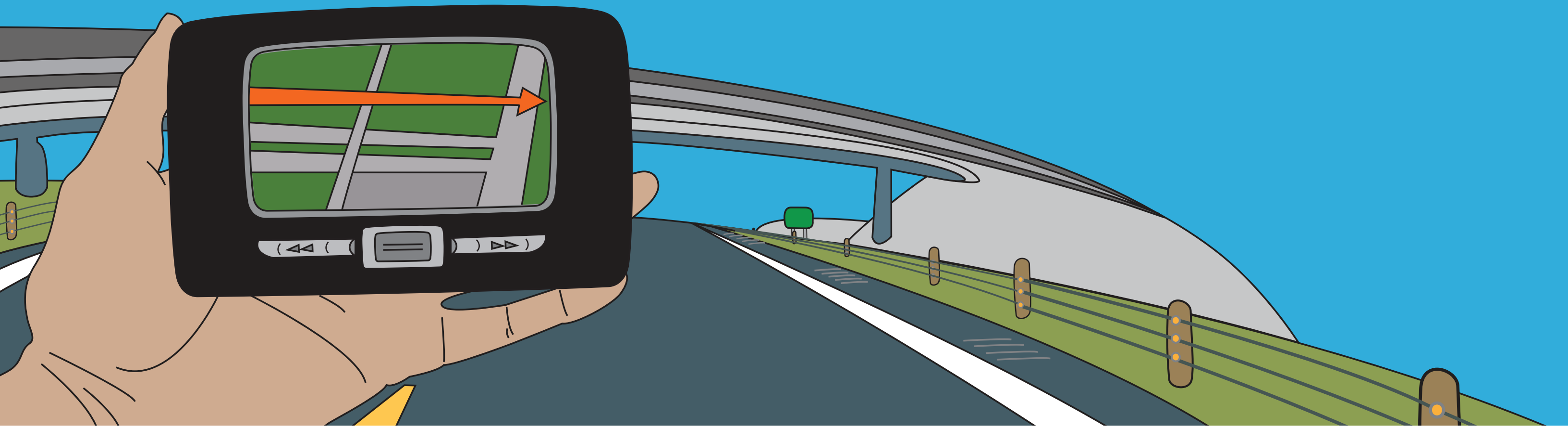
Partnerships in Transportation Research, Innovation, and Training

Mid-Continent Transportation Research Symposium 2009

August 20–21, 2009



Event Program

Mid-Continent Transportation

Schedule of Events for August 20–21, 2009

Thursday, August 20th

Continental Breakfast
Registration

Opening Session Speakers:

Sandra Larson, Iowa DOT
Kevin Mahoney, Iowa DOT
Shashi Nambisan, InTrans, ISU
Sharron Quisenberry, ISU
Lubin Quinones, FHWA Iowa Division
Marty Wachs, Rand Corp.
Ed Kannel, CCEE Dept., ISU

Break

Concurrent Session 1

- Traffic and safety
- Structures and construction
- PCC pavements
- Planning, modeling and logistics
- Geotechnical engineering
- Finance

Luncheon and Speakers:

Jon Wickert, Dean ISU College of Engineering
Ann Brach, Transportation Research Board

Concurrent Session 2

- Traffic and safety
- Structures and construction
- PCC pavements (Sustainable Concrete Pavement Technologies, Part 1)
- Planning, modeling and logistics
- Asphalt pavements
- Modes

Break

Concurrent Session 3

- Traffic and safety
- Structures and construction
- PCC pavements (Sustainable Concrete Pavement Technologies, Part 2)
- Planning, modeling and logistics
- Asphalt pavements
- Technology considerations

7:00 a.m.
7:30 a.m.
8:00
8:30
9:00
9:30
9:45
10:00
11:00
11:45
12:00 p.m.
1:00
1:15
1:30
2:00
3:00
3:30
4:00
5:00 p.m.

Thursday, August 20th

Social: Meet and greet + poster session

Banquet and Speaker:

Bill Fennelly, Head Coach, Iowa State University Women's Basketball on "Leadership and the Challenge of Maintaining a Successful Organization"

5:00 p.m.
5:30
6:00
6:30
7:00
8:00
8:30 p.m.

Friday, August 21th

Continental Breakfast
Registration

Concurrent Session 4

- Traffic and safety
- Hydrology
- Road maintenance
- Environment
- Asphalt pavements
- Technology considerations

Break

Concurrent Session 5

- Traffic and safety
- Structures and construction
- Highway engineering and construction
- Materials

7:00 a.m.
7:30 a.m.
8:00
8:30
9:00
9:30
10:00
10:30
11:00
11:30
12:00 p.m.

Research Symposium 2009

Special Speakers

Opening Session Speakers

Sandra Larson, Director, Research and Technology Bureau, Iowa DOT

Kevin Mahoney, Director, Highway Division, Iowa DOT

Shashi Nambisan, Director, Institute for Transportation, Iowa State University

Sharron Quisenberry, Vice President, Research and Economic Development, Iowa State University

Lubin Quinones, Iowa Division Administrator, FHWA

Ed Kannel, Professor, Department of Civil, Construction, and Environmental Engineering, Iowa State University

Marty Wachs, Director, Transportation, Space, and Technology, Rand Corporation

Jon Wickert, Dean, College of Engineering, Iowa State University

Luncheon Speaker

Ann Brach, Deputy Director, Strategic Highway Research Program 2, Transportation Research Board

Banquet Dinner Speaker

Bill Fennelly, Head Coach, Women's Basketball, Iowa State University

Symposium Sponsors

Iowa Department of Transportation

Iowa State University's
  Institute for Transportation;
  Department of Civil, Construction,
  and Environmental Engineering;
  and Midwest Transportation Consortium

The University of Iowa's
  Department of Civil and
  Environmental Engineering;
  Department of Industrial and
  Mechanical Engineering;
  and Public Policy Center

University of Northern Iowa

University of Wisconsin's
  National Center for Freight &
  Infrastructure Research & Education

Wisconsin Department of Transportation

Concurrent Sessions Thursday, August 20th 10:00 a.m. — 12:00 p.m.

Session 1A

Traffic and Safety

(Larry Rilett,
University of Nebraska,
Lincoln,
Moderator)

1. **Development of Predictive Median Barrier Warrants**
   Madhav Chitturi, Andrew Ooms, David A. Noyce, University of Wisconsin, Madison
2. **Effectiveness of Localized Deer Management in Reducing Deer-Vehicle Crash Rates in Iowa—Some New Evidence**
   Michael Baird, Konstantina Gkritza, Zachary Hans, Institute for Transportation, Iowa State University
3. **Evaluation of Centerline Rumble Strips for Prevention of Highway Crossover Accidents in Kansas**
   Daniel Karkle, Margaret Rys, Eugene R. Russell, Kansas State University
4. **Predicting and Mitigating Wind-Induced Truck Crashes on I-70 in Kansas**
   Robert Rescot, Romika Jasrotia, Yue Li, Steven Schrock, University of Kansas;
   Kelly Hovey, Kansas Department of Transportation
5. **Identification of Contributory Factors for Cross-Median Crashes**
   Madhav Chitturi, Andrew Ooms, David A. Noyce, University of Wisconsin, Madison

Session 1B

Structures and Construction

(Terry Wipf,
Iowa State University,
Moderator)

1. **An Owner's Perspective on Implementing an Accelerated Bridge Construction Program**
   James McMinimee, Utah Department of Transportation
2. **Development, Evaluation, and Implementation of a Precast Paving Notch System**
   Brent Phares, Institute for Transportation, Iowa State University;
   Dean Bierwagen, Iowa Department of Transportation;
   Mike LaViolette, HNTB Corporation
3. **Mass Concrete Thermal Control Case Study: I-80 over the Missouri River Bridge Construction**
   James Nelson, Ahmad Abu-Hawash, Iowa Department of Transportation;
   John Gadja, CTL Group
4. **Precast Bridge Approach Slab Replacement Project in Denver, Iowa (Design, Casting, Construction, and Monitoring)**
   Dean Bierwagen, Ahmad Abu-Hawash, Wayne Sunday, Iowa Department of Transportation
5. **The Kansas Experience with Polymer Concrete Bridge Deck Overlays**
   Dave Meggers, Kansas Department of Transportation

Session 1C

PCC Pavements

(Tom Cackler,
Iowa State University,
Moderator)

1. **Can Air be Entrained in Roller-Compacted Concrete Mixes?**
   Chetan V. Hazaree, Halil Ceylan, Kasthurirangan Gopalakrishnan, Iowa State University
2. **Curling of New Concrete Pavement and Long-Term Performance**
   Daba Gedafa, Mustaque Hossain, Kansas State University;
   Zahidul Q. Siddique, ARCADIS LNW;
   Kirk Fredrichs, FHWA Nebraska Branch;
   Dave Meggers, Kansas Department of Transportation
3. **Determining Pavement Damage Cost Attributed to Truck Traffic in Southwest Kansas**
   Chunxiao Liu, Yong Bai, University of Kansas;
   Yingfeng Li, Texas Transportation Institute, Texas A&M University
4. **Study of Iowa PCC Thermal Properties for Mechanistic-Empirical Pavement Design**
   Jiong Hu, Texas State University, San Marcos;
   Kejin Wang, Iowa State University;
   Zhi Ge, North Dakota State University
5. **Laboratory and Field Testing of Hycrete Corrosion-Inhibiting Admixture for Concrete**
   John Wojakowski, Hycrete, Inc.
   Jennifer Distlehorst, Kansas Department of Transportation

Concurrent Sessions **Thursday, August 20th** **10:00 a.m. — 12:00 p.m.**

Session 1D
Planning, Modeling and Logistics

(Daniel Yeh,
Wisconsin DOT,
Moderator)

1. Review of Congestion Pricing Established Systems in Europe and Asia: Evaluation of Success
Aikaterini Rentziou, Konstantina Gkritza, Institute for Transportation, Iowa State University

2. Estimating Economic Benefits Due to Increased Seat Belt Use: A Case Study
Sunanda Dissanayake, Kansas State University

3. The Actual Cost of Food Systems on Roadway Infrastructure
Inya Nlenanya, Omar Smadi, Institute for Transportation, Iowa State University

4. Continuous Productive Urban Landscapes: A Sustainable Design Option to Growing Urban Communities in Iowa
Jason Grimm, Iowa State University

Session 1E
Geotechnical Engineering

(Lubin Quinones,
FHWA Iowa Division,
Moderator)

1. Advances in Intelligent Construction Methods in Geotechnical Engineering
David J. White, Earthworks Engineering Research Center, Iowa State University

2. SHRP2 R02—Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform
Vernon R. Schaefer, Iowa State University

3. Construction QA/QC Testing versus Selection of Design Values for PCC Pavement Foundation Layers
Samuel Caleb Douglas, David J. White, Earthworks Engineering Research Center, Iowa State University;
Jeffery R. Roesler, University of Illinois at Urbana-Champaign

4. The Use of Propane as a Subbase Drying Technique
Michael Blahut, Vernon R. Schaefer, R. Christopher Williams, Iowa State University

5. Use of Bio-Oil for Pavement Subgrade Soil Stabilization
Halil Ceylan, Kasthurirangan (Rangan) Gopalakrishnan, Sunghwan Kim, Iowa State University

Session 1F
Finance

(Jeramy Ashlock,
Iowa State University,
Moderator)

1. Interactions between Transportation Capacity, Economic Systems, and Land Use
Stephen Andrle, Transportation Research Board

2. Financing Road Projects in India Using PPP Scheme
Satyanarayana Kalidindi, L. Boeing Singh, Indian Institute of Technology Madras

3. Alternative Gas Tax Options
Paul Hanley, University of Iowa

4. Comparative Study of Costs Incurred by Transportation Users and Charges Compensated
Abhisek Mudgal, Institute for Transportation, Iowa State University

5. Highway Capacity Improvements and Land Value Responses: Some Estimates of the Economic Impacts of Upgrading Roads
Michael Iacono, David M. Levinson, University of Minnesota

Concurrent Sessions Thursday, August 20th 1:30 p.m. — 3:00 p.m.

Session 2A
Traffic and Safety

(JP Miller,
Mossouri DOT,
Moderator)

1. Access Management Considerations for Roundabout Design and Implementation
Mark T. Johnson, MTJ Engineering, LLC;
Hillary N. Isebrands, Iowa State University

2. Iowa's Roundabout Experience
Tim Simodynes, Iowa Department of Transportation

3. Rural Safety Innovation Program: An Overview and Status Update
Theodore A. Smith, Noblis;
Linda Dodge, United States Department of Transportation

4. Update on Iowa's 2009 Roadway Departure Strategic Action Plan and Synthesis of Neighboring State Practices to Address Roadway Departure Crashes
Eric J. Fitzsimmons, Shashi Nambisan, Shauna Hallmark, Institute for Transportation, Iowa State University;
Tom Welch, Iowa Department of Transportation

Session 2B
Structures and Construction

(Norm McDonald,
Iowa DOT,
Moderator)

1. Laboratory and Field Evaluation of the 24th Street Bridge
Terry J. Wipf, Brent M. Phares, Travis Hosteng, Jake Bigelow, Institute for Transportation, Iowa State University

2. Application of Wireless Sensor Networks in Infrastructure Monitoring: A Preliminary Study
Jin Zhu, M.D. Salim, University of Northern Iowa

3. Monitoring of an Integral-Abutment Bridge Supported on Steel Piles/Concrete Drilled Shafts in Glacial Clay
Ashley Disburg, Robert E. Abendroth, Iowa State University;
Brent M. Phares, Institute for Transportation, Iowa State University;
Ahmad Abu-Hawash, Kenneth Dunker, Iowa Department of Transportation

4. Monitoring of the Saylorville and Red Rock Reservoir Bridges for Wind-Induced Vibrations
Travis Hosteng, Brent Phares, Institute for Transportation, Iowa State University;
Ahmad Abu-Hawash, Bruce Brakke, Iowa DOT;
Doug Wood, Iowa State University

Session 2C
Pavements (Sustainable Concrete Pavement Technologies, Part 1)

(Gordon Smith,
Iowa Concrete Paving
Association,
Moderator)

1. Common Sense Sustainability for Concrete Pavements
Peter Taylor, Institute for Transportation, Iowa State University

2. Performance Properties of Ternary Mixes
Paul Tikalsky, University of Utah;
Peter Taylor, Institute for Transportation, Iowa State University;
Kejin Wang, Iowa State University

3. Concrete Overlays Helping to Address the Nation's Preservation/Rehabilitation Needs
Dale Harrington, Snyder and Associates

Session 2D
Planning, Modeling and Logistics

(Adam Shell,
Iowa DOT,
Moderator)

1. **A Large-Scale Traffic Simulation Model for Hurricane Evacuation of Hampton Roads, Virginia**
   Praveen Edara, University of Missouri, Columbia
2. **Iowa's Multi-Level Linear Referencing System**
   Steven Kadolph, Ryan Wyllie, Iowa Department of Transportation
3. **Linking Highway Improvements to Patterns of Regional Growth and Land Use with Quasi-Experimental Research Design**
   Richard G. Funderburg, University of Iowa;
   Marlon G. Boarnet, University of California, Irvine;
   Hilary Nixon, San Jose State University

Session 2E
Asphalt Pavements

(Lisa Rolds,
FHWA Iowa Division,
Moderator)

1. **Development of Mix Design Process for Cold In-Place Recycling**
   Hosin Lee, Yongjoo Kim, University of Iowa
2. **Missouri's Use of Recycled Asphalt Shingles (RAS) in Hot Mix Asphalt**
   Joe Schroer, Missouri Department of Transportation
3. **Warm Mix Asphalt: Laboratory Evaluation and Pavement Design**
   Zhanping You, Michigan Technological University;
   Shu Wei Goh, Michigan Technological University

Session 2F
Modes

(Jason Bittner,
University of Wisconsin,
Madison,
Moderator)

1. **An Overview of Passenger Transportation in Iowa**
   Amanda Martin, Iowa Department of Transportation
2. **Rider and Non-Rider Opinions of Rural Public Transportation**
   Brian Geiger, Sunanda Dissanayake, Kansas State University
3. **Development of a Freight Analysis Framework for the Kansas City Metropolitan Area**
   Erin Wurfel, Transportation Research Institute, University of Kansas;
   Yong Bai, Luke Huan, Vincent Buhr, University of Kansas
4. **Freight Bottlenecks in the Upper Midwest: Identification, Collaboration, and Alleviation**
   Qi Gong, Andrew Obernesser, Jessica Guo, University of Wisconsin, Madison

Concurrent Sessions **Thursday, August 20th** **3:30 p.m. — 5:00 p.m.**

Session 3A
Traffic and Safety

(Mary Stahlhut,
Iowa DOT,
Moderator)

1. Effectiveness of Driver Improvement Programs
Wei Zhang, Konstantina "Nadia" Gkritza, Shashi Nambisan, Institute for Transportation, Iowa State University

2. Drivers' Preferences for LED Vehicular and Pedestrian Signals
Madhav V. Chitturi, University of Wisconsin, Madison;
Rahim F. Benekohal, Ali Hajbabaie, Juan C. Medina, University of Illinois at Urbana-Champaign

3. Safety Analysis for Older Drivers at Signalized Intersections in Kansas
Daba S. Gedafa, Kansas State University

4. Fatality Risk of Older Drivers under Different Conditions Based on Vehicle Miles Traveled
Abhiteja Rallabandi, Sunanda Dissanayake, Kansas State University

Session 3B
Structures and Construction

(Ahmad Abu-Hawash,
Iowa DOT,
Moderator)

1. Modified Sheet Pile Abutment for Low-Volume Road Bridge
Ryan R. Evans, F. Wayne Klaiber, Iowa State University;
David J. White, Terry Wipf, Caleb Douglas, Institute for Transportation, Iowa State University;

2. Post-Grouted Drilled Shaft Load Test Program Council Bluffs, Iowa
Curtis Monk, Iowa Division, Federal Highway Administration

3. Review of Ultra-High-Performance Concrete (UHPC) PI Girder Bridge in Buchanan County, Iowa (Design, Construction, Testing, and Monitoring)
Dean Bierwagen, Ahmad Abu-Hawash, Iowa Department of Transportation;
Brian Keierleber, Buchanan County Engineer;
Terry Wipf, Institute for Transportation, Iowa State University

4. Self-Centering Bridge Piers with Structural Fuses
Jon "Matt" Rouse, F. Wayne Klaiber, Iowa State University;
Mark C. Currie, HNTB;

Session 3C
Pavements (Sustainable Concrete Pavement Technologies, Part 2)

(Gordon Smith,
Iowa Concrete Paving
Association,
Moderator)

1. Two-Lift Concrete Pavement Construction
Peter Taylor, Institute for Transportation, Iowa State University

2. Concrete Pavement Surface Characteristics Program: An Update
Paul Wiegand, Institute for Transportation, Iowa State University;
Robert Otto Rasmussen, Transtec Group, Inc.;
Dale Harrington, Snyder and Associates;
Ted Ferragut, TDC Partners, Ltd.

3. Pervious Concrete Mix Design for Wearing Course Applications
Vernon R. Schaefer, Kejin Wang, Iowa State University;
John T. Kevern, University of Missouri, Kansas City

4. Quantifying Environmental Impact: What Should We Measure and How Is It Done?
Thomas Van Dam, Applied Pavement Technology, Inc.

Concurrent Sessions Thursday, August 20th 3:30 p.m. — 5:00 p.m.

Session 3D
Planning, Modeling and Logistics

(Garrett Pedersen,
Iowa DOT,
Moderator)

1. **The Iowa Travel Analysis Model iTRAM**
   Phil Mescher, Iowa Department of Transportation
2. **Infrastructure Asset Management in Hillsborough County, Florida**
   Michael Maltezos, Hillsborough County Public Works Dept.
3. **Assessing Transportation Impediments to Medicaid Members in the State of Iowa**
   Nikhil Sikka, Paul Hanley, University of Iowa
4. **Lessons Learned from 100% Airport Network Condition Survey**
   Jose A. Villarreal, Mustaque Hossain, Kansas State University;
   George Laliberte, Kansas Department of Transportation

Session 3E
Asphalt Pavements

(Bill Rosener,
Asphalt Paving Association
of Iowa,
Moderator)

1. **Alternative Test Methods for Measuring Permeability of Asphalt Mixes**
   R. Christopher Williams, Institute for Transportation, Iowa State University;
   Mohamed Abdel Raouf, Iowa State University;
   Joe Schroer, Missouri Department of Transportation
2. **Measurement and Evaluation of Tack Coat Application Rate for 4.75 mm NMAS Superpave Overlay Mix**
   Farhana Rahman, Mustaque Hossain, Kansas State University;
   Stefan Romanoschi, University of Texas at Arlington
3. **Some Observations on Sorption-Desorption Behaviors of Roller-Compacted Concrete Mixtures**
   Chetan V. Hazaree, Halil Ceylan, Kasthurirangan Gopalakrishnan, Iowa State University
4. **Utilization of the Mechanistic-Empirical Pavement Design Guide in Moisture Susceptibility Prediction**
   R. Christopher Williams, Tamer M. Breakah, Iowa State University

Session 3F
Technology Considerations

(Bruce Holdhusen,
Minnesota DOT,
Moderator)

1. **Combining Lidar and NAIP Imagery at Iowa DOT**
   Karen Carroll, Ryan Wyllie, Iowa Department of Transportation
2. **IaRTN and 511**
   Michael Jackson, Iowa Department of Transportation
3. **Plug-In Hybrid Electric Vehicles: Assessing Readiness for the Electrification of Personal Vehicle Transportation**
   Anthony Smick, Megan Mallette, Chris Getter, Jessica Guo, Giri Venkataramanan, University of Wisconsin, Madison

Concurrent Sessions **Friday, August 21st** **8:00 a.m. — 10:00 a.m.**

Session 4A
Traffic and Safety

(Tom Welch,
Iowa DOT,
Moderator)

1. Characteristics of Fatal Truck Crashes in the United States
Nishitha Bezwada, Sunanda Dissanayake, Kansas State University

2. Horizontal Curves – A New Method for Identifying At-Risk Locations for Safety Investment
Howard Preston, CH2M HILL, Inc.

3. Development of a Statewide Horizontal Curve Database for Crash Analysis
Zachary Hans, Reg Souleyrette, Institute for Transportation, Iowa State University;
Rachael Larkin, Iowa State University

4. Spatial Analysis of Crash Location Relative to Population Change
Jielin Sun, Adwetta Joshi, Lei Sun, Paul Hanley, University of Iowa

5. Review of Crashes at Bridges in Kansas
Mark Hurt, Kansas Department of Transportation;
Robert Rescot, Steven D. Schrock, University of Kansas

Session 4B
Hydrology

(Roger Schletzbaum,
Marion County, Iowa,
Moderator)

1. 2008 Floods in the City of Des Moines
William G. Stowe, City of Des Moines Public Works Department

2. The Iowa Floods of 2008—Iowa City
Ron Knoche, City of Iowa City

3. The Iowa Floods of 2008—Cedar Rapids
Dave Elgin, City of Cedar Rapids

4. Connecting Self-Similarity in Channel Network Topology to Scaling of Flood Data
Ricardo Mantilla, University of Iowa

5. Iowa's Floods of 2008 and the Iowa DOT's use of BridgeWatch to Monitor Scour Critical Bridges
Dave Claman, Iowa Department of Transportation

4. Sedimentation of Multi-Barrel Culverts
H-C. Ho, M. Muste, University of Iowa

Session 4C
Road Maintenance

(Bob Younie,
Iowa DOT,
Moderator)

1. Development of Updated Guidelines and Specifications for Roadway Rehabilitation
Charles T. Jahren, Ryan Shropshire, Iowa State University;
Paul Wiegand, Larry J. Stevens, Institute for Transportation, Iowa State University

2. Iowa DOT Winter Maintenance Research
Tina Greenfield, Iowa Department of Transportation

3. Modeling of Chip Seal Performance on Kansas Highways
Litao Liu, Mustaque Hossain, Kansas State University;
Rick Miller, Kansas Department of Transportation

4. Stabilization Procedures to Mitigate Edge Rutting for Granular Shoulders
Charles T. Jahren, Thang H. Phan, Chase Westercamp, Peter Becker, Iowa State University;
David J. White, Institute for Transportation, Iowa State University

Concurrent Sessions Friday, August 21st 8:00 a.m. — 10:00 a.m.

