water cannot be correctly measured.
- patterns of cracks can only be crudely determined.
- cracks cannot easily be distinguished from holes.

The addition of video-based image processing should be able to eliminate these problems. The processing pipeline first removes any impulse noise present, followed by filtering to reduce the distracting effects of varying road texture and small features in the background, leaving only the larger distress patterns. The filtered and enhanced image is then adaptively thresholded to a binary image. This step is very difficult to do efficiently in a pipeline architecture. A proprietary adaptive thresholding procedure has been specifically developed for use with PAVUE scanline-based pipeline. The output of any thresholding procedure is typically noisy, in the sense of small binary points, pinholes within larger blobs, and gaps in boundaries. These are cleaned up with several stages of binary filtering.

At this point, the objects in the image should represent regions of surface distress. Which binary objects are separate and which blobs are branches of other objects is established by a special one-pass connectivity algorithm. This raw blob information is then processed by the feature extraction module. For this module, it was necessary to develop a new algorithm for extracting the first three geometric moments of a binary blob in a single sequential pass. From these moments for each blob and its perimeter, a set of shape descriptors or features is generated. The feature extractor is the last module in the pipeline. The feature vector for each blob, along with its branching information, is output to the pattern recognizer, a software module running on the supervisory microprocessor.

The pattern recognizer accepts inputs from the PAVUE pipeline and yields an output report as to the type and extent of cracking present. A decision-tree pattern recognizer was developed for yielding a distress type and severity measure in a manner useful for road maintenance engineers.

The performance of a prototype system based on these techniques was tested on several different types of continuous input images. The algorithms performed as expected, yielding binary thresholded images clean and complete enough for the connectivity and feature extraction modules to properly process. As expected, performance is less than perfect with more ambiguous images (such as those involving bleeding, shadows, and tire marks), with errors made in both false alarms, misses, and misclassifications of crack type. However, the system still performs well on a large number of pavement videotapes of many different types of pavement, yielding detection rates of over 89% for all images. The misclassifications found are those that also tended to be ambiguous to a human rater. In summary, the algorithms work well with each other and yield an adequate level of performance when analyzing real-world images.

A final informal observation is that even with just a software simulation of the pipeline on a single-processor system, the processing runs faster than using the conventional image-processing modules commercially available.

The recognition results presented show that automated high-speed object recognition can be performed with a specially designed surface inspection system. By utilizing a parallel pipeline architecture to perform the bulk of the computation-intensive image processing and feature extraction, the entire system can be made to run at real-time rates. Simplifying the control flow of the algorithms and shifting the decision-making from an external controller to the data path allows pipelining to be used more extensively, increasing the number of operations/second that can be performed. The use of scanline-based algorithms
minimizes the total pipeline delay, simplifies the implementation and control, and saves storage and space costs.

The real-world realities of video pavement distress analysis are a constantly changing contrast level, very uneven and variable lighting, interference from ambient illumination, and extremely variable textured backgrounds, often of a size and gray-level comparable to the cracks one needs to discriminate. Of course, cracks 1/2-inch wide are simple to discriminate and could be detected by a simple system. But this capability is nearly useless—a 1/2-inch crack does not allow much in the way of preventative maintenance; at that point, total reconstruction may be necessary. Instead, a feasible, automated pavement-distress system must have the ability to analyze pavement cracking at a level of detail and with a degree of reliability and accuracy which would be of use in pavement maintenance. It must be able to analyze pavement distresses under a variety of conditions and on different pavement types accurately and reliably.
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