Session 4D
Environment
(Becky Hiatt,
FHWA Iowa Division,
Moderator)

1. **Comparison of On-Road Biodiesel Emissions in Transit Buses**
   Abhisek Mugdal, Shauna Hallmark, Massiel Orellana, Institute for Transportation, Iowa State University
2. **Asset Management Tool for Collecting and Tracking Commitments on Selected Environmental Mitigation Features**
   Teresa Adams, Jason Bittner, Stacy Cook, University of Wisconsin-Madison
3. **Effect of Pavement Type on Fuel Consumption and Emissions**
   Palinee Sumitsawan, Siamak A. Ardekani, Stefan Romanoschi, The University of Texas at Arlington
4. **Deploying Hybrid Electric School Buses in Iowa**
   Shauna Hallmark, Institute for Transportation, Iowa State University;
   Dennis Kroeger, Office of Motor Carriers and Highway Safety, Federal Highway Administration;
   Brittany L. Hallmark, Ankeny High School
5. **Cost Analysis of Alternative Culvert Installation Practices in Minnesota**
   Brad Hansen, John Nieber, Chris Lenhart, University of Minnesota

Session 4E
Asphalt Pavements
(John Hinrichsen,
Iowa DOT,
Moderator)

1. **A Framework for Performance-based Permeability and Density Acceptance Criteria for HMA Pavements in Wisconsin**
   Sam Owusu-Ababio, Robert L. Schmitt, University of Wisconsin, Platteville
2. **Dynamic Modulus of HMA: Preliminary Criteria to Prevent Field Rutting of Asphalt Pavements**
   Zhanping You, Shu Wei Goh, Michigan Technological University;
   R. Christopher Williams, Iowa State University
3. **Development of Accelerated Superpave Mix Testing Models**
   Chandra Manandhar, Mustaque Hossain, Paul Nelson, Kansas State University;
   Cliff Hobson, Kansas Department of Transportation
4. **Simulation of Flexible Pavement Design in Kansas**
   Daba S. Gedafa, Mustaque Hossain, Kansas State University;
   Stefan A. Romanoschi, University of Texas at Arlington;
   Andrew J. Gisi, Kansas Department of Transportation

Session 4F
Technology Considerations
(Jerry Roche,
FHWA Iowa Division,
Moderator)

1. **Iowa DOT Pavement Marking Practices**
   William Zitterich, Iowa Department of Transportation
2. **Spatial Infrastructure at the Iowa DOT**
   Eric Abrams, Iowa Department of Transportation
3. **An Overview of ITS Research Project by the Iowa DOT**
   Willy Sorenson, Iowa DOT
4. **An Analysis of Emergency Message Delivery Scheme in Inter-Vehicular Networking**
   Weidong Xiang, Hong Nie, University of Michigan
5. **Feasibility of Using Cellular Telephone Data to Determine the Truckshed of Intermodel Facilities**
   Ming-Heng Wang, Steven D. Schrock, Thomas Mulinazzi, University of Kansas

Concurrent Sessions **Friday, August 21st** **10:30 a.m. — 12:00 p.m.**

Session 5A
Traffic and Safety

(Brad Estochen,
Minnesota DOT,
Moderator)

1. **Determining the Effectiveness of Portable Changeable Message Signs in Work Zones**
   Umar Firman, Yue Li, Yong Bai, University of Kansas
2. **Effectiveness of Dynamic Messaging on Driver Behavior for Late Merge Lane Road Closures**
   Thomas J. McDonald, Robert Sperry, Shashi Nambisan, Institute for Transportation, Iowa State University
3. **Evaluation of Technology-Enhanced Flagger Devices: Focus Group and Survey Studies in Kansas**
   Chen Fei See, Steven D. Schrock, Wai Kiong "Oswald" Chong, Yong Bai, Jamila Saadi, University of Kansas
4. **Characteristics of Work Zone Crashes in the SWZDI Region: Differences and Similarities**
   Sunanda Dissanayake, Sreekanth Reddi Akepathi, Kansas State University

Session 5B
Structures and Construction

(Brent Phares,
Iowa State University,
Moderator)

1. **I35W Bridge: Accelerated Procurement Design and Construction**
   Kevin Western, Minnesota Department of Transportation
2. **Multimodal Condition Assessment of Bridge Decks by NDE and Its Validation**
   Nenad Gucunski, Ruediger Feldmann, Francisco Romero, Sabine Kruschwitz, Rutgers University;
   Ahmad Abu-Hawash, Mark Dunn, Iowa Department of Transportation
3. **Web-Based Collaboration for Iowa DOT Bridge Construction**
   James S. Nelson, Iowa Department of Transportation;
   Aaron C. Zutz, Charles T. Jahren, Iowa State University

Session 5C
Highway Engineering and Construction

(Mark Nahra,
Woodbury County, Iowa,
Moderator)

1. **Methods at Iowa DOT—Flooded Backfill and Plastic Pipe**
   Deanna Maifield, Dean F. Herbst, Iowa Department of Transportation
2. **Construction Project Administration and Management for Mitigating Work Zone Crashes and Fatalities: An Integrated Risk Management Model**
   Daniel Enz, Jennifer Shane, Kelly Strong, Iowa State University
3. **Identification of Practices, Design, Construction, and Repair Using Trenchless Technology**
   Charles T. Jahren, Muhannad T. Suleiman, Halil Ceylan, Bill Conway, Iowa State University;
   Larry J. Stevens, Institute for Transportation, Iowa State University

Session 5E

Materials

(Scott Schramm,
Iowa DOT,
Moderator)

1. **Determination of Pre-treatment Procedure Required for Developing Bio-binders from Bio-oils**
   Mohamed Abdel Raouf, R. Christopher Williams, Iowa State University

2. **Antioxidant Effect of Bio-Oil Additive ESP on Asphalt Binder**
   Sheng Tang, R. Christopher Williams, Iowa State University

3. **Investigation of Warm-Mix Asphalt Performance Using the Mechanistic-Empirical Pavement Design Guide**
   Ashley Buss, Mohamed Rashwan, Tamer Breakah, R. Christopher Williams, Iowa State University; Andrea Kvasnak, Auburn University

4. **Nanotechnology to Manipulate the Aggregate-Cement Paste Bond: Impacts on Concrete Performance**
   Jessica M. Sanfilippo, Jose F. Muñoz, M. Isabel Tejedor, Marc A. Anderson, Steven M. Cramer, University of Wisconsin-Madison

About the Symposium

The Mid-Continent Transportation Research Symposium provides an opportunity for transportation professionals from the Midwest and beyond to network with their peers, learn about advancements and applications in their fields, and future directions for research.

Researchers and practitioners from around the country will present papers at this sixth biennial event at Iowa State University. The day-and-a-half symposium will cover a broad spectrum of transportation issues with sessions on both basic and applied research.

Symposium Planning Committee

Sandra Larson, Director, Research and Technology Bureau, Iowa Department of Transportation

Shashi Nambisan, Director, Institute for Transportation, Iowa State University

Ahmad Abu-Hawash, Chief Structural Engineer, Iowa Department of Transportation

John Adam, Director, Statewide Operations Bureau, Iowa Department of Transportation

Teresa Adams, Director, National Center for Freight & Infrastructure Research & Education, Professor of Civil Engineering, University of Wisconsin–Madison

Jim Alleman, Chair, Department of Civil, Construction, and Environmental Engineering, Iowa State University

Stu Anderson, Director of Planning, Programming and Modal Division, Iowa Department of Transportation

Steve Andrle, Chief Program Officer / Capacity, SHRP 2, Transportation Research Board

Jason Bittner, Deputy Director, National Center for Freight & Infrastructure Research & Education, University of Wisconsin–Madison

Linda Boyle, Associate Professor of Mechanical and Industrial Engineering, University of Iowa

Dennis Burkheimer, Winter Operations Administrator, Iowa Department of Transportation

Tom Cackler, Director, National Concrete Pavement Technology Center, InTrans, Iowa State University

Mara Campbell, Director, Organizational Results, Missouri Department of Transportation

Carol Culver, Research Coordinator, Iowa Department of Transportation

Mark Dunn, Operations Research Director, Highway Research Board, Iowa Department of Transportation

Nadia Gkritza, Assistant Professor of Civil Engineering, Iowa State University

Max Grogg, Program Delivery Engineer, Federal Highway Administration–Iowa Division

Paul Hanley, Director, Transportation Policy Research Program, Public Policy Center, University of Iowa

Neal Hawkins, Associate Director of Transportation Operations, InTrans, Iowa State University

Bret Hodne, Public Works Director (interim), City of West Des Moines

Brian Keierleber, Buchanan County (Iowa) Engineer

Becky Hiatt, Planning & Development Manager, FHWA–Iowa Division

Aemal Khattak, Associate Professor of Civil Engineering, University of Nebraska–Lincoln

Mike LaViolette, Bridge Practice Leader, HNTB Corporation, Omaha

Hosin "David" Lee, Associate Professor, Department of Civil and Environmental Engineering, University of Iowa

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Tom Maze (1952–2009)

These proceedings are dedicated to the memory of Professor Tom Maze, who had the vision to initiate this important biennial event, and who nurtured from infancy to maturity the organization that is today's Institute for Transportation at Iowa State University.

2009 Mid-Continent Transportation Research Symposium
Ames, Iowa, August 20–21, 2009
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The papers, extended abstracts, and posters included in these proceedings summarize around 135 presentations scheduled for this seventh biennial event at Iowa State University. Short abstracts are also included that summarize presentations for which extended abstracts, posters, or manuscripts were not available at the time of publication of these proceedings. The topics cover a broad spectrum of transportation issues in the following areas: education and workforce development; environment; finance; freight; geotechnical engineering; highway engineering and construction; hydrology; materials; transportation modes; pavements; planning, modeling, and logistics; road maintenance; structures and construction; technology considerations; and traffic and safety. These proceedings also include materials related to keynote presentations by Dr. Martin Wachs of the RAND Corporation and Dr. Ann Brach of the Transportation Research Board.

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Terry Wipf, Director, Bridge Engineering Center, InTrans, Professor of Civil Engineering, Iowa State University
Daniel Yeh, Section Chief, Research and Communication Services, Wisconsin Department of Transportation

Spatial Infrastructure at the Iowa DOT

Eric Abrams
Iowa Department of Transportation
800 Lincoln Way
Ames, Iowa 50010
Eric.abrams@dot.iowa.gov

ABSTRACT

This presentation will look at the Iowa Department of Transportation's (Iowa DOT's) spatial infrastructure from staffing to technology. The focus will be on implementation of the Iowa DOT's geospatial service layer GeoNexus and how this acts as a central hub for geospatial data.

Key words: GIS—service layer—spatial

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Interactions between Transportation Capacity, Economic Systems, and Land Use

Stephen Andrle
Strategic Highway Research Program 2
Transportation Research Board
500 Fifth Street NW
Washington, D.C. 20001
sandrle@nas.edu

ABSTRACT

This presentation describes research that included studying the interactions between transportation capacity, economic systems, and land use. Objectives included determining net economic changes in the area of impact of a transportation improvement, developing enough cases to demonstrate impacts by analogy, and linking to a collaborative decision-making framework.

Key words: collaborative decision making—economic impact analysis

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Effectiveness of Localized Deer Management in Reducing Deer-Vehicle Crash Rates in Iowa—Some New Evidence

Michael J. Baird
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
mbaird@iastate.edu

Zachary Hans
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
zhans@iastate.edu

Konstantina Gkritza
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
nadia@iastate.edu

ABSTRACT

Using data on deer population from the Iowa Department of Natural Resources (Iowa DNR) as well as data on deer-related vehicle crashes from the Iowa Department of Transportation (Iowa DOT), this study investigates the effectiveness of localized deer management on the reduction of deer-vehicle crashes in three selected cities in Iowa over the period 2002–2007. Preliminary results, including trends (increasing or decreasing) in deer population and deer-vehicle crashes during the analysis period, were presented at the 2008 Mid-Continent Transportation Symposium. This paper presents some new findings based on additional data collection. In addition to the deer-vehicle crash data, carcass reports were collected and geo-coded. The deer carcass locations were assigned to the nearest milepost and the carcass data were converted into a form that would be compatible to use in geographic information systems software. The deer-vehicle crash database maintained by the Iowa DOT was compared to that of the carcass data to eliminate double counting of crashes. Visualization tools and statistical analysis were used to establish a relationship between the deer-vehicle crash rates and the annual deer population counts within each selected city. Concluding remarks and recommendations are offered regarding the effectiveness of the localized deer management in Iowa. In addition, this study illustrates the types of data necessary to document the effectiveness and demonstrates how analysis can be carried out and ultimately improved.

Key words: deer-vehicle crash—localized deer management

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Precast Bridge Approach Slab Replacement Project in Denver, Iowa (Design, Casting, Construction, and Monitoring)

Dean Bierwagen
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Dean.Bierwagen@dot.iowa.gov

Wayne Sunday
Office of Construction
Iowa Department of Transportation
800 Lincoln Way
Ames, IA, 50010
Wayne.Sunday@dot.iowa.gov

Ahmad Abu-Hawash
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Ahmad.Abu-Hawash@dot.iowa.gov

ABSTRACT

Bridge owners are frequently faced with the need to replace critical bridge components during limited or overnight road closure periods. This paper presents the development, testing, installation, and monitoring of a precast concrete bridge approach slab specifically designed by the Iowa Department of Transportation to address the problem of deteriorated bridge approach slabs and the need for accelerated replacement.

A precast concrete approach slab was designed and constructed on twin bridges north of Waterloo on Highway 63. Driving lane and shoulder sections of the bridge approach were replaced in 11-hour windows during the day and at night to demonstrate the feasibility of using the precast bridge approaches in an accelerated repair procedure. The contractor was able to meet the strict time requirements using the precast units and successfully completed the work.

Key words: accelerated bridge repair—precast pavement

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Review of Ultra-High–Performance Concrete (UHPC) PI Girder Bridge in Buchanan County, Iowa (Design, Construction, Testing, and Monitoring)

Dean Bierwagen
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Dean.Bierwagen@dot.iowa.gov

Ahmad Abu-Hawash
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Ahmad.Abu-Hawash@dot.iowa.gov

Brian Keierleber
Buchanan County Engineer
1511 First St. E.
Independence, IA 50644
bcengineer@co.buchanan.ia.us

Terry Wipf
Bridge Engineering Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
tjwipf@iastate.edu

ABSTRACT

Buchanan County, Iowa, was granted funding through the TEA-21 Innovative Bridge Construction Program (IBRC), managed by the Federal Highway Administration (FHWA), to construct a highway bridge using an optimized PI girder section with ultra-high–performance concrete (UHPC). UHPC is a relatively new structural material that is marketed by Lafarge, Inc., under the name Ductal®. The PI girder section was developed to optimize the amount of material used in a girder, since currently, the cost is relatively expensive. The Buchanan County project was the first time the PI section was used for a highway bridge in the United States. The girders were pretensioned longitudinally, and the deck was then connected transversely with dowels in grouted pockets.

The design was based on conventional as well as finite element analysis, which was validated by prior laboratory testing at the FHWA Turner Fairbank Laboratory in McLean, VA, near Washington, D.C. The paper will provide information regarding the design, casting, construction, testing, and monitoring of the PI girder bridge.

Key words: Ductal concrete—Lafarge North America—PI section—reactive powder concrete—steel fibers—ultra-high–performance concrete

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Asset Management Tool for Collecting and Tracking Commitments on Selected Environmental Mitigation Features

Teresa Adams
National Center for Freight and Infrastructure
Research and Education
University of Wisconsin, Madison
2205 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
adams@engr.wisc.edu

Stacy Cook
Department of Urban and Regional Planning
University of Wisconsin, Madison
2205 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
stacymcook@gmail.com

Jason Bittner
National Center for Freight and Infrastructure
Research and Education
University of Wisconsin, Madison
2205 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
bittner@engr.wisc.edu

ABSTRACT

Wisconsin has constructed many environmental mitigation projects in conjunction with transportation projects that have been implemented pursuant to the National Environmental Policy Act (NEPA). Other mitigation projects have been constructed pursuant to discussions and negotiations with the Wisconsin Department of Natural Resources (WDNR). These mitigation projects offset or replace a certain environmental function(s) lost as a result of construction of the transportation project. Examples include storm water management facilities, wetland replacement projects, stream restoration projects, reforestation projects, construction of sound walls, replacement of parklands and wildlife crossing structures.

In order for the environmental mitigation projects to continue to provide long-term functionality intended when they were first constructed, they must be properly maintained and, when necessary, rehabilitated or reconstructed. These environmental mitigation projects may be considered as assets similar to other transportation features. The Wisconsin Department of Transportation (WisDOT) identified the need to track these selected features in the overall scheme of project development and ongoing maintenance.

This project explored the current state of environmental mitigation project activities, discussed the literature on existing environmental inventory and asset management programs, developed an inventory of selected environmental mitigation features in Wisconsin, and developed a tool to track commitments in order to help WisDOT provide long-term functionality intended when the mitigation features were first constructed.

The research approach to develop the tracking tool and list of priority features and inventory included interviews with WisDOT and WDNR staff, a review of WisDOT environmental records such as environmental impact statements (EIS) and environmental assessment (EA) documents, and a literature survey regarding best practices for environmental tracking and asset management systems being used by other state agencies.

Key words: asset management—environmental commitment—mitigation—NEPA—tracking

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The Use of Propane as a Subbase Drying Technique

Michael Blahut
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
394 Town Engineering
Ames, IA 50011
meblahut@iastate.edu

R. Christopher Williams
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
482 Town Engineering
Ames, IA 50011
rwilliam@iastate.edu

Vernon R. Schaefer
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
488 Town Engineering
Ames, IA 50011
vern@iastate.edu

ABSTRACT

The use of propane as a soil drying technique is being studied to determine the technical and economic feasibility for use in transportation-related projects. The need originates from the earthwork portion of a construction schedule, which can be severely delayed if the soil is too wet for compaction. The ultimate goal is to develop an implemention that can be used in the field and can be adapted to work with existing equipment. The laboratory drying device has provisions to receive the propane fuel, transfer the propane to heat through interchangeable instruments, output the heat onto the soil, and till the soil as it passes over the sample. A laboratory-based technical feasibility analysis is currently underway to determine the amount of propane in British Thermal Units (BTUs) that is required to dry a soil to a desirable level with a higher than optimum initial moisture content. Several types of drying instruments will be implemented, with all using propane as their fuel source. Some of these heating instruments include forced air, direct flame, and infrared heating. Soils typically used in Iowa for subbase and base courses will be used. The properties of the soils will be measured before and after drying. The properties that will be measured are dry unit weight, moisture content, and the Atterberg limits. All soil types could benefit from a drier device, but those with more than 20% fines content will benefit the most.

Key words: compaction—earthwork—propane—soil drying

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INTRODUCTION

This project examines the technical and economic feasibility of using propane as a heating source to assist in drying wet subbase and base materials in transportation earthwork projects. A laboratory-based technical feasibility analysis is being conducted along with conceptual development of field equipment for combined soil scarifier and propane heater. Additionally, the economic feasibility of propane-assisted drying of subbase and base soils will be evaluated.

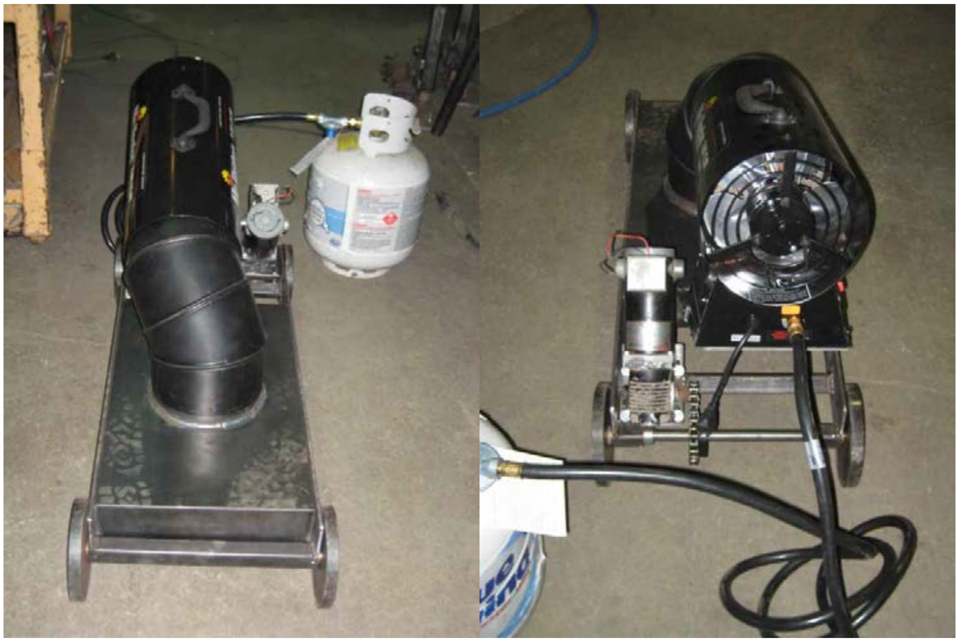
BACKGROUND

The moisture condition of subbase and base layers can greatly impact the placement and compaction effort required along a highway construction project. Generally, placement near the optimum moisture content is desired to obtain suitable engineering properties. Often, soils available for subbase and base layers are too wet for use and must be dried prior to placement. Available methods to dry soils include scarification and exposure to sun and wind through spreading of the soil and discing to turn the soil and the addition of chemical admixtures such as fly ash, lime, or cement that absorb the excess water in the soil. These methods are time-consuming and expensive. A device or equipment to dry unbound materials that are beyond optimum moisture content could lead to faster construction of highways projects. Less than 40 years ago, a contractor built a device to dry wet earth fill prior to compaction. Great Lakes Construction Company of Cleveland, Ohio, was awarded the contract to build a four-level interchange for I-77 and I-80. The contractor built a jet drier and was able to decrease the time to compact one lift by one-half to one-third the time compared to air drying. The jet drier modified a Caterpillar D8H dozer to include a rotating scarifier that mixes the produced hot air with the soil, and a jet engine from a navy fighter plane. The rotating scarifier would penetrate the soil about 9 in. to mix up the soil while being heated by the hot gases from the jet exhaust. The development of a propane heater to dry soil could potentially increase the speed of construction depending on the soil type. All soil types could benefit from a drier device, but fine-grained soils with greater than 20% fines could have the highest benefit in terms of drying times. An effective means of drying soils involves a combination of tilled soil, wind, and sunshine. The important components of a propane heater for drying soil will be a scarifier to till the soil and a gas-fired, forced-air propane heater to simulate the wind and sunshine.

METHODS AND MATERIALS

Development of Lab Scale Device

The development of a laboratory scale device was predicated on an ability to deliver heat to the surface of a soil layer in the lab, while at the same time providing for scarification of the soil. Preliminary meetings were held with a large earthwork contractor to discuss desirable features in a field-scale device so that such features could be incorporated into a lab prototype. Key features for a lab model included an ability to raise and lower the heat source, self propulsion, control of the heat amount, scarification, and use of portable propane tanks. Conceptual drawings were developed and discussed amongst the research team, a heating specialist, and a metal fabricator to move the concept forward to the point of fabricating a lab prototype device. The research team decided the most economical method to apply a substantial amount of British Thermal Units (BTUs) for soil drying would be to use a forced air system. The lab scale unit is rated for 75,000 to 125,000 BTU per hour. The unit is approximately 16 in. wide, 24 in. in length, and about 15 in. in height. The unit can vary in elevation from about 2 in. from a soil surface to an 8 in. clearance. The system operates with a variable speed motor, a chain drive system, and a controller to be installed shortly for speed control. Front and back views of the device are shown in Figure 1.



(a) Front view **(b) Back view**

Figure 1. Lab-scale soil drying device

The device was completed by the fabricator and delivered to the Iowa State University Geotechnical Laboratory in early April 2009. A side view of the fabricated device is shown in Figure 2. The system operates with a variable speed motor, a chain drive system , and a controller for speed control. A 40 lb propane tank with a 20 ft hose attaches to the device while in operation. A soil tilling mechanism is mounted on the front of the device. The tined based type tiller is currently being used. A disc type tiller is currently being designed and will be fabricated soon.

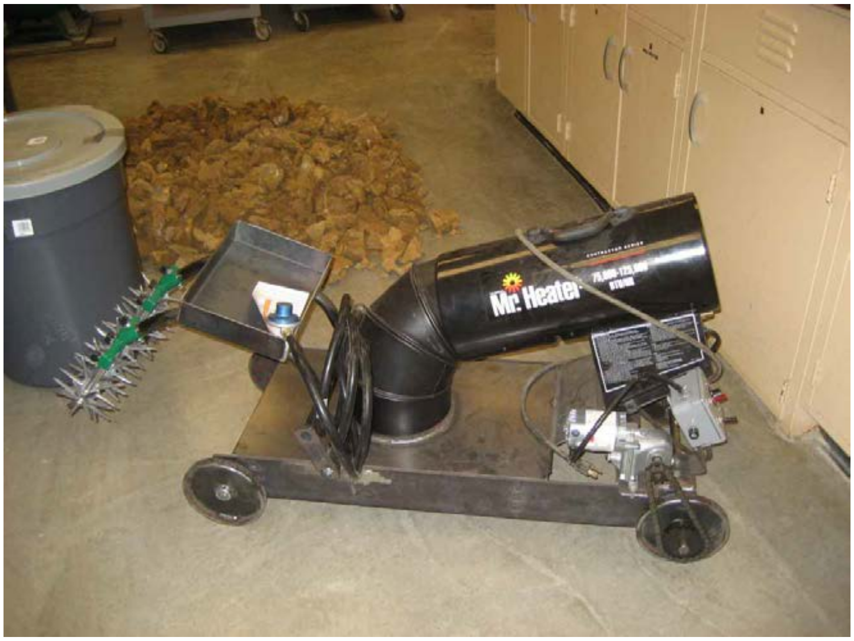


Figure 2. Lab-scale soil drying device, side view with tiller attachment

Experimental Plan for Soils Testing

The first step of the drying process will involve application of heat to the soil. This has to be done in a timed manner. Second, the soil will be mixed for a fixed amount of time. Third, either more heat can be applied, or a sample of soil can be measured for water content change over the last heat cycle. For all soil types and equipment, the water content will be measured at different time intervals to determine the most efficient use of propane, mixing, and time.

Considering different soils will take varying amounts of propane heat and time to dry; an experimental plan has been developed, as shown in Table 1. The plan consists of five main experimental factors: soil type, equipment type, device elevation, device speed, and number of passes. The research team’s preliminary thinking is that three soils will be tested initially, will consist of three different types of tilling systems (no tilling, disc tilling, and tine tilling), three different elevations (2, 5, and 8 in. from soil surface), and three different speeds. Considering the factors, a full factorial design would consist of the following: three soils, three tilling systems, three elevations, three speeds, and three passes. This would result in 243 test runs with the laboratory device, and thus, a partial factorial will be pursued.

Table 1. Proposed experimental plan for soil testing

		Tilling								
		None			**Disc**			**Tines**		
	Elevation	2”	5”	8”	2”	5”	8”	2”	5”	8”
Soil Type	Fat clay	X	X	X	X	X	X	X	X	X
	Lean clay	X	X	X	X	X	X	X	X	X
	Sand	X	X	X	X	X	X	X	X	X

The testing will be done initially on the fat clay with the three different speeds and three passes with the moisture content of the soil being measured after each pass. Based upon the results of the fat clay, a more focused partial factorial will be done for the lean clay and sand. In this manner, the results of the fat clay can best be used to project the anticipated outcomes of the lean clay and sand, and thus, not as many factors will be considered.

Soil Classification

A high plasticity silty clay was selected as the first test soil because it is the most difficult to dry and is the most troublesome soil to deal with in the field. Sometimes it takes days for soil of this type to dry out on its own in the field, so implementation of this device could be beneficial. The silty clay is proven to be highly plastic by the liquid limit of 50. These types of clay hold moisture very well. The properties of the silty clay are shown in Table 2.

Table 2. Soil properties for silty clay material

Liquid Limit	50
Plastic Limit	24
% Passing No. 200 Sieve	72.0%
Specific Gravity	2.61
Optimum Moisture Content	20.5%
Maximum Dry Density	103.0 pcf

RESULTS

The shakedown testing of the device is being conducted on a concrete floor to ensure efficient operation and use of the silty clay soil. A soil test strip of 5 ft long,1 ft wide, and 1 1/2 in. thick makes optimum use of the device dimensions and available laboratory space. To begin the tests, the propane tank was weighed, the soil was wetted to a target of 30% moisture (approximately 10% above the optimum moisture content), the heat setting was on low, and the speed setting was on 6 out of 10 (approximately 4 ft per minute). During initial trials it was determined that the low heat setting was more efficient because on a higher setting there was too much heat loss towards the outside of the device. The device is shown in operation in Figure 3.

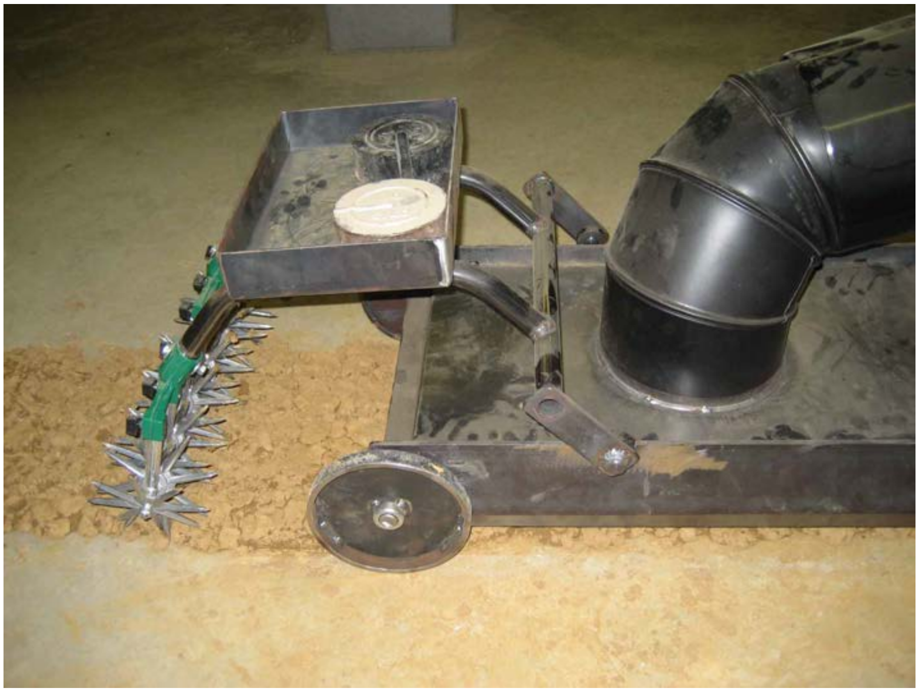


Figure 3. Soil drying device in operation

The device is equipped with a reversible, variable speed motor. After a single pass, the motor was switched to the reverse position to allow another pass. Moisture contents were taken at the beginning of the test and originally after every four passes. Subsequent tests revealed that taking moisture contents every eight passes would produce longer uninterrupted heating cycles and therefore greater moisture content change. A single pass takes approximately 5 min to complete. Table 3 shows the moisture content change per pass in early trials. It can be seen that all three tests managed to dry the soil to just drier than the optimum moisture content (20.5%). The moisture content change per pass was about one-half of a percentage point, indicating that multiple passes are necessary to dry soils that are very wet of optimum. Tests conducted in which the moisture content was measured in the upper and lower zones of the soil indicate the need to better mix the soils during heating. Hence, efforts will be made to find a more efficient tilling system, such as discs, that would better optimize the mixing of the soil while the heat is applied. The amount of propane used for each test is determined by weighing the tank before and after application of the heat (Figure 4).

Table 2. Preliminary results of drying tests

Test #	# of passes	Initial moisture content, %	Average total MC change, %	MC change per pass, %
1	24 (4 passes x 6 times)	32.1	13.6	0.567
2	20 (4 x 5)	29.2	9.2	0.46
3	20 (4 x 1 & 8 x 2)	31.0	12.5	0.625

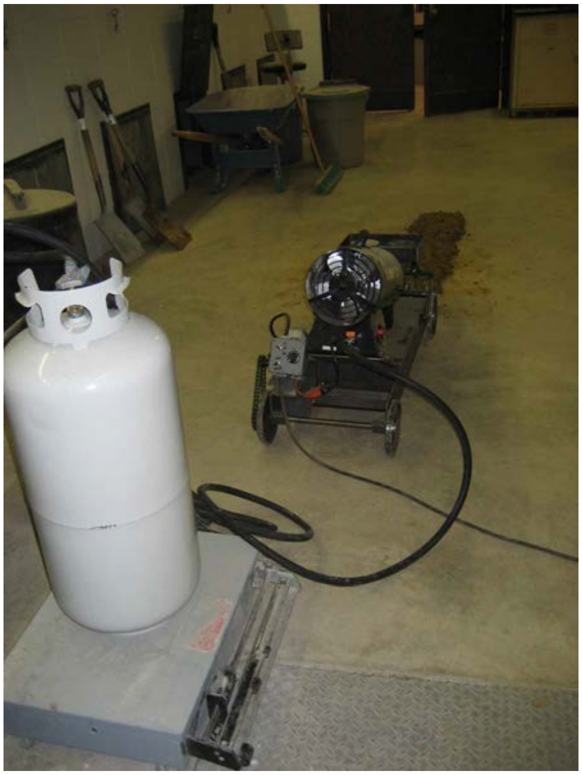


Figure 4. Soil drying device in operation

SUMMARY

A laboratory-scale prototype for drying soils has been constructed and is presently being tested. Preliminary results with a fat silty clay soil show that wet soils can be effectively dried through several passes of the device. Future enhancements to the device include fabrication of disc tillers for the laboratory device and a track to guide the device and allow for higher soil depths. Two additional soils—a sand and Iowa loess—will be subject to testing. Economic analysis of the cost of drying the soils will be conducted to ascertain the economic feasibility of propane soil drying.

SHRP2: Accomplishments and Opportunities

Ann Brach
Strategic Highway Research Program 2
Transportation Research Board
500 Fifth Street, NW
Washington, D.C. 20001

ABSTRACT

This presentation discusses the accomplishments and opportunities of the second Strategic Highway Research Program (SHRP2). The four research focus areas of safety, renewal, reliability, and capacity are reviewed, and recommendations for the future are provided.

Key words: capacity—reliability—renewal—safety—Strategic Highway Research Program 2

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Utilization of the Mechanistic-Empirical Pavement Design Guide in Moisture Susceptibility Prediction

Tamer M. Breakah
Department of Civil, Construction, and Environmental Engineering
Iowa State University
174 Town Engineering
Ames, IA 50011
breakah@iastate.edu

R. Christopher Williams
Department of Civil, Construction, and Environmental Engineering
Iowa State University
174 Town Engineering
Ames, IA 50011
rwilliam@iastate.edu

ABSTRACT

Moisture susceptibility of asphalt pavements is considered a major problem that shortens pavement service life. Moisture susceptibility is most commonly tested using the modified Lottman test. The shift towards mechanistic design calls for the utilization of a more fundamental test to evaluate moisture damage. It has been recommended to use the dynamic modulus test for moisture damage evaluation. The dynamic modulus results can be used as an input for the *Mechanistic-Empirical Pavement Design Guide* (MEPDG).

This research used field-procured/laboratory-compacted mixtures from the State of Iowa. The mixes had varying levels of moisture susceptibility. Two sets of samples were tested for dynamic modulus. The first set was a control set, while the second was moisture conditioned. The MEPDG was utilized to predict the pavement response. The simulation results showed that the MEPDG is a good tool to predict the effect of moisture on the major pavement distresses. The results also showed a difference between the various mixes in the amount of distress levels associated with their moisture susceptibility.

Key words: hot mix asphalt—MEPDG—moisture susceptibility—pavement distresses

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INTRODUCTION

Pavements are subjected to different stresses during their design lives. A properly designed pavement will perform adequately during its design life, and the distresses will not exceed the allowable limits. A good design is one that provides the expected performance with appropriate economic considerations. One of the factors that leads to premature failure of pavements is moisture sensitivity. The presence of water in pavements can be detrimental if combined with other factors such as freeze-thaw cycling. Many factors can affect the moisture sensitivity of a mix and can be divided into three main categories. The first category is the material properties, which include the physical and chemical properties of the asphalt and the aggregates. The second category is the mixture properties, which include asphalt content, film thickness, and the permeability of the mixture (interconnectivity of the air voids). The third category is the external factors; these factors include construction, traffic, and environmental factors (Santucci 2002). The American Association of State Highway and Transportation Officials (AASHTO) *Mechanistic-Empirical Pavement Design Guide* (MEPDG) enables a designer to simulate many of these factors, but it does not include the effect of moisture on mixture properties. The objective of this paper is to investigate the effect of moisture on three different asphalt-concrete mixes by testing the material properties for each of them with and without moisture conditioning and by simulating the results using the MEPDG.

BACKGROUND

Moisture damage has been a major concern to asphalt technologists for many years. Researchers have been searching for a test that differentiates between good- and bad-performing asphalt-concrete mixtures from stripping potential since the 1920s (Solaimanian et al. 2003). Since the 1920s, it has been known that the problem relates to the loss of adhesion between asphalt and aggregate and the loss of cohesion within asphalt. The challenge has been to find a test that identifies moisture susceptible mixes (Solaimanian et al. 2003). The standard test used to identify the moisture susceptibility of asphalt mixtures is the modified Lottman test (AASHTO T 283). Although AASHTO T 283 has been used for several years as the standard test for moisture sensitivity, it assists in minimizing the problem, and it does not appear to be a very accurate indicator of stripping (Brown et al. 2001). One test that has the potential to replace indirect tensile-strength testing contained within AASHTO T 283 is the dynamic modulus test. This test has been around for a long period of time, but its use started to become more common when the Federal Highway Administration (FHWA) developed a request for proposals for a research project to develop a simple performance test in 1996 (McGhee 1999). This test can characterize the performance of asphalt mixtures to be used in a particular pavement layer based upon fundamental engineering properties in conjunction with the established volumetric-testing procedures. Various tests were employed, analyzed, and correlated with performance data from test-track facilities that could be used as the Superpave Simple Performance Test (SPT). Witczak et al. (2002) has shown that dynamic modulus and flow number have promising correlations with field performance (Witczak et al. 2002). The dynamic modulus test is commonly used with the MEPDG. The MEPDG is a design guide developed after the proposal made in 1996 by the AASHTO Joint Task Force in Pavements. The MEPDG includes computational software that provides a prediction of pavement performance taking into consideration traffic, climate, and pavement structure; special consideration of loading with multiple tires and axles; and an approach for evaluating design variability and reliability (NCHRP 2004).

The use of the dynamic modulus test for moisture susceptibility evaluation was recommended by the National Cooperative Highway Research Program (NCHRP) Report 589 (Solaimanian et. al 2007). The researchers also suggested that the results of the dynamic modulus could be simulated using the MEPDG (Solaimanian et al. 2007).

EXPERIMENTAL PROGRAM

Loose field mix was procured from three projects in the state of Iowa. The three mixes are labeled Rose, 235S (2005 construction season), and 235I (2006 construction season). The loose field mixes were compacted with a Superpave Gyratory Compactor, using a 100 mm diameter mold compacted to a 150 mm height. A total of 10 samples were compacted for each project. The samples were divided into two groups, each containing five samples with equal average air voids. One group was tested dry and used as the control group, while the other group was moisture conditioned. The testing procedure for dynamic modulus testing was derived from the NCHRP Report 513 "Simple Performance Tester for Superpave Mix Design" (Bonaquist et. al 2003). Four linear variable displacement transducers were mounted on the sides of the specimen with a gauge length of 100 mm. The sample was then axially loaded under a strain-controlled test at 80 microstrains. The test setup is shown in Figure 1. The conditioning of the specimens followed the procedure outlined in AASHTO T 283, with one freeze-thaw cycle. A total of nine test frequencies were run at two test temperatures for inclusion in MEPDG simulations. The concept of time-temperature superposition was used to develop a master curve for each mix. The master curve was used to calculate the dynamic modulus at other temperature-frequency combinations to satisfy the MEPDG input requirements.

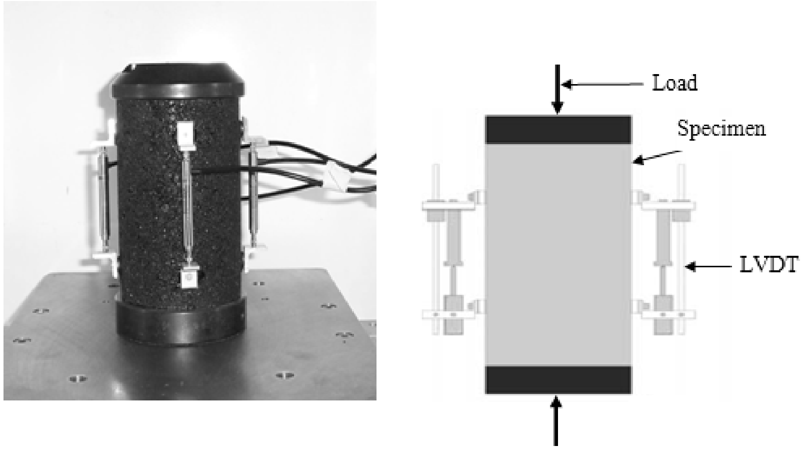

Figure 1. Dynamic modulus test configuration

MOISTURE SUSCEPTIBILITY TESTING

Table 1 summarizes the dynamic modulus results for the three mixes for both the conditioned and unconditioned groups. The ratios of dynamic modulus of moisture-conditioned samples to unconditioned samples (E* ratio) are presented in Table 2. It can be seen from the results that the best performer was 235S, followed by 235I, and the worst performer was Rose. It is also evident that the E* ratio values increase with the increase of frequency and decrease with the increase of temperature.

MEPDG SIMULATION

The MEPDG was used to investigate the difference between the two projects for both control and moisture conditioned samples. The cross section shown in Figure 2 was used as a typical cross section to evaluate the difference in pavement performance. All the inputs were maintained constant in all the designs, except those of the top layer. To be able to capture the effect of the moisture conditioning on the pavement performance, a level 1 design was used for the top asphalt-concrete layer. The results from the dynamic modulus tests for the two sample groups were used together with the results from the volumetric analysis of the specimens and the properties of the asphalt binder used. Level 3 design was used for all the other layers. The location of the project was assumed to be Des Moines, Iowa, and a traffic spectrum was used that gives a traffic level equivalent to 10,000,000 equivalent single axle loads (ESALs).

Table 1. Dynamic modulus test results

			E* (GPa)					
			Frequency (Hz)					
Project	Conditioning	Temperature (°C)	0.1	1	5	10	15	25
Rose	Control	4	10.33	13.07	14.96	15.65	16.34	16.39
		21	3.30	5.09	6.83	7.60	8.13	8.86
	Moisture	4	8.17	11.02	13.31	13.76	14.49	15.44
		21	2.27	3.84	5.63	6.39	6.86	7.52
235S	Control	4	6.62	9.22	11.40	12.30	13.02	13.83
		21	1.64	2.81	4.40	5.09	5.50	6.13
	Moisture	4	7.23	10.36	12.89	14.00	14.69	15.88
		21	1.81	3.40	5.22	6.07	6.62	7.40
235I	Control	4	6.43	9.37	11.67	12.62	13.35	14.13
		21	1.46	2.62	4.18	4.89	5.34	5.90
	Moisture	4	5.39	7.85	10.13	11.06	11.73	12.70
		21	1.20	2.24	3.70	4.38	4.81	5.34

Table 2. Dynamic modulus ratios

		E* (Ratio)					
		Frequency (Hz)					
Conditioning	Temperature (°C)	0.1	1	5	10	15	25
Rose	4	0.79	0.84	0.89	0.88	0.89	0.94
	21	0.69	0.75	0.82	0.84	0.84	0.85
235S	4	1.09	1.12	1.13	1.14	1.13	1.15
	21	1.11	1.21	1.19	1.19	1.20	1.21
235I	4	0.84	0.84	0.87	0.88	0.88	0.90
	21	0.83	0.86	0.89	0.90	0.90	0.90

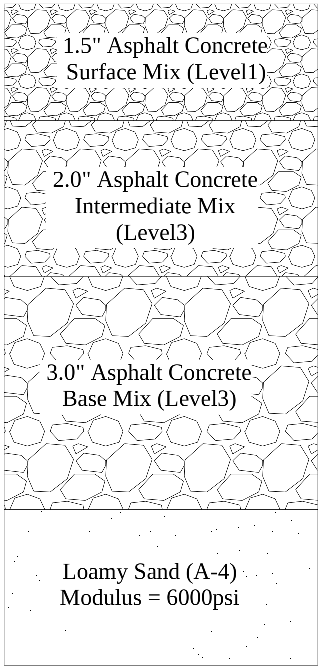
1.5" Asphalt Concrete
Surface Mix (Level1)
2.0" Asphalt Concrete
Intermediate Mix
(Level3)
3.0" Asphalt Concrete
Base Mix (Level3)
Loamy Sand (A-4)
Modulus = 6000psi

Figure 2. Pavement cross section

Figure 3 through Figure 7 present the results from the MEPDG simulations. The results of the analysis show the difference between the three asphalt concrete mixes. In the case of the Rose design, which is expected to be a moisture-susceptible mix using the E* ratio results, all distresses increased significantly. In the case of the 235I, there is also an increase in the predicted distresses, but the increase is lower than the predicted for Rose. For 235S, because of the E* ratio that is higher than one, there was a slight decrease in the predicted distresses. Although the Rose mix is ranked as the most moisture-susceptible mix and is predicted to have a higher increase in distresses with moisture conditioning, its overall performance is good and is better than 235I mix. This emphasizes the role of the MEPDG evaluation in quantifying the damage caused by moisture.

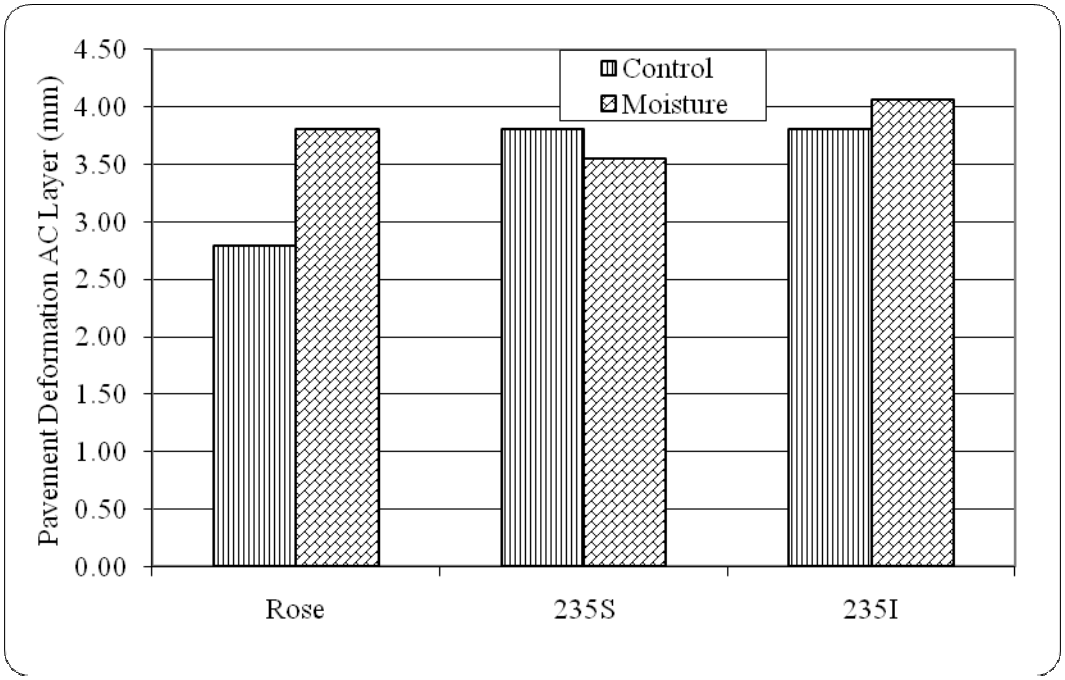

Figure 3. Permanent deformation in the AC layer

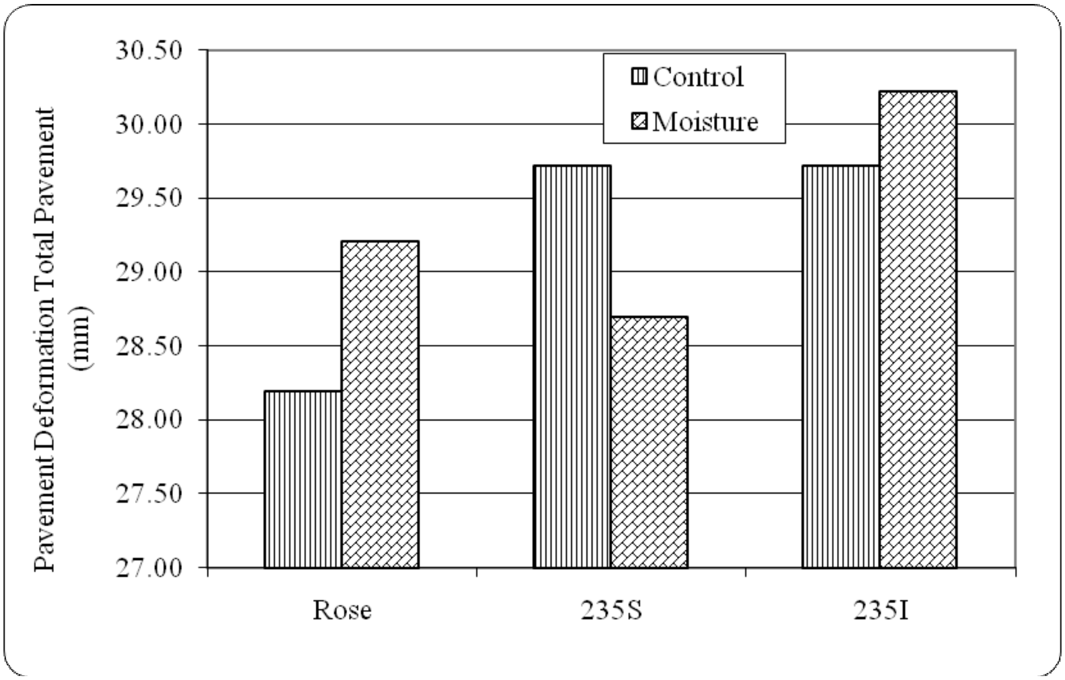

Figure 4. Permanent deformation in total pavement structure

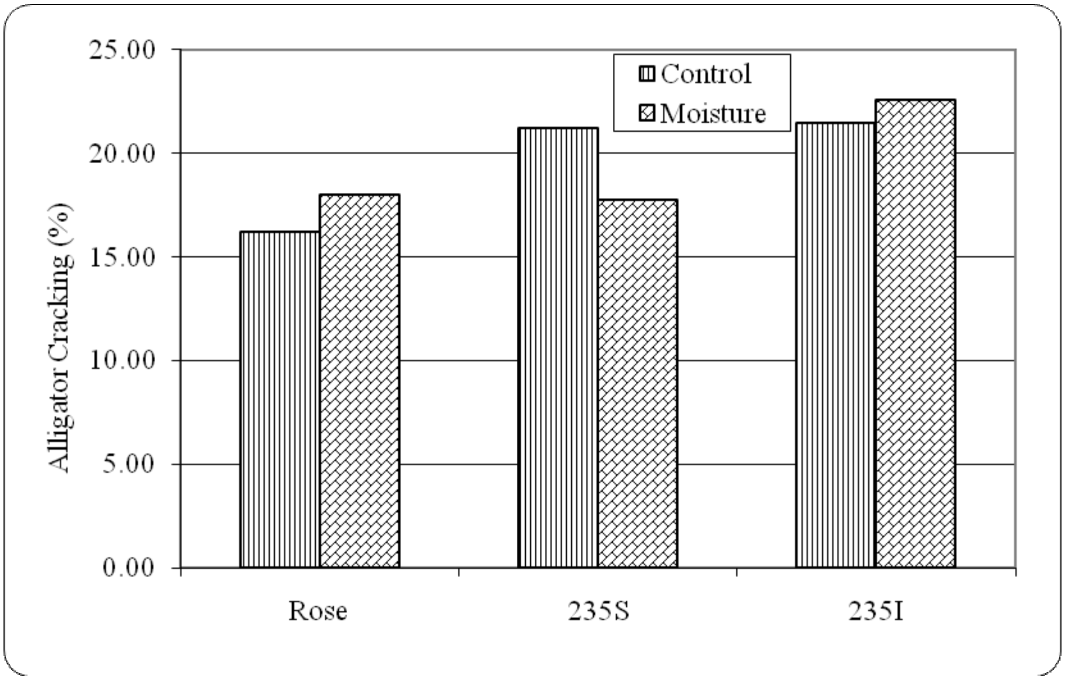

Figure 5. Alligator cracking

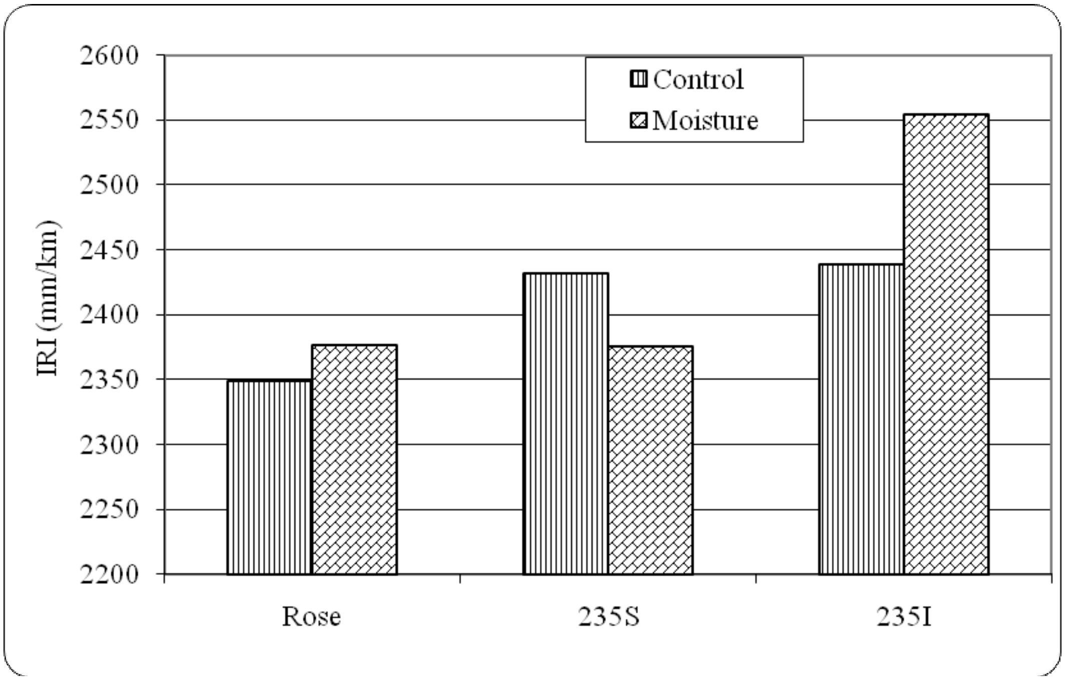

Figure 6. International Roughness Index (IRI)

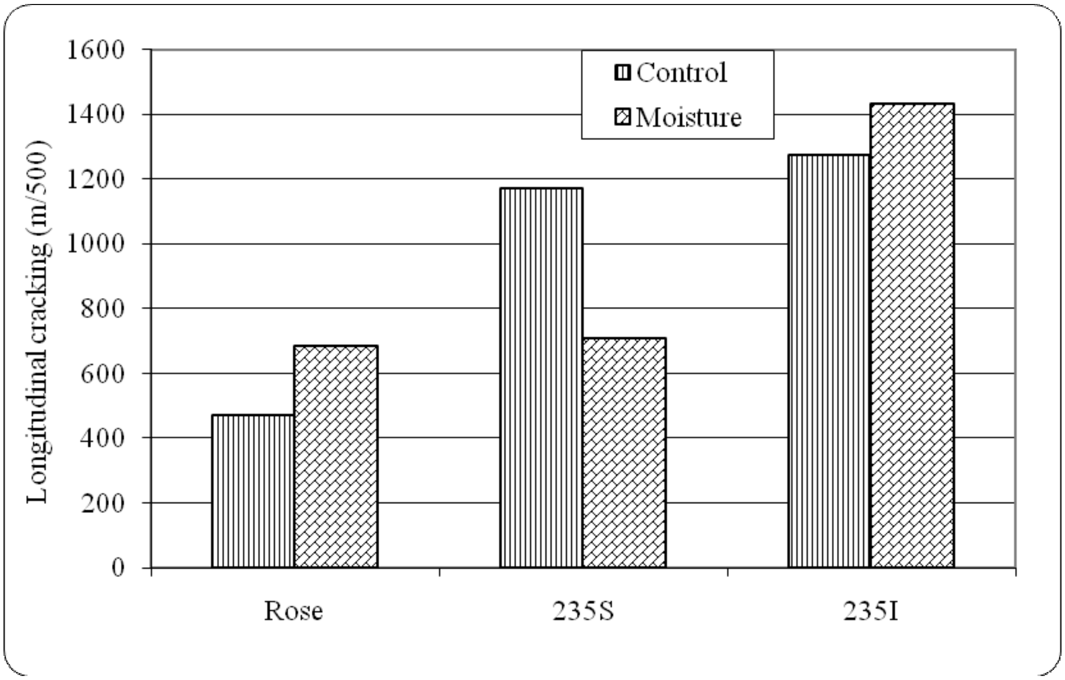

Figure 7. Longitudinal cracking

CONCLUSIONS

This paper has presented the possibility of using the MEPDG in simulating the difference of performance of asphalt pavement when moisture is present. This, in part, allows for the evaluation of this environmental effect in the MEPDG. This approach evaluates only the major distresses simulated in the design guide. Moisture damage can cause other problems in the pavement, such as raveling and stripping, and this needs to be further investigated. This paper presents the approach, and further validation is needed by simulating more test sections and comparing the results to field performance.

Based on the results of the experimental work and the output of the MEPDG simulations for the three sections investigated, the following can be concluded:

- Rose asphalt-concrete mixture is expected to be a more moisture-susceptible concrete mixture based upon the E* ratio.
- For all the mixes, E* ratios are directly proportional to the loading frequency and inversely proportional to the test temperature.
- Combining the dynamic modulus testing with the MEPDG offers a tool to predict the effect of moisture damage on pavement performance. Further calibration of the design guide equations might be needed based on the field validation of the results.
- The MEPDG results showed a clear distinction between the mixes, but further evaluation of other mixes and comparison with field data is needed.
- E* ratio for all frequencies shows only whether the mix is moisture susceptible or not. Utilizing the dynamic modulus test results in the MEPDG shows the effect of moisture damage on pavement performance from a pavement distress prospective. This can be a good approach in judging whether the increase in distresses due to moisture susceptibility will exceed the allowed design limits or may increase within the user-prescribed design limits.

ACKNOWLEDGMENTS

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Advancing Driver Safety through Simulator Research

Timothy L. Brown
National Advanced Driving Simulator
University of Iowa
2401 Oakdale Blvd.
Iowa City, IA 52242-5003
tbrown@nads-sc.uiowa.edu

Jane Moeckli
National Advanced Driving Simulator
University of Iowa
2401 Oakdale Blvd.
Iowa City, IA 52242-5003
jane-moeckli@uiowa.edu

ABSTRACT

Over the last thirty years, there have been between 40,000 and 52,000 fatalities per year on our nation's roadways. The National Advanced Driving Simulator (NADS) at the University of Iowa was designed and built to help researchers and regulators explore the root causes of motor vehicle crashes in order to help reduce roadway fatalities. To this end, staff and faculty at the University of Iowa in collaboration with researchers from around the country have been and continue to be involved in a number of cutting-edge research projects that positively impact road safety. Recent research into the effectiveness of electronic stability control (ESC) systems on behalf of the National Highway Traffic Safety Administration (NHTSA) provided valuable information to federal regulators who have subsequently mandated ESC on all light vehicles by model year 2012. NADS has continued this line of research and is now involved in assessing the effectiveness of stability control in heavy trucks. Other ongoing research efforts include the development of algorithms to detect alcohol-related impairment based on driving performance, the relationship between obstructive sleep apnea and crash risk, aging drivers, novice drivers, advanced vehicle safety system evaluation, and the impact of visual decrements on driving safety. NADS also continues to push the state of the art in driving simulation through such new technologies as the NADS MiniSim, which allows greater access of simulators to researchers through a low-cost portable simulation platform. Use of the MiniSim could allow for advances in roadway design through interactive visualization of various roadway designs and potential design optimization. Additionally, NADS researchers are in the process of developing a virtual Iowa City driving environment that will allow parallel simulator and on-road data collections using the same driving environments. This technique has applicability for easier visualization of existing road networks that could be used to evaluate current areas with unexplained high crash rates. An overview of these research and development programs will be provided.

Key words: electronic stability control—NADS—simulation

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Investigation of Warm-Mix Asphalt Performance Using the Mechanistic-Empirical Pavement Design Guide

Ashley Buss
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
174 Town Engineering
Ames, IA 50011
abuss@iastate.edu

R. Christopher Williams
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
482 Town Engineering
Ames, IA 50011
rwilliam@iastate.edu

Mohamed Rashwan
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
174 Town Engineering
Ames, IA 50011
rashwan@iastate.edu

Andrea Kvasnak
National Center for Asphalt Technology
Auburn University
277 Technology Parkway
Auburn, AL 36830
ank0004@auburn.edu

Tamer Breakah
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
174 Town Engineering
Ames, IA 50011
breakah@iastate.edu

ABSTRACT

Warm mix asphalt (WMA) has been on the horizon of new asphalt technologies, and now it is at the forefront of many research and field projects. The process of investigating the implementation of WMA is a task that many state and local agencies are now facing. The purpose of this paper is to describe WMA technologies and provide a summary of laboratory and field tests that have explored the use of WMA.

Several of the driving forces of WMA research are the potential for a reduction in energy, fuel consumption, and emissions as well as other benefits. The objective of this study is to compare the performance of the WMA additives DuraLife^{TM} and DuraClime^{TM} to hot mix asphalt (HMA) control mixtures. The comparison will be based upon dynamic modulus data that were collected at multiple temperatures and frequencies. The dynamic modulus data are input into the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) version 1.0, and the performance of the WMA mix is compared to the respective HMA control mixture. A statistical analysis was also performed on the dynamic modulus data.

The statistical analysis of the experimental data has shown differences in the values of the dynamic modulus between the HMA and WMA mixtures tested at various temperatures. Meanwhile, the results of the MEPDG showed WMA had a better MEPDG predicted pavement performance. The performance of WMA has thus far aligned well with the HMA, and research in this area is continuing.

Key words: aggregates—dynamic modulus—warm mix asphalt

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INTRODUCTION

Warm mix asphalt (WMA) has been on the horizon of new asphalt technologies and now it is at the forefront of many research and field projects. The process of investigating the implementation of WMA is a task that many state and local agencies are now faced with. The intent of this paper is to present information about WMA for the evaluation of WMA use in the state of Iowa. This paper will include a brief literature review, an experimental plan, results, and conclusions.

The literature review section will present information about WMA by describing the commonly used types of WMA and the benefits of WMA. The literature review will also provide information and background about the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) and dynamic modulus testing.

The objective of the experimental plan was to compare field WMA mixes to an HMA mix, where all materials and mix-design proportions were the same, except for the WMA additive. The field mixes were collected and sent to Iowa State University for testing. The mixes were reheated and compacted into samples, and then dynamic modulus testing was performed. The dynamic modulus values were input into the MEPDG, and each WMA mix's predicted pavement performance was compared to the predicted pavement performance of the corresponding HMA mix. Results show WMA having slightly better performance in the areas of rutting and fatigue based on the values predicted by the MEPDG.

LITERATURE REVIEW

Types of WMA

There are three main types of WMA technologies. These include foaming, organic wax additives, and chemical processes. Foaming technologies use small amounts of water in the binder to foam the binder, which lowers the viscosity. There are several foaming technologies available, such as Aspha-min®, WAM-Foam® developed by Shell Petroleum and Kolo-Veidekke, as well as the Astec Double Barrel Green® system (D'Angelo et al. 2008).

The next most commonly used WMA technology is the organic wax additive formed from a Fisher-Tropsch wax. These are created by the treatment of hot coal with steam in the presence of a catalyst. A commonly used organic wax additive is Sasobit® (Hurley 2006).

The third type of WMA technology that is commonly used is a chemical additive. The additive is used in the form of an emulsion and then mixed with hot aggregate. The typical mixing temperature ranges between 85°C and 116°C (Hodo, Kvasnak, and Brown 2009). The most commonly used chemical additive was Evotherm®. Other commonly used chemical additives are Evotherm DAT® and Evotherm 3G®.

The warm mix additives used in this study were DuraLife™ and DuraClime™. These technologies are manufactured by Lafarge. According to Lafarge, DuraClime™ asphalt employs WMA technology and is produced between 100°C and 135°C. This reduces the energy consumed and emissions produced. Tests have repeatedly demonstrated the superior technical performance of DuraClime™, as well as the environmental benefits (Lafarge 2009).

Benefits of Warm Mix Asphalt

The benefits of WMA are dependent upon which technology is utilized. There are varying degrees of benefits for each method. This is an overview of the benefits thus far realized by the industry, but the specific benefits for each technology, in some cases, are not entirely quantified. Some benefits may not yet be completely economically quantifiable, such as emission reduction. Also, the benefit may be a variable cost, such as the asphalt binder cost. If stricter emissions standards are implemented, there may be higher economic potential for WMA. The purpose of this section is to present the potential benefits of WMA. Since WMA technology is in the beginning stages of implementation, there are many questions about benefits that have not yet been answered.

Air Quality

The WMA technology reduces the asphalt's temperature at the time of paving, and there are several resulting benefits. These include an improved and cooler working environment, decreased exposure to asphalt fumes, higher employee retention, and an improved quality of work (Newcomb 2009). According to the National Institute for Occupational Safety and Health (NIOSH) website, the current recommended exposure limit (REL) for asphalt fumes is 5mg/m^3 as total particulate matter (TPM) during any 15 minute period (Roberts et al. 1996). The reduced temperatures of WMA will produce fewer fumes and create better paving environments in areas such as tunnels or underground paving (Kristjansdottir 2006, Brown 2008).

Environmental Protection Agency Regulations

As the country and the world move to become more sustainable, more requirements concerning pollution will be implemented. One example of a more stringent air pollution policy is the Clean Air Interstate Rule (CAIR). The CAIR will achieve the largest reduction in air pollution in more than a decade. CAIR emission standards apply to 28 eastern states (including Iowa), and achieving the required reductions is predominately focused on controlling emissions from power plants, but states are given the option to meet an individual state emissions budget through measures of the state's choosing. The Environmental Protection Agency (EPA) has shown that cap-and-trade systems have worked for other programs and will be used in the CAIR for both SO_2 and NO_x. Both SO_2 and NO_x are emissions created in the production of HMA. The EPA's website states the following about the CAIR cap-and-trade for SO_2 and NO_x (U.S. EPA 2009),

> EPA already allocated emission 'allowances' for SO_2 to sources subject to the Acid Rain Program. These allowances will be used in the CAIR model SO_2 trading program. For the model NO_x trading programs, EPA will provide emission 'allowances' for NO_x to each state, according to the state budget. The states will allocate those allowances to sources (or other entities), which can trade them. As a result, sources are able to choose from many compliance alternatives, including: installing pollution control equipment; switching fuels; or buying excess allowances from other sources that have reduced their emissions.

The asphalt industry, with WMA technology, would potentially be an example of a "source that has reduced their emissions" causing the asphalt industry to have "excess allowances" and would potentially be able to sell these to a non-compliant pollution source. This strategy would help put an economic value on the emission reductions seen in WMA. The CAIR will be completely implemented by 2015 (U.S. EPA

2009). Specifically for Iowa, the CAIR will reduce SO_2 emissions by 5% and NO_x emissions by 49% (U.S. EPA 2008).

WMA Paving Benefits

There are numerous paving benefits for WMA. Some of these include less compaction effort, longer haul distances, and better workability with high-recycled asphalt pavement (RAP) mixes. WMA has been shown in both field and laboratory studies to have similar or better compatibility than traditional HMA mixes. A laboratory study conducted at the National Center for Asphalt Technology (NCAT) compared three different WMA additives to traditional HMA. The additives used were Evotherm®, Sasobit®, and Aspha-min®. The study found that all three additives significantly aided in the compaction compared to the control mix with no WMA additive. It was also found that Evotherm® reduced the air-void content the most (Hurley 2006). On a project in Canada located on Autoroute 55 southeast of Drummondville, Aspha-min® zeolite was found to be a compaction aid in the field in comparison to a similar mix without zeolite (Davidson 2007). Another study was conducted using the Astec Double Barrel Green® System and found that the WMA foaming technology provided compaction effort similar to HMA mixes but at a lower temperature (Wielinski, Hand, and Rausch 2009).

Other paving benefits may include a lower cooling rate, crack sealant improvements (MeadWestvaco 2008), and lower temperature paving (D'Angelo et al. 2008).

Incorporating WMA with RAP Paving

Lower production temperatures for RAP mixes are a potential benefit of WMA. The viscosity-reducing properties of WMA additives, such as Sasobit® or Advera®, have been shown in numerous studies to enhance the workability of RAP mixes. The incorporation of higher RAP percentages could potentially save money because less-virgin aggregate and binder would need to be purchased. This cost savings would be variable due to the potential for high fluctuations in virgin binder prices (Tao and Mallick 2009).

In summary, WMA offers many benefits to the workers, contractors, citizens, and government agencies. The lower temperatures create a cooler and improved air quality work environment. The contractors may also benefit from fuel savings. Studies have shown that fuel savings can reach up to 30%. The lower temperatures reduce the amount of odor that the asphalt plants emit. There is an additional benefit because asphalt plants could potentially be placed in areas of non-attainment. This would create shorter haul distances in these areas.

Mechanistic-Empirical Pavement Design Guide

The objective of the MEPDG is “to provide the highway community with a state-of-the-practice for the design of new and rehabilitated pavement structures, based on mechanistic-empirical principles” (NCHRP 2004a). The MEPDG addresses deficiencies in the 1993 AASHTO pavement design guide that have evolved due to the way the guide was developed and the advancement that has occurred since the original AASHO road test was finished in 1960. The design process begins by knowing the site condition parameters (traffic, climate, subgrade, and existing pavement conditions in case of rehabilitation). The second step is to evaluate a proposed design using the MEPDG software to check for the susceptibility for different distresses, such as rutting, fatigue cracking, longitudinal cracking, transverse cracking, and roughness. The pavement designer then needs to adjust the design based on the results and evaluate the updated design. These iterations are repeated until an acceptable design is achieved (NCHRP 2004). The designer can choose for each input from three input levels depending on the amount of available information. The three levels can be defined as follows: level 1 is used when the input is obtained from direct testing, level 2 uses correlations to establish or determine the required input, and level 3 uses national or regional default values to define the input (NCHRP 2004).

Dynamic Modulus Test

Dynamic modulus is one of the oldest mechanistic tests to be used to measure the fundamental properties of asphalt concrete. Dynamic modulus testing has been studied since the early 1960s by Papazian (1962) and became a standard test in 1979 by the American Society for Testing and Materials (ASTM) under D 3497 ”Standard Test Method for Dynamic Modulus of Asphalt Concrete Mixtures” (ASTM 2003). A sinusoidal (haversine) compressive axial stress is applied to a test specimen, using the testing procedure for dynamic modulus. The testing procedure includes using various frequencies and temperatures to capture the linear visco-elastic properties of the asphalt concrete.

Dynamic modulus is a measure of the relative stiffness of a mix. Mixes that tend to have good rut resistance at high-service temperatures have a corresponding high stiffness. Although the trade-off is at intermediate temperatures, stiffer mixes are often more prone to cracking for thicker pavements (NCHRP 2004). For this reason, dynamic modulus testing is conducted at a range of test temperatures (i.e., -10 to 54.4°C) and frequencies (i.e., 0.1 to 25 Hz) to measure the linear visco-elastic properties of asphalt-concrete mixtures. The tested ranges of temperature and frequencies are used to develop a master curve for each mixture in order to exhibit the properties of the mixture over a range of temperatures and reduced frequencies.

EXPERIMENTAL APPROACH

The objective of this study is to compare the performance of the WMA additives DuraLife^{TM} and DuraClime^{TM} to HMA control mixtures. The comparison was based upon dynamic modulus data that were collected at multiple temperatures and frequencies. The dynamic modulus data was input into the MEPDG version 1.0, and the performance of the WMA mix will be compared to the respective HMA control mixture. A statistical analysis was also performed on the dynamic modulus data.

The mixes from this study were provided by Lafarge. There were a total of four field mixes: two control mixes and two WMA mixes. On the first day of production, the first HMA mix was produced (HMA 1), as well as a WMA mix that contained the WMA technology DuraClime^{TM}. The WMA additive was the only difference between HMA 1 and the DuraClime^{TM} mix. On the second day of production, the second HMA mix was produced (HMA 2), and the DuraLife^{TM} mix was produced. Again, there were no differences except for the WMA additive between the HMA 2 mix and the DuraLife^{TM} WMA mix. The mixes were collected and sent to Iowa, and the samples were later reheated and compacted into dynamic modulus samples. The reheat temperature for the WMA mixes was 100°C and 135°C for the HMA.

The following sections will discuss the data that were input into the MEPDG.

Dynamic Modulus Values

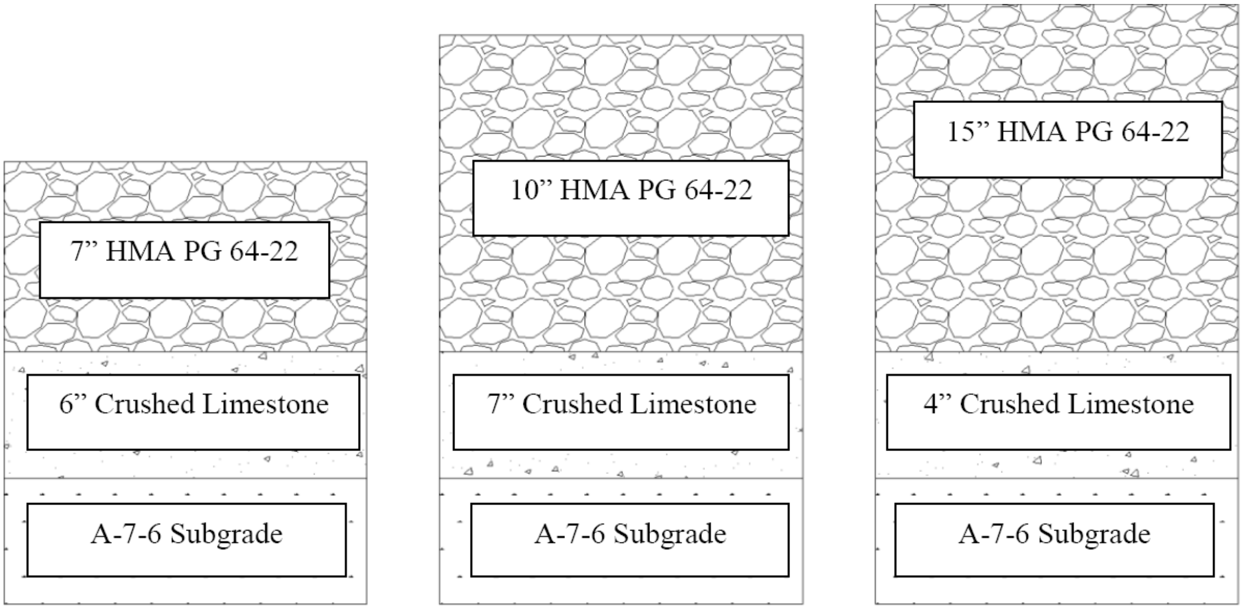
Dynamic modulus testing was performed on all of the samples to obtain the dynamic modulus value (E*) and measure the relative stiffness of the mix. The dynamic modulus tests were performed at 4, 21, and 37 °C and at the following frequencies: 25, 15, 10, 5, 3, 1, 0.5, 0.3, 0.1 Hz. The MEPDG requires E* values from five temperatures and six frequencies. Since the data from the dynamic modulus values had nine frequencies and only three temperatures, additional E* values had to be calculated based on shift factors generated from the master curves. The master curves were graphed using the dynamic modulus data. The master curves aided in calculating the shift factors for determining the E* values at different temperatures. It should be noted that this method does not extrapolate the data because E* is a function of frequency and temperature. The additional frequencies measured allowed for the calculation of the E* value at different temperatures using the relationship between E*, frequency, and temperature. The MEPDG input values for the E* values were obtained. The dynamic modulus values that were input into the MEPDG are shown in Figure 2.

Traffic Levels

The performance of the four mixtures investigated in this study was analyzed in three simulated road sections, mimicking different levels of traffic volumes: low, medium, and high. Table 1 provides the AADTT traffic inputs used in the MEPDG for each traffic volume. The structural design of the three roads with a design life of 20 years is illustrated in Figure 1.

Table 1. MEPDG traffic volumes

Traffic Level	**MEPDG Input Values**	
	AADTT	**Growth Rate (%)**
Low	100	2.01
Medium	696	1.17
High	2000	1.37



a. Low Volume Traffic b. Medium Volume Traffic c. High Volume Traffic

Figure 1. Pavement structure cross sections

Other MEPDG Inputs

With the exception of the input data for the asphalt concrete (AC) layers, all other parameters were entered into the MEPDG software at Level 3. The resilient modulus of the crushed stone, granular base layer was assumed to be 207 MPa, while resilient modulus for an A-7-6 subgrade was taken as 24 MPa. All other inputs for the Level 3 design were the same for all the mixes.

RESULTS

In this section, the dynamic modulus values for the various mixtures tested are discussed. Moreover, the performance of different road cross sections simulated using MEPDG is analyzed for the four mixes.

Dynamic Modulus Statistical Analysis

The dynamic modulus values at six different frequencies (0.1, 1, 5, 10, 15, and 25 Hz) were measured at three different temperatures (4°C, 21°C, and 37°C), as illustrated in Figure 2. A paired t-test was conducted to investigate the relationship between the dynamic modulus of the DuraClime^{TM} and DuraLife^{TM} mixtures and their corresponding mixtures. The significance level used was $\alpha = 0.05$. The paired t-test shows that the dynamic modulus data are statistically different when comparing the WMA mix to its respective HMA mix. The statistical data is presented in Table 2. The paired t-test results should be analyzed with caution because the experimental data may not reflect a perfectly normal distribution. According to the data, it does appear the addition of the WMA additive has an affect on the properties of the asphalt mixtures, and consequently, it affects the behavior of the mixtures under traffic load.

By showing statistically that WMA additives impacted the dynamic modulus values, their differences can now be discussed. The dynamic modulus values for the HMA mixes appear to be higher when compared to the WMA, as shown by the charts in Figure 2. This would indicate that the HMA is a stiffer mix. Stiffer mixes traditionally perform better at high temperatures. This makes comparing pavement distresses attributed to exposure to higher temperatures an important factor in the acceptance of WMA. For these reasons, the performance in rutting and fatigue were compared. At low temperatures, WMA is expected to perform similarly to HMA mixes. Creep-compliance data for each mix should be collected and input into the MEPDG to investigate the thermal cracking performances and to help confirm adequate WMA mix performance.

Table 2. Statistical analysis data

Mixes Compared in T-Test	Statistical Analysis Data From SAS Version 9.2				
	Mean Difference Between Mixes	Standard Deviation	Standard Error	t-Value	p-Value
HMA1-DuraClime^{TM}	388607.44	471154.31	111052.14	3.5	0.0027
HMA2-DuraLife^{TM}	2692295.72	1825533.43	430282.36	6.26	<.0001

Fatigue Performance

The analysis of the fatigue performance of the two WMA mixtures and their corresponding controls was conducted using MEPDG version 1.0 software. The analysis was performed on the three road sections with the distinctly different traffic volumes.

Figure 3 illustrates the performance of the four mixtures with respect to fatigue, with all the mixtures demonstrating alligator cracking well below the maximum critical value set in this study (25%). The WMA mixtures DuraClime^{TM} and DuraLife^{TM} have exhibited slightly better resistance to fatigue cracking in comparison with their control mixtures HMA 1 and HMA 2, respectively. This trend is common across the range of traffic levels adopted in this study.

Rutting Performance

The rutting values predicted in the asphalt-concrete layer of all four mixtures were well below the maximum critical value of 6.35 mm. At a given road section, the DuraClime^{TM} and the DuraLife^{TM} mixtures performed better than their respective control mixtures in terms of amount of permanent deformation taking place in the asphalt layer. For example, the rut depth predicted for the medium traffic road constructed with DuraLife^{TM} was 2 mm, while the rut depth for HMA 2 for the same road was 3.30 mm.

Moreover, the rutting values predicted for the total pavement cross section of all mix groups was below the failure criterion set for this study (19 mm), except for the HMA 2 mixture utilized in the medium traffic road. This pavement experienced a predicted permanent deformation of 19.5 mm. The trend of slight superiority of the WMA mixtures DuraClime^{TM} and DuraLife^{TM} with respect to rutting is captured in Figures 4 and 5.

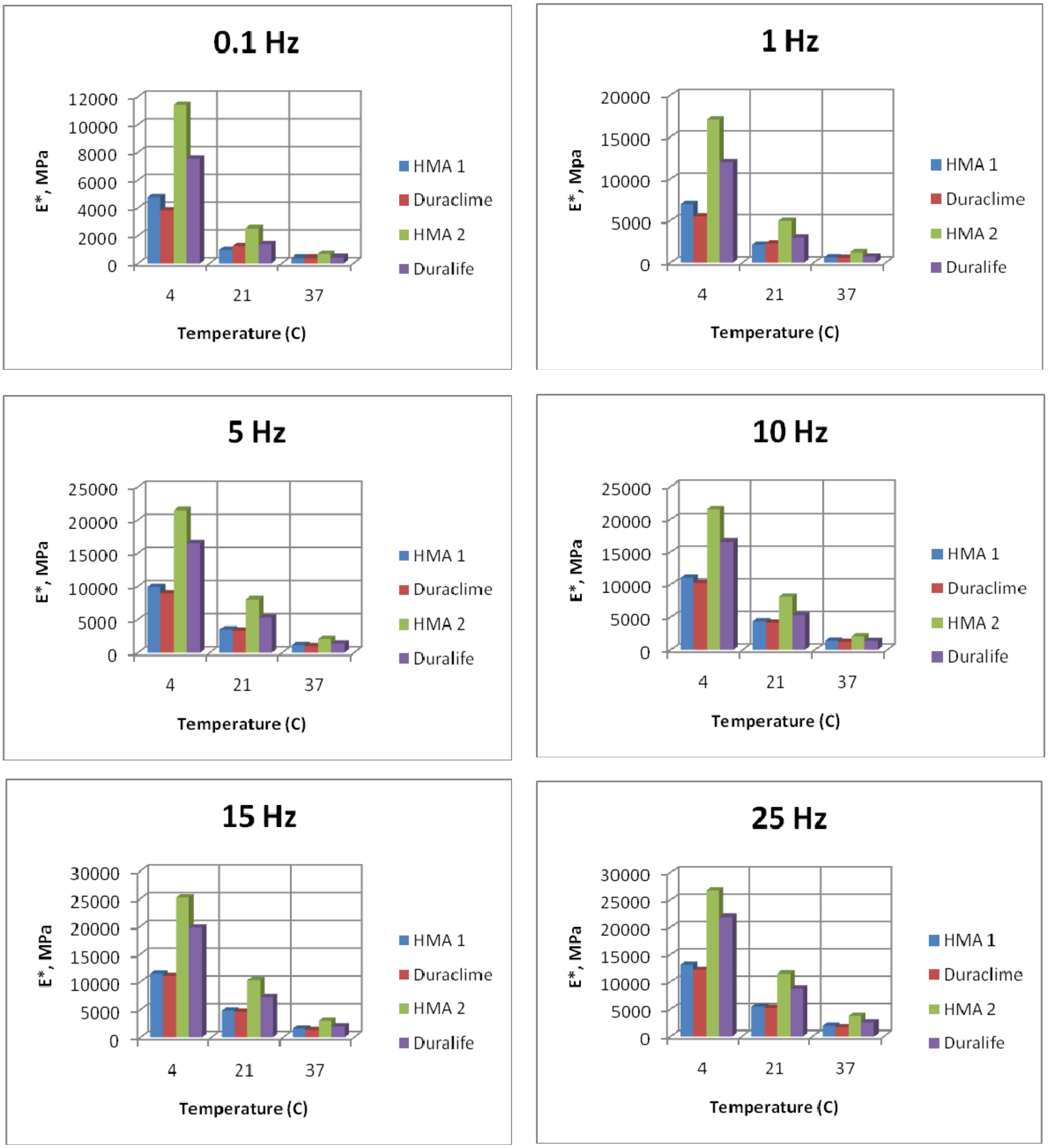


Figure 2. Dynamic modulus data

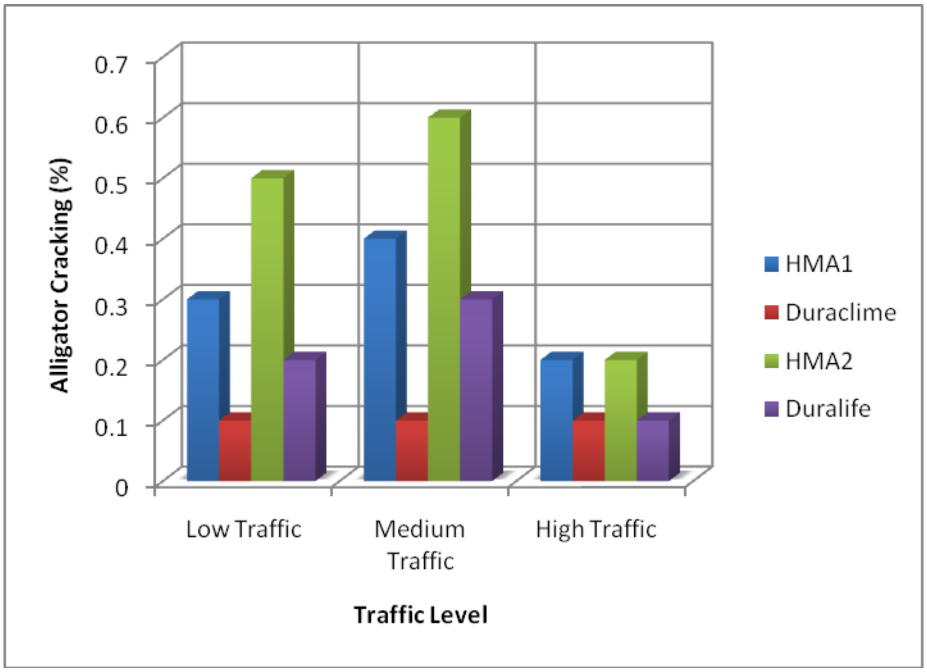


Figure 3. Comparison of alligator cracking and mix performance by traffic level

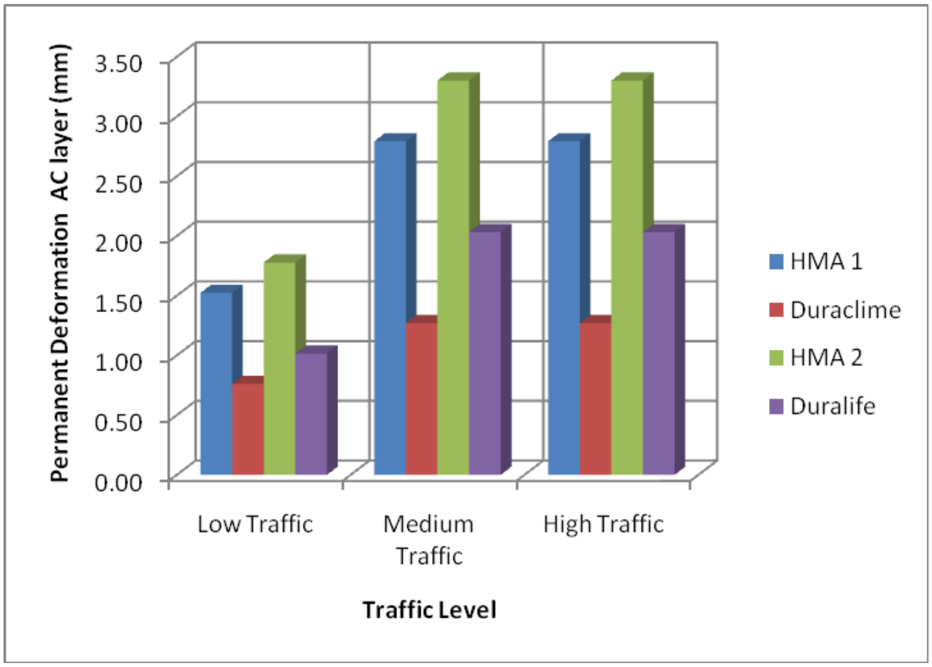


Figure 4. Asphalt concrete permanent deformation

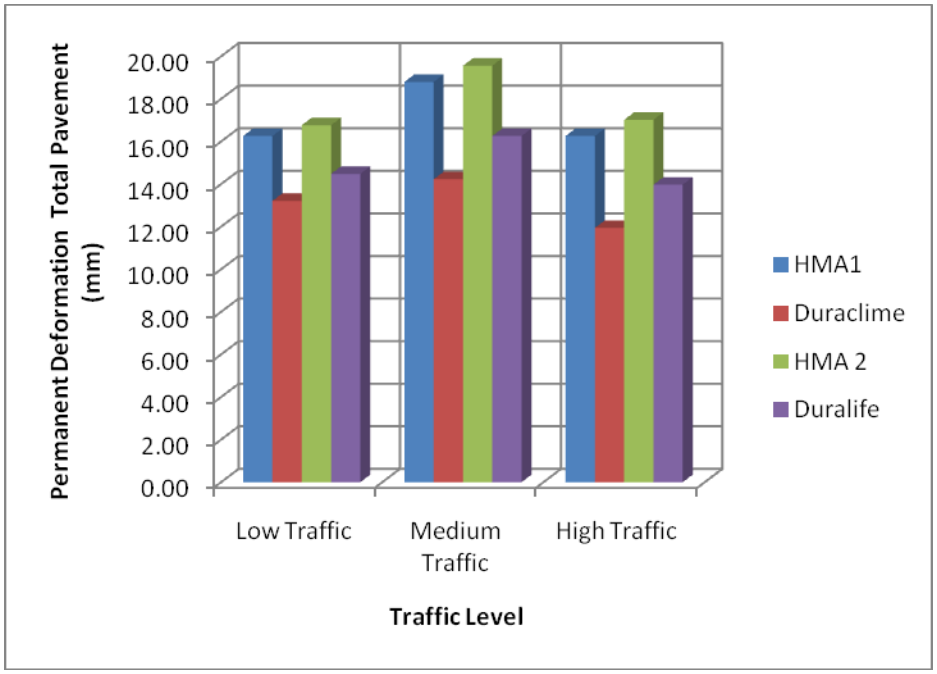


Figure 5. Pavement permanent deformation

CONCLUSIONS AND RECOMMENDATIONS

This paper compares the MEPDG predicted performance of two WMA pavements against conventional control mixtures at three roads with different traffic volumes. The performance of the four mixtures is assessed in terms of two main types of distresses: fatigue and rutting.

The behavior of the mixtures was similar with the WMA mixtures, recording slightly better results in comparison to their control mixtures with the WMA and having the additional benefit of making the asphalt pavements more sustainable. In addition, the production of WMA pavements generates fewer emissions and consumes less energy and fuel in comparison to conventional HMA pavements, as discussed in the WMA benefits section.

In conclusion, based on the E* data and the output of the MEPDG software, the performance of the WMA pavements investigated is equal or slightly better than the performance of similar HMA pavements. However, further research is needed to investigate the impact of low-temperature cracking on WMA pavements, as well as the performance of other types of WMA technologies.

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Crash Experience—What Makes Low-Volume Roads Different and What Can Be Done About It

Mehmet Caputcu
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
mcaputcu@iastate.edu

Reginald Souleyrette
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
reg@iastate.edu

Thomas McDonald
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
tmcdonal@iastate.edu

Robert Sperry
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
rsperry@iastate.edu

ABSTRACT

Emphasis given to safety on roads in recent years has led to the rise of the concept “safety culture,” which can briefly be defined as promoting safe driving and riding in the common sense within the community. However, low-volume rural roads have been paid less attention than they actually deserve in this context. It has been determined that very low-volume rural roads have relatively higher crash rates than high-speed freeways and other high-volume highways in the state of Iowa. Besides, a higher crash rate than many other states has been observed on secondary low-volume rural roads of Iowa. Since traffic crash modeling is a helpful tool for assessing risk factors and design issues in roadway travel, the aforementioned roads were part of a research project to create a system-level generalized model in order to investigate the underlying reasons. By utilizing the statewide crash data, this model attempted to find the trends in frequency, rate, and severity with respect to crash, driver, and/or roadway variables that would be the best predictors of crashes. Before running the statewide model, candidate sites were identified to establish a better, more comprehensive understanding of crash factors and circumstances. After gaining a clearer understanding of the factors contributing to the severity of crashes by looking at these candidate sites, a further study was done at the driver/vehicle level. The candidate sites were determined based on a decision considering crash rates and densities by counties, particular high-crash locations, and other characteristics such as larger growth rate and very low population density. Video analysis was also utilized and cross-comparison tables were prepared to examine the proportions of various crash characteristics across roads with different volumes and surface type.

Key words: crash rate—low-volume rural roads—safety culture

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Iowa DOT’s Pavement Management Information System (PMIS) and the Mechanistic-Empirical Pavement Design Guide

Halil Ceylan
Department of Civil, Construction, and Environmental Engineering
Iowa State University
482B Town Engineering
Ames, IA 50011-3232
hceylan@iastate.edu

Kasthurirangan Gopalakrishnan
Department of Civil, Construction, and Environmental Engineering
Iowa State University
354 Town Engineering
Ames, IA 50011-3232
rangan@iastate.edu

Sunghwan Kim
Department of Civil, Construction, and Environmental Engineering
Iowa State University
192 Town Engineering
Ames, IA 50011-3232
sunghwan@iastate.edu

Omar Smadi
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
smadi@iastate.edu

Chris Brakke
Office of Design
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
chris.brakke@dot.iowa.gov

Fereidoon (Ben) Behnami
Office of Design
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Ben.Behnami@dot.iowa.gov

EXTENDED ABSTRACT

Reliable and cost-effective design of a rehabilitation project requires the collection and detailed analysis of key data from the existing pavement. The first step in the pavement rehabilitation selection process involves assessing the overall condition of the existing pavement and fully defining the existing pavement problems.

In 2004, the National Cooperative Highway Research Program (NCHRP) released a new pavement design guide called the *Mechanistic-Empirical Pavement Design Guide* (MEPDG). The MEPDG is a design guide for not only new pavement sections but also for rehabilitated pavement systems to enhance and improve pavement design for many state highway agencies. MEPDG rehabilitation analysis and design requires not only input parameters identical to those used for new pavement design but also additional input parameters related to the existing condition of the pavement systems.

Information on many of the factors related to the existing pavement condition can be obtained from the Iowa Department of Transportation’s (Iowa DOT’s) existing Pavement Management Information System (PMIS); however, depending on how regularly data are collected and how recent the latest data are, there may be a need to supplement the pavement management data with a more up-to-date field survey and testing data.

Key words: Mechanistic-Empirical Pavement Design Guide (MEPDG)—pavement design—pavement management—rehabilitation

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The study aims to systematically evaluate the Iowa DOT's existing PMIS with respect to the input information required for MEPDG rehabilitation analysis and design. All of the available PMIS data for interstate and primary roads in Iowa were retrieved from the Iowa DOT's PMIS. The retrieved data were evaluated with respect to the input requirements and outputs for the latest version of the MEPDG software system (version 1.0).

The Iowa PMIS database contains more than 3,000 data records for each year, corresponding to segments of Interstate and primary roads in Iowa. These data records include detailed information for hot mix asphalt (HMA), jointed plain concrete (JPC), continuously reinforced concrete (CRC), and composite pavement systems. Each data record consists of lots of information, including traffic volume, pavement material and structure, and distress survey results, which add up to about 270 columns when the database is formatted in an Excel Spreadsheet. However, Iowa DOT's PMIS does not have the detailed material property inputs (e.g., subgrade resilient modulus, HMA dynamic modulus, portland cement concrete [PCC] elastic modulus, etc.) and the detailed traffic characterization inputs (e.g., vehicle class distribution, hourly traffic distribution, etc). The available information from Iowa DOT's PMIS was also compared to the rehabilitation-related input information required for running the latest version of the MEPDG software (version 1.0). These comparisons for HMA and PCC rehabilitation designs are summarized in Table 1. Only four among nine input parameters of MEPDG HMA rehabilitation design and only three among seven input parameters of MEPDG PCC rehabilitation design are available in the current Iowa DOT PMIS. These results indicate that the Iowa DOT PMIS should be revised/updated to incorporate periodically collected data for the identified unavailable parameters.

Pavement distress types and the units for each distress type collected from distress surveys are recorded in Iowa DOT's PMIS were compared to those of MEPDG pavement performance predictions (see Table 2). In general, it was found that most of the MEPDG performance measures are also available in Iowa DOT's PMIS. However, three performance measures for CRCP such as punch-out, maximum crack width, and minimum crack load transfer efficiency (LTE) are not available in Iowa DOT's PMIS. Also, the measurement units for JPCP transverse cracking as well as HMA alligator and thermal (transverse) cracking reported by MEPDG cannot be compared with that of Iowa DOT's PMIS. These results indicate that the proper conversion methods of pavement distress measurement units from PMIS to MEPDG should be developed for the future local calibration of MEPDG under Iowa conditions. Iowa DOT's PMIS provides only accumulated (total) surface rutting observed on the pavement surface, while MEPDG provides rutting predictions for individual pavement layers. This can lead to difficulties in the local calibration of MEPDG rutting models for corresponding pavement layers. The PMIS data are reported in SI units, whereas English units are used in MEPDG, although this is not a big concern.

Table 1. Input requirements for MEPDG HMA and PCC rehabilitation design

Type of rehabilitation		Input variable available in Iowa PMIS	Input variable not available in Iowa PMIS
HMA rehabilitation	Rehabilitation for existing PCC pavement	• Modulus of subgrade reaction	• Before restoration, percent slabs with transverse cracks plus percent previously replaced/repaired slabs • After restoration, total percent replaced/repaired slabs • CRCP punch out (per mile) • Monthly modulus of subgrade reaction measured
	Rehabilitation for existing HMA pavement	• Total rutting • Milled thickness • Existing pavement condition	• Placement of geotextile prior to overlay
PCC rehabilitation	Rehabilitation for existing PCC or HMA pavement	• Modulus of subgrade reaction • Milled thickness • Existing pavement condition	• Before restoration, percent slabs with transverse cracks plus percent previously replaced/repaired slabs • After restoration, total percent replaced/repaired slabs • CRCP Punchout (per mile) • Monthly modulus of subgrade reaction measured

Table 2. Comparison of MEPDG and Iowa PMIS outputs and measurement units

Type of Pavement		Performance Model	MEPDG	Iowa PMIS
HMA		Longitudinal cracking	ft/mile	m/km
		Alligator cracking	%/total lane area	m^2/km
		Thermal cracking	ft/mile	m^2/km
		Rutting	In.	m
		Smoothness	in./mile	m/km
PCC	JPCP	Faulting	in.	m
		Transverse cracking	% slab cracked	number/ km
		Smoothness	in./mile	m/km
	CRCP	*Punch-out*	number/mile	N/A[a]
		Maximum crack width	mils	N/A[a]
		Minimum crack LTE	%	N/A[a]
		Smoothness	in./mile	m/km

a. N/A = Not Available

Based on the results of this study, it is recommended that the Iowa DOT’s PMIS should be updated, if possible, to include the identified parameters that are currently unavailable but are required for MEPDG rehabilitation design. Similarly, the measurement units of distress survey results in Iowa DOT’s PMIS should be revised to correspond to those of MEPDG performance predictions.

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Use of Bio-Oil for Pavement Subgrade Soil Stabilization

Halil Ceylan
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
394 Town Engineering
Ames, IA 50011-3232
hceylan@iastate.edu

Sunghwan Kim
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
192 Town Engineering
Ames, IA 50011-3232
sunghwan@iastate.edu

Kasthurirangan Gopalakrishnan
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
354 Town Engineering
Ames, IA 50011-3232
rangan@iastate.edu

EXTENDED ABSTRACT

Use of biofuel may partially offset energy requirements currently fulfilled by fossil fuels (Paustian et al. 1998). Corn stover and other plant materials with a high concentration of cellulose have potential as biofuel or in ethanol production (Johnson et al. 2004). The co-product during ethanol/biofuel production from biomass contains lignin, modified lignin, and lignin derivatives. Traditional uses for lignin and modified lignin include concrete admixtures, binders, well-drilling mud, dust control, vanillin production, and dispersants (Kamm and Kamm 2004). Lignin has also been implicated in soil stabilization, which improves the properties of less-desirable road soils and provides a suitable foundation for paved or unpaved roads (Nicholls and Davidson 1958; Tingle and Santoni 2003). However, most lignin and modified lignin in use is derived from the paper mill industry. The utilization of lignin derived from biofuel production needs to be investigated to provide additional revenue streams to improve the economics of the bioconversion facilities.

This study investigates the utilization of lignin-containing biofuel co-product in soil stabilization. The experimental plan was developed for unconfined compressive strength (UCS) testing of two categories of treatment types: (1) untreated soil sample (control) and (2) soil sample treated with lignin-containing biofuel co-products.

The natural soil used in this study conforms to class 10 soil as described in the Iowa Department of Transportation (Iowa DOT) specification (Iowa DOT 2008). The Iowa class 10 soil in this study could be classified as CL and A-6(8) in accordance with the Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASHTO) soil classification system, respectively. Commercially available bio-oil (DESC 2007) was used as the experimental lignin-containing biofuel co-product for this study.

Soil was mixed with additive (lignin/bio-oil) at varying amounts to identify the optimal additive content for strength. The additive contents evaluated are 1%, 3%, 6%, 12%, and 15% by dry soil weight. The untreated soils were also tested without the addition of any co-products. Similarly, the moisture contents

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and curing periods were incorporated into the experimental test plan. The levels of moisture content used in testing the samples were optimum moisture content (OMC), OMC+4%, and OMC-4% of untreated soil. Curing periods investigated were one and seven days after sample fabrication for strength tests.

The results are shown graphically in Figures 1–3. The strength values at 0% additive content on these figures indicate those of untreated soil after one and seven days of curing. The strengths of untreated soils are, in all cases, lower than those of additive-treated soils. Overall, the soil strengths in the drier side are higher than those in the wetter side. A high increase in the UCS occurs with 12% of co-product in all cases. Within the scope of this study, bio-oil as the lignin-containing biofuel co-product was found to be effective in stabilizing the Iowa class 10 soil classified as CL or A-6(8). Utilization of biofuel co-product as a soil stabilization agent appears to be one of many viable answers to the profitability of the bio-based products and the bioenergy industry, especially in and around Iowa. Since there is much more biofuel co-product that is disposed of rather than utilized, making more productive use of biofuel co-product would have considerable benefits for sustainable development.

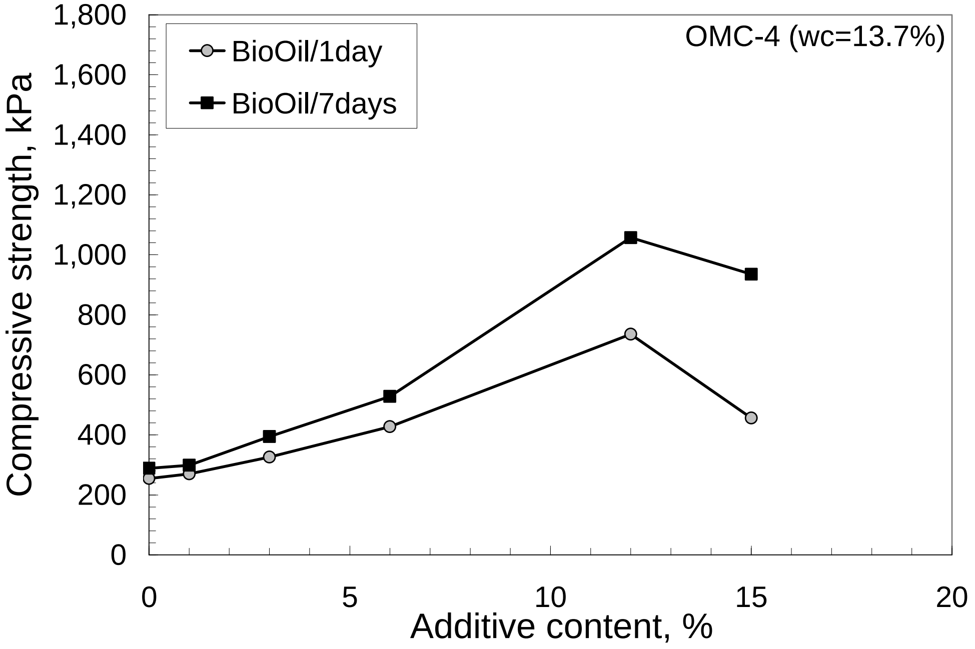

Figure 1. Variation of strengths under OMC-4% condition

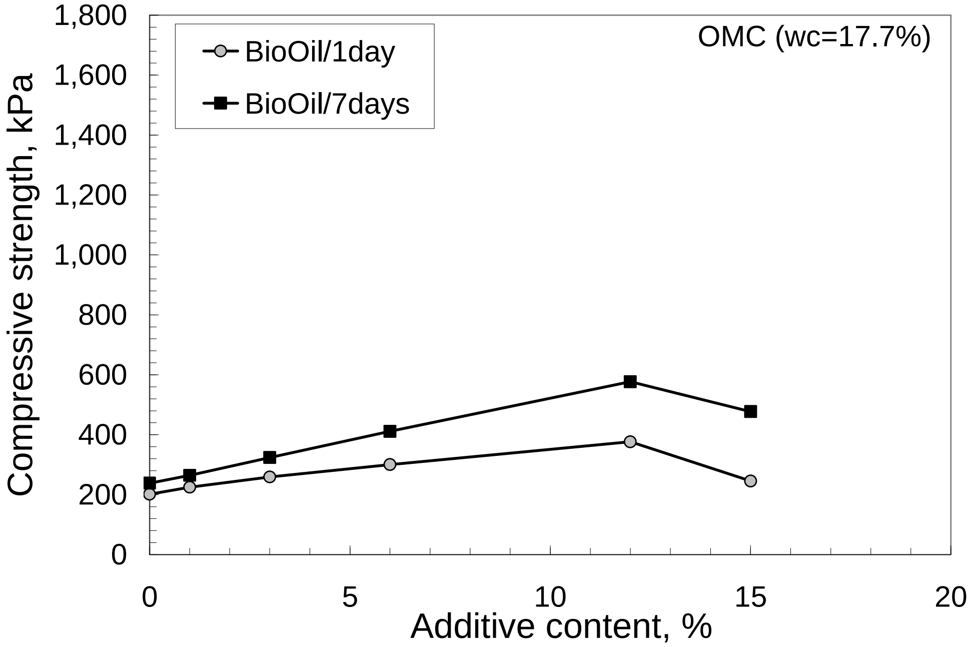

Figure 2. Variation of strengths under OMC condition

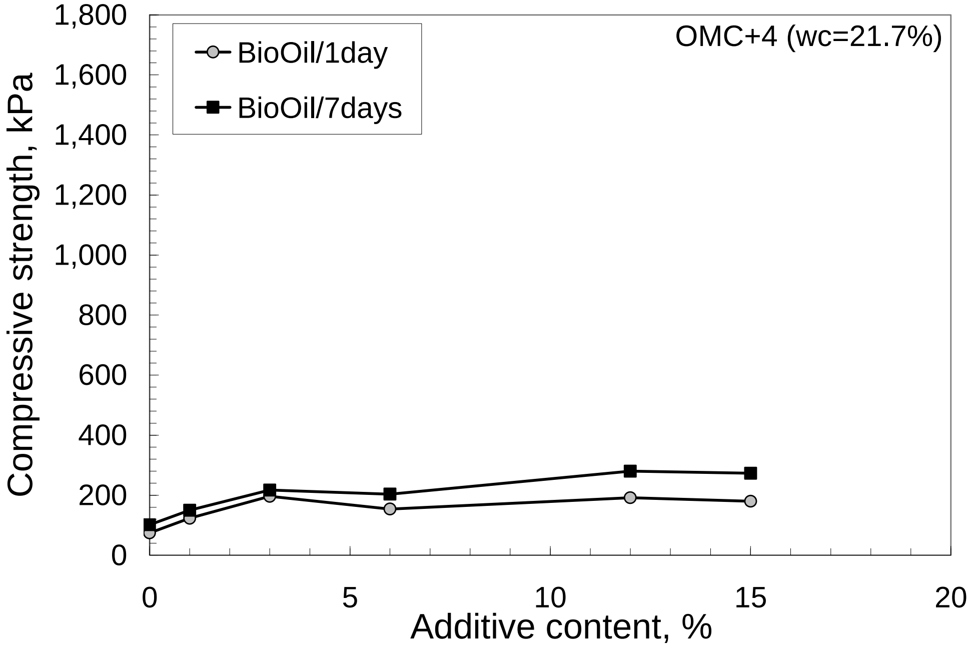

Figure 3. Variation of strengths under OMC+4% condition

Key words: biofuel—bio-oil—lignin—pavement system—soil stabilization—unconfined compressive strength

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Drivers’ Preferences for LED Vehicular and Pedestrian Signals

Madhav V. Chitturi
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
B243 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
mchitturi@wisc.edu

Ali Hajbabaie
Department of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
Room 3150 NCEL
205 N. Mathews Ave.
Urbana, IL 61801
ahajbab2@illinois.edu

Rahim F. Benekohal
Department of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
1213 Newmark Civil Engineering Laboratory
205 N. Mathews Ave.
Urbana, IL 61801
rbenekoh@illinois.edu

Juan C. Medina
Department of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
Room B106 NCEL
205 N. Mathews Ave.
Urbana, IL 61801
jcmedina@illinois.edu

ABSTRACT

In recent years, the Light Emitting Diodes (LEDs) traffic signal modules have been replacing the incandescent lamp modules in the United States. Approximately 260,000 signalized intersections exist in the United States and each intersection would have a minimum of 24 and on average 40 signal indicators. The incandescent lamps use 69, 135, or 150 watts of energy. The incandescent lamps are inexpensive; however, they consume a lot of power and require annual preventative lamp replacement. On the other hand, LED signal modules are more expensive but consume a fraction of the power (around 10 watts) consumed by incandescent lamps. Furthermore, LED modules, in general, are guaranteed by the manufacturers to last for five years. The Institute of Transportation Engineers (ITE) has developed specifications for traffic signal modules. However, there has been no research on the drivers’ preferences for LED traffic signal modules; specifically, what characteristics of the LED traffic signal modules are important for the drivers and their relative importance. For instance, uniformity of appearance is required by the current ITE specifications; however, there has been no research to evaluate if drivers consider uniformity or the lack of it as important for their comprehension of the traffic signal modules. This research presents the results from a survey of 120 drivers’ preferences for the LED traffic signal modules.

Key words: Light Emitting Diodes—signalized intersections—traffic signal modules

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Identification of Contributory Factors for Cross-Median Crashes

Madhav V. Chitturi
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
B243 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
mchitturi@wisc.edu

David A. Noyce
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1204 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
noyce@engr.wisc.edu

Ghazan Khan
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
B243 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
gkhan@wisc.edu

ABSTRACT

Crossover median crashes are a concern for transportation officials across the country. The nature of a crossover crash—a vehicle that traverses a median and collides with another vehicle either head-on or side-swipe—creates a situation that is high cost, both financially and in terms of human injury. In Wisconsin, median barriers are installed on highways that meet a certain median width and average daily traffic (ADT) requirement. Under these requirements, highway segments with a speed limit greater than 55 mph are not required to install median barriers with a median width greater than 60 ft or under specific ADT conditions for median widths of less than 60 ft. Nevertheless, many crossover crashes are observed on highway segments that do not meet the current warrants for median barrier protection. Therefore, there is a pressing need to develop alternate warrants for median barriers that are not based exclusively on median width and traffic volume. This study identified cross-median crashes that occurred in Wisconsin from 2001 to 2007 and used geospatial techniques to identify factors that contribute to cross-median crashes.

Key words: crossover median crash—median barriers

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Trajectory Analysis of Vehicles Approaching Speed Feedback Signs

Madhav V. Chitturi
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
B 243 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
mchitturi@wisc.edu

David A. Noyce
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1204 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
noyce@engr.wisc.edu

Kelvin Santiago
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1241 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
ksantiago@wisc.edu

ABSTRACT

Speed feedback signs (SFSs) are being increasingly used as a speed control strategy. Several studies have been performed to evaluate the effect of SFSs on reducing the speeds of drivers. However, all of the previous studies determined the effect of the SFS on point speeds (speeds measured at a point). None of the studies have examined the driver response to SFS and their effects on safety, particularly the possibility of rear-end crashes. The Wisconsin Department of Transportation (WisDOT) has installed four SFSs on a stretch of State Trunk Highway (STH) 164 in Washington County in Wisconsin to reduce the speeds of drivers. The research team used radar devices to collect the trajectory data of individual vehicles as they approached and passed the SFS. This paper will present the results of "Trajectory Analysis of Vehicles Approaching Speed Feedback Signs."

Key words: speed feedback signs—trajectory—Wisconsin

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Development of Predictive Median Barrier Warrants

Madhav V. Chitturi
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
B 243 Engineering Hall
1415 Engineering Drive
Madison, WI 53706
mchitturi@wisc.edu

David A. Noyce
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1204 Engineering Hall
1415 Engineering Dr
Madison, WI 53706
noyce@engr.wisc.edu

Andrew Ooms
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1241 Engineering Hall
1415 Engineering Dr
Madison, WI 53706
ooms@wisc.edu

ABSTRACT

Crossover median crashes are a concern for transportation officials across the country. The nature of a crossover crash—a vehicle that traverses a median and collides with another vehicle either head-on or side-swipe—creates a situation that is high cost, both financially and in terms of human injury. In Wisconsin, median barriers are installed on highways that meet a certain median width and average daily traffic (ADT) requirement. Under these requirements, highway segments with a speed limit greater than 55 mph are not required to install median barrier with a median width greater than 60 ft or under specific ADT conditions for median widths of less than 60 ft. Nevertheless, many crossover crashes occurred on highway segments that do not meet the current warrants for median barrier protection. This study developed a framework for developing predictive median barrier warrants and involved the development of crash frequency and severity prediction models for crossover median and median barrier crashes, as well as determining cost and benefit information.

Key words: crossover median crash—median barrier—predictive median barrier warrants

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Iowa's Floods of 2008 and the Iowa DOT's use of BridgeWatch to Monitor Scour Critical Bridges

Dave Claman
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
david.claman@dot.iowa.gov

ABSTRACT

The Iowa Department of Transportation (Iowa DOT) has taken a proactive approach towards implementing the FHWA-mandated action plans for scour critical bridges. The Iowa DOT has approximately 180 scour critical bridges that must have an action plan (monitoring plan) that will reduce the risk to the public during a flood.

The Iowa DOT has contracted with a firm that provides a web-based software program (BridgeWatch) that will predict, identify, record, and monitor flood events that may impact scour-susceptible structures. BridgeWatch identifies the occurrence of flood events that affect our scour-critical bridges by acquiring real-time flood (USGS gages) and rainfall data (NWS Doppler Radar).

The floods of 2008 were significant in the state of Iowa, and the BridgeWatch program provided a real-time alert system so that the Iowa DOT could allocate its resources to inspect the most vulnerable structures during the floods.

The program will automatically notify Iowa DOT maintenance personnel (by cell phone) of possible flooding at scour critical bridge sites. The BridgeWatch program provided an invaluable tool during the flooding, which enabled the Iowa DOT to make prioritized decisions regarding inspection efforts and bridge closures, which will save time, resources, and needless inspection of bridges that did not experience significant flooding.

Key words: BridgeWatch—floods—scour critical bridges

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Monitoring of an Integral-Abutment Bridge Supported on Steel Piles/Concrete Drilled Shafts in Glacial Clay

Ashley Disburg
Bridge Engineering Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
adisburg@iastate.edu

Robert E. Abendroth
Bridge Engineering Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
rabendro@ iastate.edu

Brent M. Phares
Bridge Engineering Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
bphares@ iastate.edu

Ahmad Abu-Hawash
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
ahmad.abu-hawash@ dot.iowa.gov

Kenneth Dunker
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
kenneth.dunker@dot.iowa.gov

EXTENDED ABSTRACT

Bridge designs consisting of the use of integral abutments supported on steel piles that are embedded in concrete drilled shafts are unique and not prevalent in the U.S. infrastructure. As a result, information is not readily available regarding the design and performance of this specific bridge system. This lack of information or bridge performance history can be an issue for a bridge designer.

This unique integral-abutment design was implemented in the construction of the 9th Street Bridge over I-235 in Des Moines, IA. Because of geotechnical concerns regarding the preservation of nearby historical sites that could be damaged by large vibrations, pile driving was prohibited. Therefore, the bridge’s integral abutments were designed to be supported by steel H-Piles that are embedded in concrete drilled shafts. The steel piles provide horizontal flexibility for bridge movement, and the concrete drilled shafts transfer the bridge loads to the supporting soil strata. Unlike other uses of concrete drilled shafts in Iowa, the shafts are embedded in glacial clay instead of bedrock.

Through the use of bridge instrumentation devices and surveying techniques, the bridge has been monitored through construction and will continue to be monitored to determine bridge behavior by engineers with the Iowa State University (ISU) Bridge Engineering Center (BEC). Monthly surveying of the bridge has established bridge movements relative to nearby benchmarks. During construction, seven steel piles were instrumented to monitor strains in the members. Displacement meters were also placed near the bottom flange of the center girder to measure change in bridge length. Monitoring of bridge behavior will be performed for approximately 18 months. The study of this unique bridge should provide

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confidence in the use of surveying as a method of monitoring for bridges, as well as confidence in future designs of this specific bridge type.

The bridge monitoring has produced results of bridge displacement behavior with respect to changes in temperature. Using surveying methodologies, strain gages, and displacement meters, the change in the bridge length is being evaluated over time and compared with the theoretical change in length based on the effective coefficient of thermal expansion and contraction of the bridge superstructure, bridge length, and temperature of the wire for the displacement meter. As seen in Figure 1, the change in length measured by the displacement meter and as theoretically predicted is plotted versus temperature with a zero date of September 30, 2008. Also included in this figure are boxes that represent the change in the bridge length as measured using surveying for seven different months. The height of each box represents the 95% confidence interval calculated for that specific survey, with the width of the box being the range of temperatures the displacement meter measured during the surveying period. By using these results, the method of surveying as a technique of bridge monitoring can be compared with the other methods of measurement. The boxes, for the most part, fall within the change in length ranges calculated by the displacement meter and theoretical values, which are the traditional methods of measurement in bridge monitoring. Also, Figure 1 illustrates the general trend of change in the bridge length with temperature. As expected, with higher temperatures, there is a positive change in length of the bridge, or expansion, and with colder temperatures, there is a decrease in the bridge length, or contraction. The change in bridge length calculated in the temperature ranges higher than 60°F show a larger spread as can be seen in Figure 1. This may be due to the larger variation in temperatures throughout the day in warmer months rather than in colder months of the year.

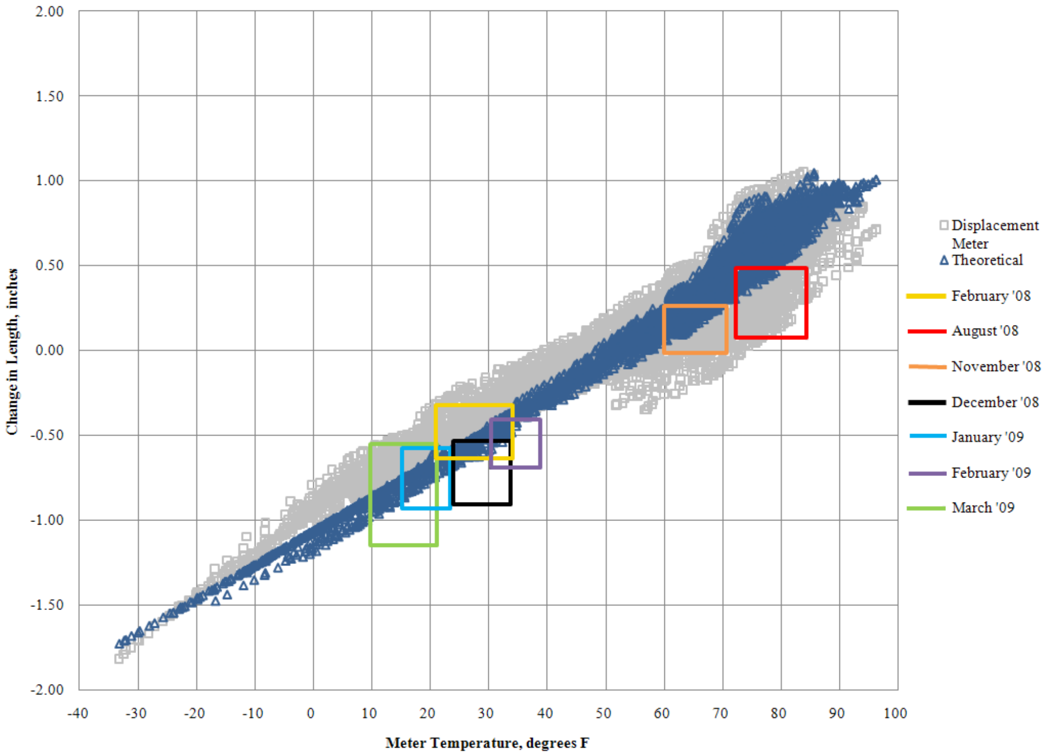

Figure 1. Bridge change in length vs. meter temperature for displacement meter, theoretical and surveying values

Another analysis technique for this bridge was to relate the bridge's behavior with time. Figure 2 shows the same data used in Figure 1 plotted with respect to time. A cyclical pattern can be seen in the graph, which is expected as a result of temperature cycles involving warming and cooling of the bridge. The yellow bars placed over the displacement meter and theoretical data represent the surveying data. The height of a surveying bar is the 95% confidence intervals for that specific survey. Except for the March 2009 survey, the surveying data fits within the data taken for the meter and theoretical calculations. A gap in the displacement meter and theoretical data represent missing results for April 2008 due to a malfunction in the equipment.

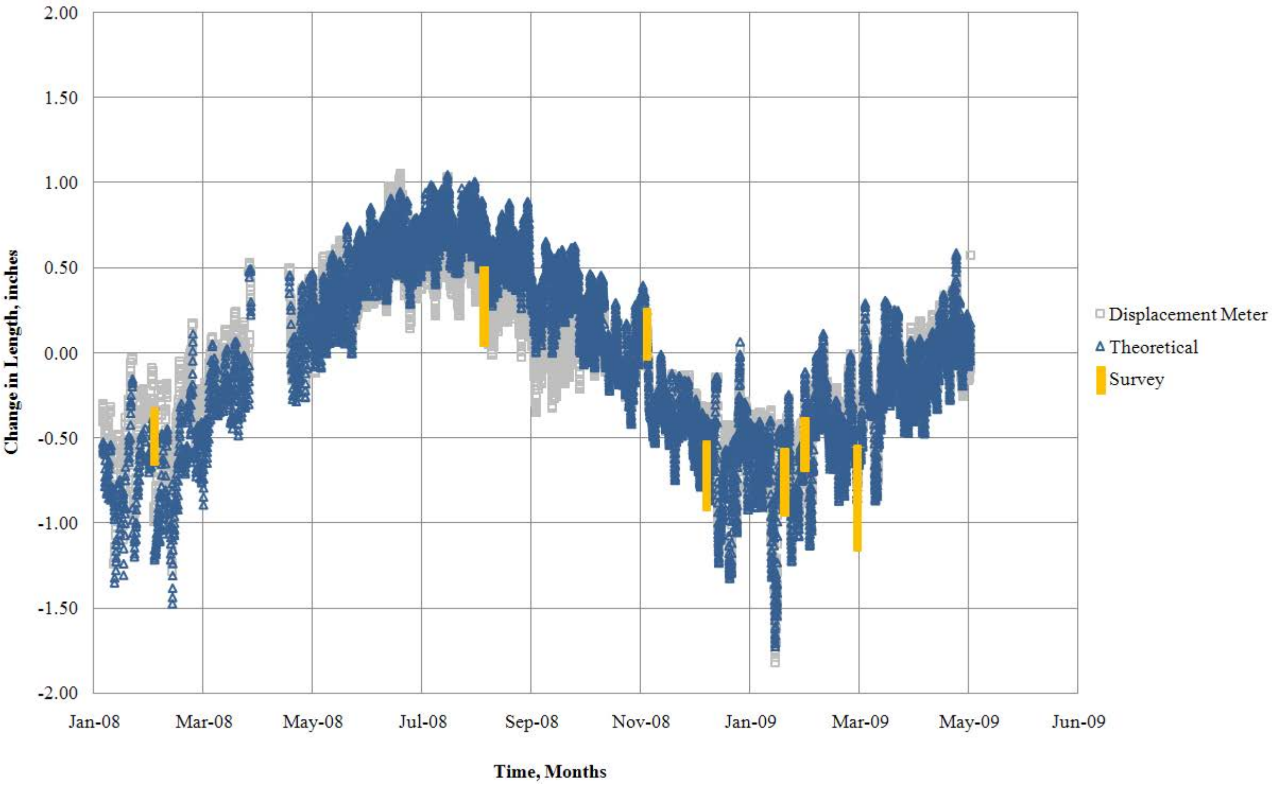

Figure 2. Bridge change in length vs. time for displacement meter, theoretical, and surveying values

Further analysis is being performed on the collected data. The data collected from the strain gages placed on the steel piles is still being evaluated. One of the main issues in the analysis is correlating the three different methods of measurement and the interpretation of the data regarding bridge movements. The initial pile displacements results calculated from the strain gage measurements have been somewhat different from the results calculated through the use of surveying measurements. This discrepancy may be caused by a temperature gradient through the depth of the bridge superstructure that induces curvature of the bridge in the vertical plane. The top of the bridge slab may be warmer or cooler than the bottom of the girders, depending on the weather conditions and time of the year. Other reasons for this discrepancy may be due to assumptions made about the steel piles' end fixity, cross-sectional properties, and alignment and exact strain gage positioning. The accuracy is still being evaluated of measuring bridge movements using surveying. Since the bridge's abutments are skewed, transverse movements of the bridge will also be evaluated in the future.

Key words: drilled concrete shafts—integral abutment—steel piles—surveying

Estimating Economic Benefits Due to Increased Seat Belt Use: A Case Study

Sunanda Dissanayake
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
sunanda@ksu.edu

EXTENDED ABSTRACT

Even though the safety benefits of using seat belts are well-known among the transportation community, the average usage rates in some states remain at relatively low levels than it is desired. Among many factors that could be contributing the situation, the type of the seat belt law could be a major one. While many states still need to aggressively work on passing the primary seat belt law, estimating the benefits associated with increased seat belt use in terms of money might help in convincing the general public and the legislators. Accordingly, this study estimated the effectiveness of seat belts in reducing injuries and the associated economic benefits using state of Kansas data.

The estimation process included two stages. In the first stage, seat belt effectiveness in reducing injuries to motor vehicle occupants was estimated using crash data from the Kansas. These values were estimated using the logistic regression method separately for two vehicle groups: passenger cars and other passenger vehicles that included vans and trucks. In the second stage, the estimated seat belt effectiveness values were used to estimate potential injury reductions due to increased seat belt usage, which were then converted into dollar values by assigning economic costs to each type of injury severity.

According to the estimations, seat belts are 56% effective in preventing fatal injuries when used by passenger car front seat occupants. In the other passenger vehicle group that included vans and pickups, seat belts were found to be 61% effective in preventing fatalities. The seat belt effectiveness in reducing incapacitating and non-incapacitating injuries was found to be 53% and 55%, respectively, for passenger cars group and 52% and 51% for other passenger vehicle group. Based on the economic analysis, it was found that 1% incremental increase in current seat belt usage rate could annually save about $14 million to the state of Kansas. If seat belt usage in Kansas reaches the 2005 national average rate of 82%, the expected annual economic savings could be estimated to be around $222 million. Similar methodology could be used by any other state to estimate the expected economic benefits due to increased seat belt use.

Key words: benefits of seat belts—highway safety—logistic regression—seat belt effectiveness—seat belts

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INTRODUCTION

The economic impacts due to highway crashes are enormous. Numerous efforts have been made to mitigate the vast impact of highway crashes. One of the remarkable implementations in this regard is the introduction of seat belts. It has been proven that seat belts are highly effective in improving passenger safety, saving many lives, and preventing many injuries to vehicle occupants. Due to high benefits already realized and potential future benefits that could be achieved through higher seat belt usage rates, many states have enacted seat belt laws to mandate the use of seat belts. As of 2005, 25 states plus the District of Columbia and Puerto Rico have primary adult seat belt laws, where police officers can stop and cite a motorist for the violation of the seat belt law. In the remaining states except New Hampshire, the law is secondary, where motorists can only be cited for violating the seat belt law after having been stopped for an unrelated traffic violation. The state of New Hampshire is the only U.S. state that does not mandate any seat belt law.

Despite the proven economical and health benefits derived from the seat belt usage, many U.S. states still observe low seat belt usage rates. According to the 2005 observational seat belt survey results, about 50% of the U.S. states still have seat belt usage rates that are less than the national average rate of 82%. In Kansas, where the law is secondary, the observed usage rate in 2005 was at 69%, which is significantly lower than the national average. Additionally, Kansas is among the 10 states with the lowest seat belt usage rates.

When considering benefits of seat belt use, it is unfortunate to note that more motorists are still not taking advantage of these benefits. Failure to use seat belts is costly, not only to the nonuser of the seat belt, but also to the whole society. Therefore, it is very important for transportation authorities to find ways that can increase the seat belt usage rate among motorists. Previous research efforts have indicated that the most effective way to increase the seat belt usage rate is through strong enforcement. In other words, seat belt usage rates can be increased by mandating the enforcement of a primary seat belt use law. According to National Highway Traffic and Safety Administration seat belt survey results, the change in law from secondary to primary has dramatically increased the observed seat belt usage rates in many states. However, such a decision should be well-supported by proven benefits at the local level. Although national estimations are available to quantify the benefits associated with seat belt use, it would be useful for the local authorities to have the estimated benefits (in dollar terms) derived from their local conditions in order to better promote the seat belt usage-related policies and campaigns.

OBJECTIVES

The main objective of this study was to estimate the effectiveness of seat belts in reducing injuries and to estimate associated economic benefits using state of Kansas data. This objective was achieved through two major stages. In the first stage, the motor vehicle crash data for the state of Kansas were used to estimate the effectiveness of seat belts in reducing fatal and nonfatal injuries. In the second stage, the expected economic benefits were estimated using the effectiveness values estimated in the first stage.

CALCULATION OF SEAT BELT EFFECTIVENESS

Highway crash data from Kansas Accident Reporting System (KARS) database were used to estimate the seat belt effectiveness. Data related to vehicles that were involved in crashes between 1993 and 2002 were extracted from the database. Only front-seat occupants of passenger cars, vans, and pickup trucks were considered in the analysis. Since the data availability for vans were limited, especially for fatalities, pickup trucks and vans were combined and considered as a single vehicle group. Thus, the estimations were based on two vehicle groups: passenger cars and other passenger vehicles. The seat belt effectiveness was then estimated using logistic regression modeling.

According to logistic regression estimations for the passenger cars group, seat belts are 56% effective in preventing fatalities to front-seat occupants in passenger cars. In other words, 56% of fatally injured front-seat occupants who were unrestrained at the time of the crash could have survived if all of them were restrained. As far as nonfatal injuries are concerned, seat belts are more effective in reducing non-incapacitating injuries (55%) compared to incapacitating injuries (53%). Additionally, seat belts are 33% effective in reducing possible injuries to passenger car front-seat occupants. For the other passenger vehicles group, seat belts are 61% effective in preventing fatal injuries to front-seat occupants. Seat belts are 52% effective in reducing incapacitating injuries and 51% effective in reducing non-incapacitating injuries in this vehicle group. The seat belt effectiveness for possible injuries in this vehicle group is 34%, which is slightly higher than the value obtained for the passenger cars group.

The procedure described by Blincoe was used to estimate the benefits associated with seat belt use. Some adjustments were made in the original procedure in order to accommodate for Kansas local conditions. The following steps were included in the process:

Step 1: Obtain injury frequencies
Step 2: Estimate average seat belt effectiveness
Step 3: Obtain seat belt usage rates
Step 4: Estimate expected safety improvements
Step 5: Obtain potential reduction in injuries
Step 6: Estimate economic savings

CONCLUSIONS

Based on the analysis conducted in this study, it was found that the first 1% incremental increase in current seat belt usage rate could annually save about $14 million to the state of Kansas. If seat belt usage in Kansas reaches the national average rate of 82% (2005 value), the resulted annual economic savings are estimated to be around $260 millions. In other words, due to the current low seat belt usage in Kansas as compared to the national average, the annual estimated economic loss is about $222 million. Moreover, about 37 additional lives could be saved if the current state seat belt usage rate of 69% is increased to the national average of 82%. It should be noted that the economic benefits estimated in this study only provide approximate values, and the real benefits may vary due mainly to the difficulty in estimating economic cost of motor vehicle crashes. Moreover, there may be some concerns about the accuracy of the data used in the analysis especially data related to seat belt usage and injury severities, which might have impacted the final estimated values. However, this type of dollar estimations would be useful to the local transportation officials to convince the general public about the benefits associated with increased seat belt use, which would in turn help win support for policy changes regarding set belt use laws.

Characteristics of Work Zone Crashes in the SWZDI Region: Differences and Similarities

Sunanda Dissanayake
Department of Civil Engineering
Kansas State University
2118 Fielder Hall
Manhattan, KS 66506
sunanda@ksu.edu

Sreekanth Reddy Akepati
Department of Civil Engineering
Kansas State University
2118 Fielder Hall
Manhattan, KS 66506
srireddy@ksu.edu

ABSTRACT

Since construction of most of the major highway networks in the United States has already been completed, the majority of the current highway work includes maintenance and rehabilitation of those highways. With the dramatically increasing traffic demand on those highways, management of such work zones has become an extremely challenging task for highway agencies. One of the critical issues in this regard is the user and worker safety at work zones. Every year, more than 1,000 people die and thousands more are injured at work zones across the United States due to traffic crashes. Thus, proper traffic management at work zones is very important to improve the safety of both highway users and workers. In this regard, detailed knowledge on crashes at work zones and crash characteristics would be highly valuable for highway agencies in setting up proper traffic-management plans at work zones based on the prevailing conditions.

Accordingly, this study summarizes the work zone crash characteristics from the states currently included in the Smart Work Zone Deployment Initiative (SWZDI) while comparing differences and similarities. The five states included are Iowa, Kansas, Missouri, Nebraska, and Wisconsin. Work zone crash data from the five states were gathered, and five years of combined data from 2002 to 2006 were analyzed to identify important crash characteristics. Various factors, such as injury severities, weather conditions, vehicle characteristics, roadway characteristics, vehicle maneuvers, alcohol involvement, and other driver characteristics related to work zones, were identified. Differences and similarities of these characteristics are discussed among the SWZDI states. Work zone crash data from the five states will then be combined to the extent possible, and common characteristics will be identified for the region.

Key words: highway safety—SWZDI —work zone crashes—work zone safety

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INTRODUCTION

Road safety is a growing concern among many road users with the increase in the number and utilization of the highway networks**.** As the number of vehicles on highways increases, the necessity for maintenance and rehabilitation of the existing highways also increases. Departments of transportation of various states and other agencies typically maintain the roads in proper standards and conditions. It is not always possible to stop the flow of traffic as the maintenance and rehabilitation works are going on the pavements. As a result, the lane where the rehabilitation work is going on should be closed, which leads the vehicles to move into other lanes. These typical scenarios, among many others, create work zones that have critical traffic flows due to less space for the movement traffic. A work zone is defined as an area on a trafficway where construction, maintenance, or utility work activities take place. Due to highway work zones, an inevitable disruption of regular traffic flows will take place, which results in traffic safety concerns.

To improve the safety and efficiency of traffic operations and highway work, in 1999, the states of Iowa (the leading state), Kansas, Missouri, and Nebraska created Smart Work Zone Deployment Initiative (SWZDI). Later on in 2001 Wisconsin joined SWZDI (Iowa Department of Transportation). Through this pooled-fund study, researchers investigate better ways of controlling traffic through work zones, thereby improving the safety and efficiency of traffic operations and highway workers. This study investigated the work zone crash characteristics from the states currently included in the SWZDI for the period 2002–2006 and identified the differences and similarities.

PROBLEM STATEMENT

In the United States, more than 1,000 people die in work zones each year (i.e., nearly three a day). Four out of five people killed in work zones are motorists, not highway workers. In the past five years (i.e., for the period 2002-2006) nearly 241,943 fatalities occurred in the United States, out of which nearly 5,406 (2.23%) fatalities occurred near work zones (The National Work Zone Safety Clearinghouse 1998)**.** The number of fatalities is less when compared to total fatalities, but it is more likely that these crashes are avoidable. Many studies have been conducted to study crash characteristics at work zones. However, the results are not always consistent with respect to different characteristics identified in each study. When it comes to work zones, even the smallest mistake can be unsafe. It just takes few more moments to travel through a work zone at reduced speeds, which when not followed, can result in bitter consequences. With the intention of proposing effective countermeasures, this study analyzed work zone crash characteristics from the region.

OBJECTIVES

The specific objectives of this study are:

a) To identify the predominant characteristics and contributory causes of crashes in work zones
b) To compare the crash characteristics and contributory causes among the five states included in the SWZDI

LITERATURE REVIEW

To gather the results of the previous work zone crash characteristics a brief literature review was done, and the results are summarized briefly as follows.

Garber and Zhao (2002) conducted a study on characteristics of work zone crashes in Virginia that occurred between 1996 and 1999. The study was done to identify the predominant location within work zones where crashes occurred, severity type, and collision type. A total of 1,484 crashes were considered from the Virginia accident database, and the location of each crash was identified by a careful examination of the police accident reports. The results showed that the proportion of crashes occurring at the activity area was significantly higher than that at each of the other locations. This indicates that activity area is more susceptible to crashes regardless of the type of highways. The study noted that only 24 crashes out of the 1,484 occurred in the termination area, which appears to be the safest area in a work zone. For all the crashes studied, "property damage only" (PDO) crashes are the predominant in terms of severity type, and rear-end crashes are predominant in terms of collision type.

Another study was conducted by Hall and Lorenz (1989) to improve the safety of highway work zones in New Mexico. The main objective of this study was to identify the characteristics of work zone crashes that differ from other crashes of comparable roadways and to develop effective countermeasures for prevalent work zone safety problems. The researchers examined highway work zone crashes in New Mexico for a three-year period from 1983 to1985 on a basis of comparing the crashes in several roadway sections during construction with those in previous years with the same road sections. The results showed that the proportion of crashes that were caused mainly by following too close was much higher in "during-work zone periods" than in "before-work zone periods." In comparison with the identical period in the prior year, crashes in construction areas increased 33% on the rural interstate system, 17% on the rural federal-aid primary highways, and 27% on the rural federal-aid secondary highways. Improper traffic control was the prevalent problem, causing high crash rates in work zones.

Pigment and Agent (1990) conducted a study by comparing highway work zone crashes with non-work zone highway crashes at the University of Kentucky. The data used in this study was obtained from Kentucky Accident Reporting System (KARS) and on-site collection. The researchers studied the traffic crash data and traffic control devices of 20 highway work zones for a three-year period (1983–1986). Based on the study, they found that most work zone crashes occur on Interstate routes and work zone crashes, especially those during darkness or involving trucks, were more severe than other crashes. The study also showed that percentage of rear-end and same-direction sideswipe crashes in the work zone was almost three times of the percentage of the same types in the statewide non-work zone crashes, and the greatest contributing factor of work zone crashes was following too close.

Mattox, Sarasua, and Eckenrode (2006) conducted a study on the development and evaluation of a speed-activated sign to reduce speeds in work zones. A leading cause of vehicle crashes near the work zone was driving too fast. In South Carolina, work zone crashes were tripled from the beginning of the year 2000 to the end of the year 2003. Due to this increasing amount of work zone crashes and fatalities in South Carolina, improving driver attention and reducing vehicle speeds in work zones has become a priority of the South Carolina Department of Transportation (SCDOT). Because of the limited availability of law enforcement and inadequate funding for widespread deployment of expensive technologies, transportation agencies need more affordable technologies to reduce speeds near work zones .To address this need, a traffic control device known as a speed-activated sign that reduces the speeds and cost effective was deployed near the work zones by SCDOT. A speed-activated sign triggers a flashing beacon when a predetermined speed threshold is exceeded. For the purpose of evaluation of the speed activated sign, three locations in each work zone were selected such that the three stations were positioned before, at, and after the speed-activated sign, and the variability of speeds of the approaching vehicles were collected by using laser speed guns with radar detectors. The speed data were collected for two conditions: one without the speed-activated sign and another one with the speed-activated sign in place. The results showed that the average mean speed was reduced by 3.29 mph, and the 85% speed is 3.22 mph; the percentage of vehicles exceeding the speed limit by +3 mph is 23.42%. The speed-activated sign should be placed in the advance-warning area of the work zones to slow vehicles prior to entering activity areas.

Vicki and Jonathan (1999) conducted a study that dealt with the effective countermeasures that were used in Arizona State to reduce accidents in work zones. The first objective of this project was characterizing the nature of work zone accidents in Arizona. To accomplish this objective, a total of 14,905 work zone accidents that took place between1992 and 1996 were collected from the Accident Location Information Surveillance System accident record database. This includes the accidents that took place near through-traffic allowed, traffic detoured, under repairs, and temporary lane closure. These accidents were analyzed by categorizing them into different groups like severity, number of fatal, injury, and property damage, and the conditions were that accidents took place. Based on the results that were obtained from the analysis, different effective countermeasures were illustrated in order to reduce accidents in work zones. The countermeasures that were illustrated were police presence, enhanced fines, changeable message signs, radar-activated horn system, display license plate number and speed of speeding vehicle, and intrusion alarm.

Li and Bai (2006) conducted a study “Determining Major Causes of Highway Work Zone Accidents in Kansas.” The primary objective of this study was to investigate the characteristics of fatal crashes and risk factors to these crashes in the work zones. A total of 157 fatal crash cases between 1992 and 2004 were collected from the KDOT accident database and examined. The results showed that male drivers cause about 75% of the fatal work zone crashes in Kansas. Drivers that are between 35- and 44-years old and older than 65, are the high-risk driver groups in work zones. Work zones on rural two-lane highways with speed limits from 51 to 70 mph were high-risk locations, accounting for 59% of the fatal crashes. Most (68%) of the crashes are multi-vehicle crashes. Among the multi-vehicle collisions, head-on, angle-side impact, and rear-end are the three most frequent collision types. The daytime non-peak hours (10:00 a.m.–4:00 p.m.) were the most hazardous time period in work zones, accounting for 32% (highest hourly rate) of the fatal crashes. Nighttime (8:00 p.m.–6:00 a.m.) had 37% of the crashes. Human errors, including inattentive driving and misjudgment/disregarding traffic control, were the top killers in work zones. Inclement weather conditions and unfavorable road features (interchange areas, intersections, ramps, etc.) do not significantly contribute to fatal work zone crashes.

DATA AND METHODOLOGY

Work zone crash data for the SWZDI region states were gathered from the respective departments of transportations for the period 2002–2006. The states are Iowa, Kansas, Missouri, Nebraska, and Wisconsin. The data that is gathered contains all the characteristics related to crashes in or near work zones in those five states. Crash characteristics of each state are retrieved by using Microsoft Excel and Microsoft Access, and Statistical Analysis System (SAS) is used for merging two files. Later work zone crash data from five states were combined using common variables, and comparisons were made.

RESULTS AND FINDINGS

Combined work zone crash data for the SWZDI states for the period 2002–2006 were analyzed to identify the characteristics, and the results are presented here.

Crashes occurred under different environmental conditions, such as light conditions, and weather conditions were analyzed to identify the characteristics of crashes. Distributions of crashes based on light conditions are shown in Figure 1. The results showed that most of the work zone crashes in five states occurred during daylight conditions, where most of the work was performed when compared to poor visibility conditions. The highest proportion (79%) of crashes under daylight conditions occurred in Iowa when compared to the other four states. Under poor visibility conditions, dark with street lights, dark without street lights, and dawn and dusk conditions were combined, and the highest percentage was recorded in Nebraska (27.4%) among the five states considered in the study.

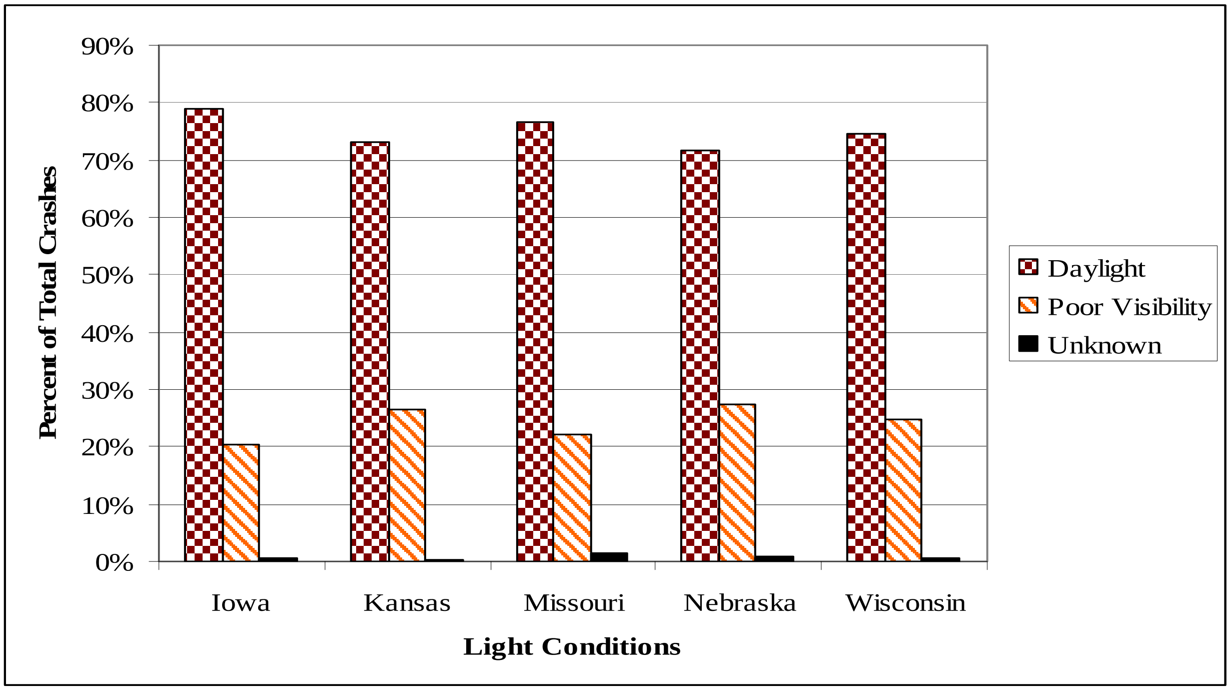

Figure 1. Distribution of crashes based on different light conditions

The majority of work zone crashes occurred under clear weather conditions. Distribution of crashes based on weather conditions is shown in Figure 2. When all the five states were considered, the majority of the work zone crashes occurred under no adverse weather conditions. Among the five states, Kansas has the highest percentage (88.2%) of work zone crashes under clear weather conditions. Wisconsin has the highest percentage (32%) of crashes occurred under cloudy conditions.

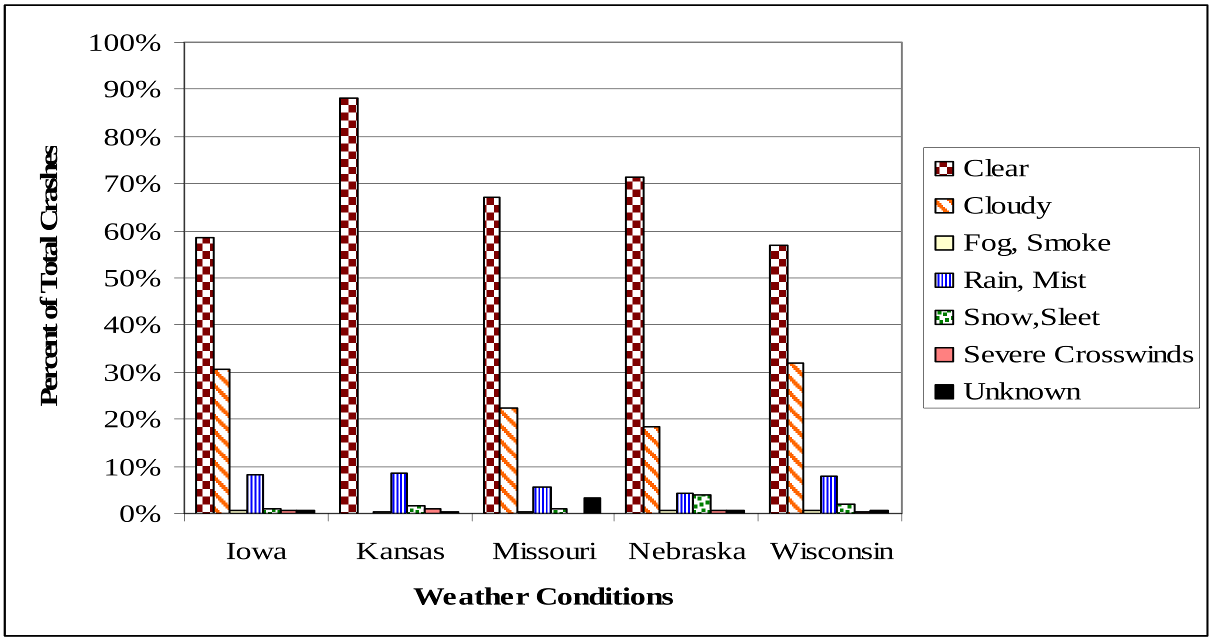

Figure 2. Distribution of crashes based on weather conditions

The type of traffic control used at the crash location plays a very important role in terms of work zone safety. The efficiency of the control can be determined based on crash severity. Work zone crashes based on type of traffic control used at the time of crash is shown in Table 1. Based on the total crashes in four states, most of the crashes occurred at a place where there is no traffic control within the work zones. Wisconsin has the highest percentage (54.3%) of crashes in work zones where there is no traffic controls when compared to the other three states. Significant percentage (52.9%) of crashes occurred in Kansas due to crossing the center or edge line within the work zones. The type of traffic controls used in work zones at the time crash for Nebraska is incomplete in the database available to the authors.

Table 1. Work zone crashes based on type of traffic control used at the location of crash

Traffic Controls	Iowa	Kansas	Missouri	Wisconsin	Total
	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)
None	1,782 (48.3)	1,390 (15.4)	1,970 (10.2)	9,825 (54.3)	14,967 (29.8)
Stop, Yield Signs	201 (5.4)	573 (6.3)	816 (4.2)	1,312 (7.3)	2,902 (5.8)
Traffic Signal	547 (14.8)	989 (10.9)	1,722 (8.9)	2,928 (16.2)	6,186 (12.3)
Officer/Flagman	33 (0.9)	106 (1.2)	429 (2.2)	444 (2.5)	1,012 (2)
Rail Road Signal/Gate	6 (0.2)	11 (0.1)	6 (0.0)	9 (0.0)	32 (0.1)
No Passing Zone	12 (0.3)	283 (3.1)	1,085 (5.6)	0 (0)	1,380 (2.8)
Center/Edge Line	0 (0)	4,787 (52.9)	0 (0)	0 (0)	4,787 (9.5)
Warning Sign	930 (25.2)	0 (0)	0 (0)	641 (3.5)	1,571 (3.1)
School Zone	4 (0.1)	0 (0)	13 (0.1)	0 (0)	17 (0.0)
Unknown	174 (4.7)	913 (10.1)	1,3299 (68.8)	2,931 (16.2)	17,317 (34.5)
Total	3,689	9,052	19,340	18,090	50,171

The maintenance of posted speed limit within the work zones is very important in improving safety. The posting of speed limit is done for the safety of road users. Work zone crashes based on posted speed limits at the time of crash is shown in Table 2. The results showed that for all the five states, most of the work zone crashes occurred within the posted speed limit range of 51–60 mph. Iowa is the one that has the highest number of crashes (39%) within the posted speed limit range of 51–60 mph when compared to other four states. A significant percentage of crashes (24.3%) of crashes occurred in Wisconsin within the speed limit range of 31–40 mph when compared to all other states.

Table 2. Work zone crashes based on posted speed limit at the location of crash

Speed Limit (mph)	Iowa	Kansas	Missouri	Nebraska	Wisconsin	Total
	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)
0–20	57 (1.5)	245 (2.8)	348 (1.8)	256 (8.9)	179 (2)	1,085 (2.5)
21–30	700 (19)	1,413 (16)	2,580 (13.3)	330 (11.5)	1,534 (17)	6,557 (15)
31–40	748 (20.3)	1,901 (21.6)	4,199 (21.7)	597 (20.7)	2,198 (24.3)	9,643 (22)
41–50	374 (10.1)	1,111 (12.6)	4,766 (24.6)	584 (20.3)	1,615 (17.9)	8,450 (19.3)
51–60	1,440 (39)	2,318 (26.3)	4,356 (22.5)	553 (19.2)	2,767 (30.6)	11,434 (26.1)
61–70	266 (7.2)	1,774 (20.1)	1,659 (8.6)	220 (7.6)	565 (6.2)	4,484 (10.2)
71–80	0 (0)	0 (0)	0 (0)	220 (7.6)	187 (2.1)	407 (0.9)
Unknown	104 (2.8)	290 (3.2)	1,432 (7.4)	118 (4.1)	0 (0)	1,697 (3.9)
Total	3,689	9,052	19,340	2,878	9,045	44,004

Crash severity is defined as the highest level of injury severity. Distribution of crashes based on crash severity is shown in Figure 3. In all five states, most of the work zone crashes lead to PDO crashes. Missouri has the highest number of PDO crashes (77%) when compared to all other states. When injury and fatal crashes are considered, Nebraska has a significant percentage (41.1% and 1.4%, respectively) as compared to the other four states.

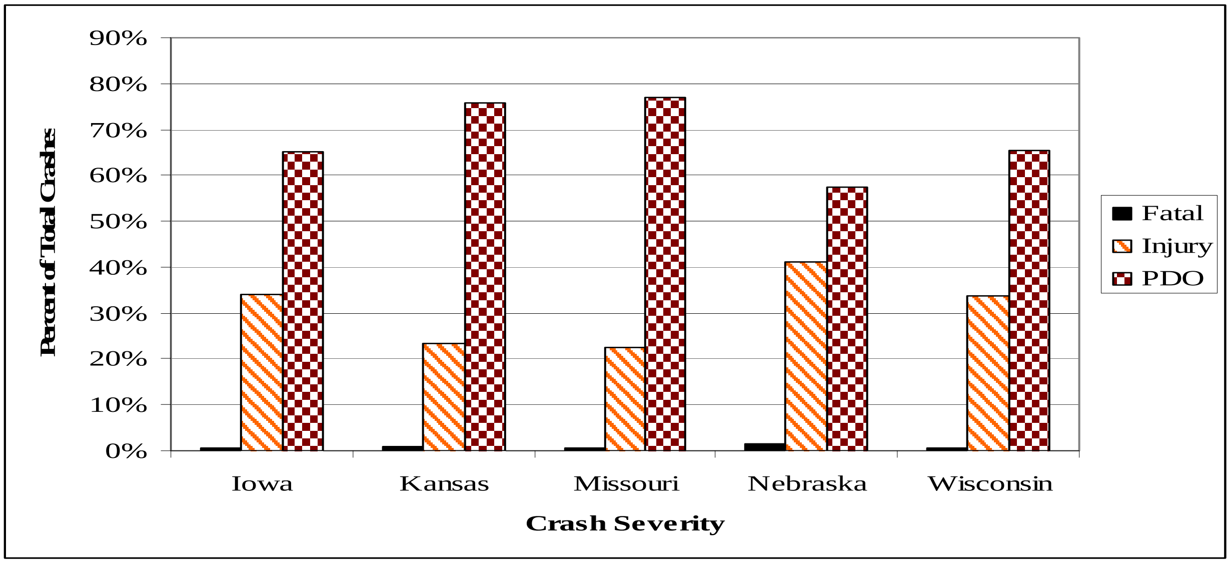

Figure 3. Distribution of crashes based on crash severity for the five states

Work zones tend to have less space for the flowing traffic, which might increase the possibility of interaction of vehicles, which leads to multiple-vehicle crashes. Distribution of crashes based on number of vehicles involved in the crash is shown in Figure 4. The results based on number of vehicles involved in work zone crashes for all five states showed that the crashes involving two vehicles are more predominant than single vehicles and crashes involving three or more vehicles. Missouri has the highest percentage (69%) of two-vehicle crashes when compared to other four states. Kansas has significant percentage (29%) of single-vehicle crashes among the five states.

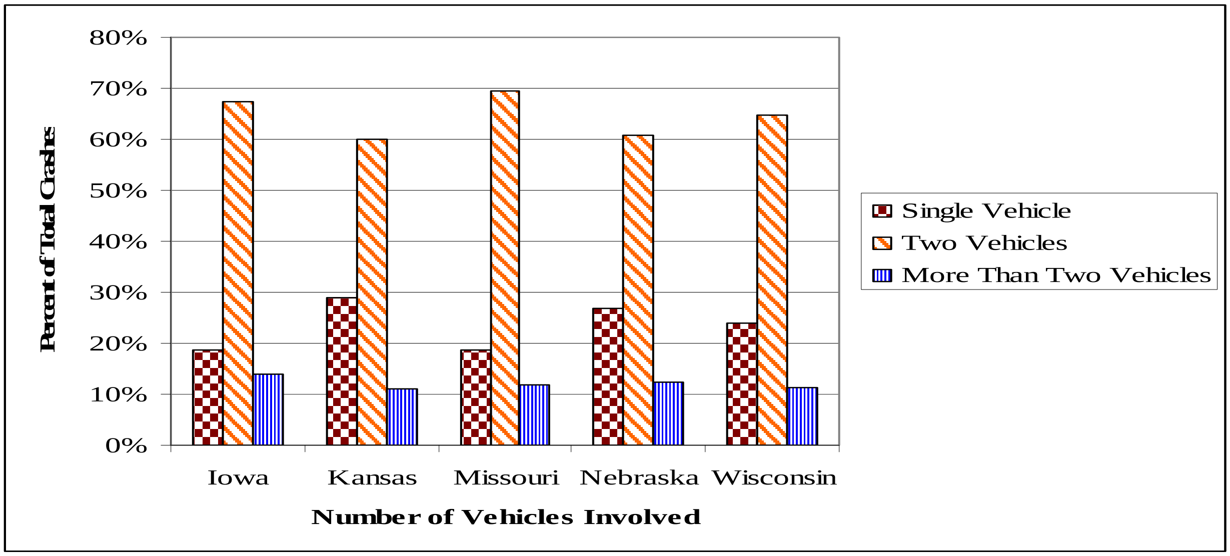

Figure 4. Distribution of crashes based on number of vehicles involved in the crash

Distribution of crashes based on the day of the week is shown in Table 3. The results showed that on average, the crashes were evenly distributed on the weekdays. When the weekends are considered, Kansas has the highest percentage (12.3%) of crashes on Saturday.

Table 3. Work zone crashes based on the day of week

Day of Crash	Iowa	Kansas	Missouri	Nebraska	Wisconsin	Total
	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)
Sunday	260 (7)	782 (8.6)	1,374 (7.1)	257 (8.9)	781 (8.6)	3,454 (7.8)
Monday	555 (15)	1,296 (14.3)	2,762 (14.3)	373 (13)	1,398 (15.5)	6,384 (14.5)
Tuesday	590 (16)	1,374 (15.2)	3,211 (16.6)	424 (14.7)	1,352 (14.9)	6,951 (15.8)
Wednesday	630 (17.1)	1,434 (15.8)	3,279 (17)	451 (15.7)	1,527 (16.9)	7,321 (16.6)
Thursday	656 (17.8)	1,429 (15.8)	3,243 (16.8)	445 (15.5)	1,471 (16.3)	7,244 (16.5)
Friday	627 (17)	1,622 (17.9)	3,475 (18)	474 (16.5)	1,522 (16.8)	7,720 (17.5)
Saturday	371 (10.1)	1,115 (12.3)	1,987 (10.3)	336 (11.7)	994 (11)	4,803 (10.9)
Unknown	0 (0)	0 (0)	9 (0.0)	118 (4.1)	0 (0)	127 (0.3)
Total	3,689	9,052	19,340	2,878	9,045	44,004

The type of maneuvering at the time of work zone crashes is shown in Table 4. The results show that most of the vehicles were going straight and following the road at the time of the crash. Missouri has highest number of crashes (68.6%) with vehicles going straight, as compared to the other four states. The significant percentage (15.5%) of crashes also occurred in Nebraska when the vehicles were stopped or when they were slowing down due to the traffic.

Table 4. Work zone crashes based on vehicle maneuver before the crash

Vehicle Maneuver Before Crash	Iowa	Kansas	Missouri	Nebraska	Wisconsin	Total
	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)
Straight/Following Road	2,159 (58.5)	4,689 (51.8)	13,271 (68.6)	1,768 (61.4)	7,526 (41.6)	16,142 (48.3)
Turning Left	163 (4.4)	512 (5.7)	590 (3.1)	178 (6.2)	1,225 (6.8)	2,078 (6.2)
Turning Right	84 (2.3)	214 (2.4)	281 (1.5)	54 (1.9)	561 (3.1)	913 (2.7)
Making U-Turn	14 (0.4)	30 (0.3)	24 (0.1)	3 (0.1)	58 (0.3)	105 (0.3)
Overtaking/Passing	21 (0.6)	86 (1)	110 (0.6)	39 (1.4)	158 (0.9)	304 (0.9)
Changing Lanes	115 (3.1)	237 (2.6)	243 (1.3)	89 (3.1)	585 (3.2)	1,026 (3.1)
Backing	52 (1.4)	167 (1.8)	256 (1.3)	23 (0.8)	344 (1.9)	586 (1.8)
Slowing/Stopping	552 (15)	1,016 (11.2)	778 (4)	445 (15.5)	2,554 (14.1)	4,567 (13.7)
Stopped in Traffic	128 (3.5)	1,448 (16)	2,778 (14.4)	0 (0)	1,596 (8.8)	3,172 (9.5)
Merging	128 (3.5)	134 (1.5)	26 (0.1)	34 (1.2)	386 (2.1)	682 (2)
Leaving Traffic Lane	19 (0.5)	0 (0)	113 (0.6)	26 (0.9)	0 (0)	45 (0.1)
Parked	94 (2.5)	23 (0.3)	49 (0.3)	2 (0.1)	316 (1.7)	435 (1.3)
Unknown	160 (4.3)	496 (5.5)	821 (4.2)	217 (7.5)	2,781 (15.4)	3,981 (11.9)
Total	3,689	9,052	19,340	2,878	18,090	33,417

The size, weight, and handling characteristics of trucks require additional consideration within the work zones. Large trucks are involved in fewer crashes when compared to passenger cars, but their involvement rate in fatal accidents is almost twice that of passenger cars. Work zone crashes based on vehicle body type is shown in Table 5.The results showed that in Missouri, around 70.5% of work zone crashes involved passenger cars when compared to other states. A significant percentage (13.5%) of crashes occurred in Wisconsin work zones involving trucks—either a single unit or combination trucks. The type of vehicle that is involved in work zones at the time of crash for Nebraska was incomplete in the available database.

Table 5. Work zone crashes based on vehicle body type

Vehicle Body Type	Iowa	Kansas	Missouri	Wisconsin	Total
	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)
Passenger Car	1,862 (50.5)	4,559 (50.4)	9,516 (49.2)	6,372 (70.5)	22,309 (54.3)
Motor Cycle	52 (1.4)	83 (0.9)	137 (0.7)	159 (1.8)	431 (1)
Recreational Vehicle, Moped	12 (0.3)	14 (0.2)	52 (0.3)	21 (0.2)	99 (0.2)
Pick-up Truck	584 (15.8)	1,726 (19.1)	3,014 (15.6)	0 (0)	5,324 (12.9)
Van	269 (7.3)	682 (7.5)	1,406 (7.3)	0 (0)	2,357 (5.7)
Sport Utility Vehicle	373 (10.1)	1,033 (11.4)	2,356 (12.2)	0 (0)	3,762 (9.1)
Farm Vehicle/Equipment	5 (0.1)	10 (0.1)	10 (0.1)	0 (0)	25 (0.1)
Trucks	416 (11.3)	783 (8.7)	2,333 (12.1)	1,221 (13.5)	4,753 (11.6)
Bus, Train	6 (0.2)	30 (0.3)	171 (0.9)	22 (0.2)	229 (0.6)
Maintenance/Construction Vehicle	55 (1.5)	0 (0)	0 (0)	0 (0)	55 (0.1)
Utility Truck, Emergency Vehicle	0 (0)	6 (0.1)	0 (0)	1,180 (13)	1,186 (2.9)
Construction Equipment	0 (0)	0 (0)	148 (0.8)	0 (0)	148 (0.4)
Unknown	55 (1.5)	126 (1.4)	197 (1)	70 (0.8)	440 (1.1)
Total	3,689	9,052	19,340	9,045	41,126

Drivers play a key role in the crash involvement, and identification of driver contribution towards crashes is highly important in suggesting possible countermeasures. Work zone crashes based on driver contributing circumstances is shown in Table 6. For a given crash, there could be more than one contributing factor, and as a result, the sum of contributing factors is greater than the actual number of crashes occurred. The results show that most of work zone crashes occurred due to inattentive driving, improper action of the driver, following too close, failed to yield right of way, and driving too fast for the conditions. In Kansas, the highest percentage of crashes (37.8%) occurred due to inattentive driving of the driver, according to the situations of the work zones. The significant percentage (7.6%) of crashes occurred in Missouri work zones due to exceeding the posted the speed limit.

Table 6. Work zone crashes based on driver contributing circumstances

Driver Contributing Circumstances	Iowa	Kansas	Missouri	Nebraska	Wisconsin	Total
	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)	Number (%)
Disregarded Traffic Controls	43 (1.1)	240 (3)	133 (0.7)	91 (3.6)	340 (1.9)	847 (1.6)
Exceeded Posted Speed Limit	27 (0.7)	46 (0.6)	230 (1.2)	17 (0.7)	246 (1.4)	566 (1.1)
Driving Too Fast for Conditions	165 (4.1)	372 (4.7)	1,229 (6.4)	102 (4.1)	874 (4.8)	2,742 (5.3)
Made Improper Turn	34 (0.8)	135 (1.7)	201 (1)	14 (0.6)	223 (1.2)	607 (1.2)
Following Too Close	349 (8.7)	1,021 (12.9)	804 (4.2)	339 (13.6)	1,265 (7)	3,778 (7.3)
Failed to Yield Right of Way	123 (3.1)	499 (6.3)	689 (3.6)	195 (7.8)	1,486 (8.2)	2,992 (5.8)
Avoiding Vehicle, Object in Roadway	72 (1.8)	217 (2.7)	0 (0)	50 (2)	0 (0)	339 (0.7)
Operating Vehicle in an Aggressive Manner	139 (3.5)	95 (1.2)	0 (0)	93 (3.7)	0 (0)	327 (0.6)
Wrong Side or Wrong Way	45 (1.1)	55 (0.7)	82 (0.4)	12 (0.5)	73 (0.4)	267 (0.5)
Other Distractions	60 (1.5)	173 (2.2)	0 (0)	54 (2.2)	0 (0)	287 (0.6)
Vision Obstructed	53 (1.3)	0 (0)	0 (0)	13 (0.5)	0 (0)	66 (0.1)
Inattentive Driving	382 (9.5)	3,001 (37.8)	1,738 (9)	180 (7.2)	2,784 (15.4)	8,085 (15.6)
Other Improper Action	361 (9)	459 (5.8)	1,332 (6.9)	132 (5.3)	359 (2)	2,643 (5.1)
No Improper Action/ None	1,676 (41.9)	0 (0)	11,984 (62)	1,111 (44.4)	0 (0)	14,771 (28.5)
Driving Under Influence	0 (0)	110 (1.4)	307 (1.6)	0 (0)	0 (0)	417 (0.8)
Unknown/Other	472 (11.8)	1,517 (19.1)	611 (3.2)	98 (3.9)	10,440 (57.7)	13,138 (25.3)
Total	4,001	7,940	19,340	2,501	18,090	51,872

CONCLUSIONS

This study analyzed the combined work zone crash data from 2002–2006 time period for the five states included in the Smart Work Zone Deployment initiative and identified characteristics and contributory causes. Among the findings, the majority of the work zone crashes occurred under clear daylight conditions and no adverse weather conditions. The results showed that most of the crashes in work zones occurred in the absence of traffic control devices. Crashes involving two vehicles are more predominant than single-vehicle crashes. The top three driver contributing circumstances for work zone crashes were inattentive driving, exceeding speed limit/driving too fast for conditions, and failing to yield right of way.

The results showed that the majority of the crashes occurred within the posted speed limit range of 51–60 mph. At the time of the crash occurrence, the majority of the vehicles were going straight or following the road. A majority of the work zone crashes lead to PDO crashes. Passenger cars are more involved in work zone crashes as they typically tend to be more on roads. There were only slight differences in characteristics among the five states considered in this study. The findings could be used to identify suitable countermeasures for improving work zone safety.

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Construction QA/QC Testing versus Selection of Design Values for PCC Pavement Foundation Layers

Samuel Caleb Douglas
Earthworks Engineering Research Center
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50011
calebd@iastate.edu

David J. White
Earthworks Engineering Research Center
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50011
djwhite@iastate.edu

Jeffery R. Roesler
Department of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
1211 Newmark Civil Engineering Laboratory
205 North Mathews Avenue
Urbana, IL 61801
jroesler@uiuc.edu

ABSTRACT

Traditional quality assurance/quality control (QA/QC) practices for construction of embankment, subgrade, and aggregate base layers in Portland cement concrete (PCC) pavement foundation systems generally rely on a soil classification scheme, percent relative compaction, and moisture content. These parameters are measured periodically during construction from a small volume of material to quantify acceptance. Pavement analysis and design, on the other hand, is based on selection of mechanistic-input parameters, such as layer thickness and elastic modulus values. Although indirect, the parameter values measured during QA/QC testing are often assumed to be surrogates to mechanistic parameters, but the relationships are complex, nonunique, and highly variable. A disconnect therefore exists between what is selected for design and the parameter values chosen to “ensure” quality during the construction process. Further, the spatial nonuniformity of the pavement foundation layers, although often recognized as “key” to pavement performance, is not addressed by construction QA/QC or pavement design and rarely in pavement analysis. This paper highlights some of the assumptions with selecting pavement design values and the methods used for construction QA/QC testing, focusing specifically on geotechnical parameters. Weaknesses with traditional approaches are identified, and new ideas are highlighted that might better link construction quality to the selection of pavement design inputs.

Key words: construction—foundation—pavement—quality control—subgrade

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PROBLEM STATEMENT

Quality pavement foundation layers are essential to achieving excellent pavement performance. In recent years, as truck traffic has greatly increased, the foundation layers have become even more critical to successful pavement performance. Unfortunately, there are still many pavement failures in the United States related to inadequate subbase, natural subgrade, and embankment, which are commonly referred to as foundation layers or roadbed. Recent accelerated pavement testing of concrete pavements has reiterated how the supporting foundation layers and stiffness affect the concrete pavement performance (Cervantes 2009). Factors that contribute to pavement foundation problems are poor construction practices, ineffective Quality Assurance/Quality Control (QA/QC) testing methods and sampling plans, material variability and nonuniformity, unpredictable long-term material behavior, poor verification of material properties during construction, insufficient development of performance-related specifications, and lowcapital investment in the foundation layers. This paper compares some of the assumptions used to select pavement design inputs and the methods used for construction QA/QC testing, focusing specifically on the geotechnical parameters.

Pavement designs are typically completed during the project design phase and are based on the data gathered from the geotechnical site investigation. The designer is required to develop foundation layer parameters based on limited information contained in the soil boring logs. Additionally, the designer has to estimate the material properties of any future fill materials based on the proposed project specifications. The pavement designer is commonly not involved in the construction QA/QC team. The QA/QC testing completed during construction is often insufficient to provide an acceptable level of reliability with regard to uniformity of the pavement foundation layer. A disconnect therefore exists between what is selected for design and what is ultimately constructed. In this paper, QA/QC testing methods that have the potential to bridge the gap between field measurements during construction and the original design inputs are highlighted.

FOUNDATION LAYERS FOR LONG LASTING PAVEMENTS

All pavements are ultimately supported by foundation layer soils. The three essential components to a high-performing pavement structure are illustrated in Figure 1 (Texas DOT 2008). If any one of the three is deficient, a poor performing pavement is likely to result. Pavement design procedures and some of the current deficiencies with the QA/QC system will be briefly introduced in subsequent sections. Although quality materials are an essential component of the pavement system, they will not be discussed in this paper.

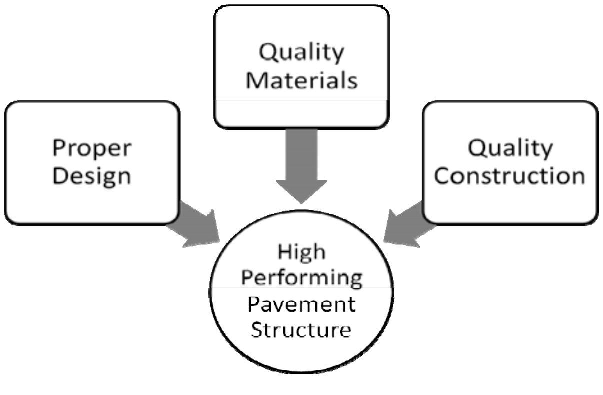


Figure 1. Essential components of a high performing pavement structure

FOUNDATION LAYERS IN PAVEMENT DESIGN

The *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, part 2 "Design Inputs," chapter 1 "Subgrade/Foundation Inputs" states, "It is advisable to use caution when selecting a design subgrade value for a non-homogeneous subgrade" (ARA 2004). Information gaps with regard to foundation soils are also mentioned in the interim *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* (AASHTO 2008). Statements from the interim guide include "Variation along a project creates a much more difficult task to obtain the appropriate inputs for a project" and "The number of samples that need to be included in the test program is always the difficult question to answer."

Pavement Foundation Analysis

The *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, part 2 "Design Inputs," chapter 1 "Subgrade/Foundation Inputs" (ARA 2004) states, "Pavement design requires a single subgrade design value, for example California Bearing Ratio (CB), Resilient Modulus (M_R), or modulus of subgrade reaction (k-value)." The type and thickness of pavement layers developed during design are based on strength tests or various empirical correlations based on tests from the laboratory prior to construction or from tests during construction. Common practice in the United States is to design pavement layers prior to construction and to provide specifications during construction that will correlate to the pavement foundation inputs considered in design. Subsequently, QA/QC practices in the field during construction are necessary to "verify" the original design assumptions.

The stiffness of the foundation layer is often considered more critical to asphalt pavement design than PCC pavement design. A stiffer subgrade in an asphalt pavement design can significantly reduce the thickness of the asphalt pavement. In PCC pavement design, a modulus of subgrade reaction from 27 MPa/m (100 psi/in) to 136 MPa/m (500 psi/in) will only reduce the concrete pavement thickness by approximately 20 percent (ACPA 2007). Therefore, pavement engineers have purported the concept that PCC pavements effectively bridge relatively weak subgrades as long as the support conditions are "uniform."

Problems with Typical Pavement Foundation Design

Pavement design requires assumptions be made to select the foundation input values. Most projects have differing materials that require the designer to use either an average, 85% greater than, or minimum design strength for the foundation soils. Many different methods and correlations exist to estimate the stiffness of the foundation layer soils. For example, the Mississippi Department of Transportation utilizes a correlation from the American Association of State Highway and Transportation Officials (AASHTO) Soil Classification System and Group Number to estimate the California Bearing Ratio (CBR). Another common correlation, presented below, relates M_R (psi) to the CBR. The constant of 1,500 may vary from 750 to 3,000 (Huang 1993).

$$M_R = 1500*CBR \tag{1}$$

After selection of the single pavement foundation parameter for design, the next step is to verify the design assumption in the field during construction. Present day field testing procedures are not capable of easily verifying the design parameter due to the fact that stiffness is difficult to rapidly measure. Further, the value measured during construction may not have any significance to the final pavement design since the foundation soils will often change strength parameters after constructingthe pavement and experiencing changes in the moisture content (Griffiths and Thom 2007).

The *Mechanistic-Empirical Pavement Design Guide* (MEPDG) varies the subgrade stiffness with geographic location and freezing days. A seasonal study in Minnesota quantified the wide range of foundation soil stiffness. A frozen resilient modulus for different types of subgrade layers was found to be in the range of 1,100 MPa (16,000 psi) to 3500 MPa (50,700 psi) during the winter and in the range of 69 MPA (10,000 psi) to 191 MPa (27,700 psi) in the summer (Ovik et al. 1999). Pavement designers attempt to control the effects of frost heave and spring thaw through material specification, soil stabilization, and more recently, with the incorporation of geosynthetics that can provide a moisture barrier.

Due to variable soil, environmental, and construction conditions, determining the appropriate foundation layer parameters for pavement design is challenging. The current pavement engineering design methodologies based on one foundation layer input parameter can simply be arrived at by connecting a few lines on a chart. This oversimplification of the pavement design process has lead to the resistance of implementing new analyses and QA/QC methodologies that more fundamentally model the foundation contributions to pavement behavior (Qubain 2009).

QUALITY ASSURANCE/QUALITY CONTROL (QA/QC) TESTING

The pavement designer attempts to achieve the assumed pavement design input parameters in the field during construction through specification and QA/QC testing. Without an effective QA/QC program, construction practices can overturn the best designs (Qubain 2009). Inadequate observation and QA/QC testing by qualified engineers and technicians have been a major cause of premature pavement deterioration over the years.

QA/QC during Construction

Over the years, the primary foundation QA/QC practice has consisted of moisture and density control (Yoder 1959). The moisture and density requirements are specified in the contract documents, and field personnel typically utilize either the sand cone method or the nuclear gauge method to determine the in situ moisture and density of the soil. Although some specific projects require some form of stiffness test, most highway and commercial work QA/QC programs are based on soil classification, percent relative compaction, and moisture content. In determining the percent relative compaction, the typical laboratory method consists of either the Standard Effort or Modified Effort Compaction Test (AASHTO T99 or 180). Some departments of transportation have specialized procedures for determining the maximum unit weights and optimum moisture contents. A typical specification requires a testing frequency of approximately one moisture and density test per lift for every 2,500 to 10,000 square feet of area, which is approximately equal to every 100 to 500 feet along a highway embankment. Often, the sampling frequency on a per volume basis decreases with increasing volume fill placement. Frequency of QA/QC testing is unique to every project. Compaction tests are typically taken on samples daily or when the material changes in classification.

Potential Problems with QA/QC during Construction

Fifty years ago, Yoder (1959) stated, “Ideally, the field control should be based upon a strength test.” The design methodologies discussed previously all require some type of stiffness input for design. Current practice utilizes moisture and density testing during construction. A single moisture and density test evaluates approximately one square foot based, and therefore, QA/QC testing is performed on about 0.01% of the constructed material. The test spacing is so infrequent that a geospatial analysis of the data and uniformity analysis cannot be completed. Current practice attempts to overcome the shortcomings of the moisture and density control QA/QC method by utilizing the experience and judgment of the soils technician or by specifying proof, rolling the construction area with a loaded vehicle. These methods do not always provide the level of reliability needed to construct long-lasting pavements.

Specifications typically require moisture and density testing during cutting and filling earthwork operations. A common problem in pavement construction is stable pavement layers are rutted and loosened from construction traffic and then not recompacted prior to placing the pavement layers. Specifications typically do not explicitly state to scarify and recompact the subgrade immediately before placement of the subbase or subsequent pavement layers. Improved specifications and trained construction personnel are two areas that could provide improvement in the construction process.

Traditional QA/QC moisture and density testing need to be supplemented with methods that will provide continuity between the design, construction, and laboratory testing. The QA/QC method of moisture and density control during earthwork, which has been the standard for over 50 years, must be improved to provide an acceptable level of reliability for pavements. These supplemental methods will provide the link so that performance-based specifications can be implemented (Nazarian and Correia 2009).

NONUNIFORMITY OF PAVEMENT FOUNDATION LAYERS

Although a significant national effort is going into pavement assessment and performance prediction, limited effort has focused on the as constructed, nonuniformity of pavement foundation layers. Improved pavement design methods do not fix problems with poor construction and QA/QC testing practices and do not adequately address the resulting nonuniformity that can be directly tied to pavement performance.

According to National Cooperative Highway Research Program (NCHRP) Report 583 (2007), “The best-performing pavements…were those with bases that were neither too weak (untreated aggregate) nor too stiff (lean concrete).” The American Concrete Pavement Association (ACPA 2007) reports that “…low strength soils where construction methods provide reasonably uniform support perform better than stronger soils lacking uniformity.” Stiffness, strength, and uniformity are clearly engineering parameters that affect performance, yet only limited specifications and protocols have been developed for construction, testing, and evaluation to verify achievement of these parameters.

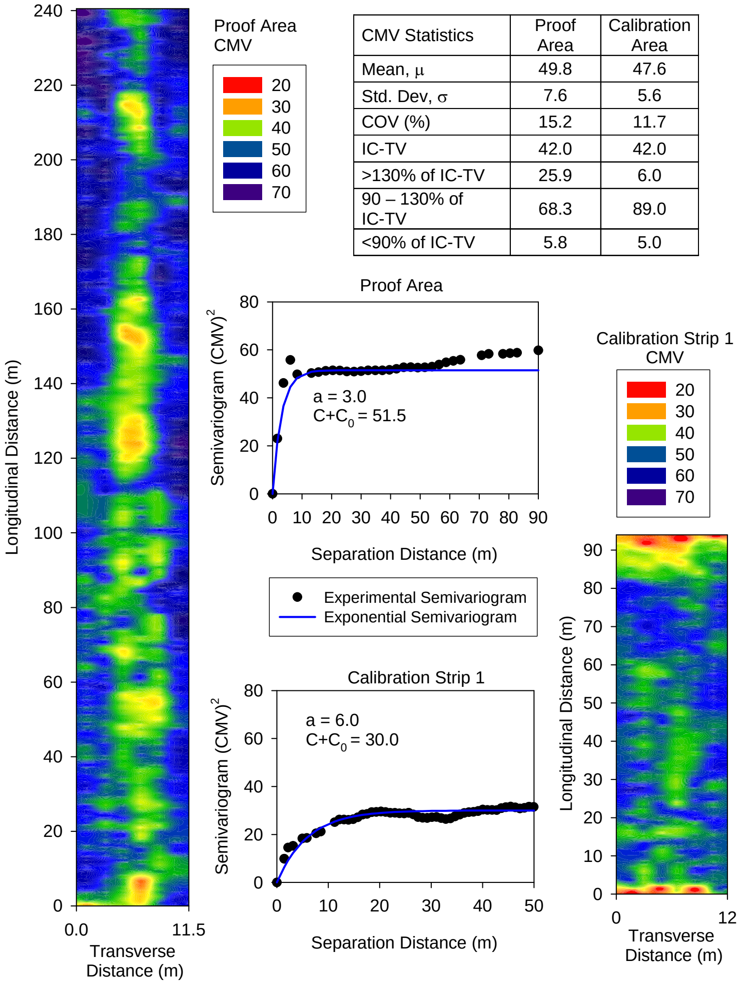
Nonuniformity is the result of either changing materials on a jobsite or poor quality construction practices. In order to mitigate the nonuniformity of as constructed foundation layers, trained construction and QA/QC personnel are required to ensure the intended pavement design parameters are achieved during construction. The disconnect between the pavement designers and the construction process has been discussed previously. Another disconnect is found when the construction and QA/QC personnel do not understand the concept of uniformity, how the construction process impacts the performance of the pavement system, and do not have the tools to characterize uniformity.

EMERGING TOOLS FOR QA/QC TESTING

Beyond proper training of construction and QA/QC personnel, new QA/QC testing tools that can measure the stiffness of the pavement foundation layers more frequently and rapidly must be incorporated. New QA/QC tools must be capable of providing an acceptable level of reliability while also providing an increase in the number of test points in relation to the present system of moisture and density control. Implementation of new construction, testing, and characterization technologies, including intelligent compaction and rapid non-destructive testing methods, have the potential to improve selection of foundation materials, characterization of performance-related engineering properties, and development of construction specifications with meaningful QA/QC testing. Fundamental to the QA/QC testing is that the measured foundation layer properties must be linked with the selected pavement design inputs.

Intelligent Compaction

Intelligent compaction technology uses accelerometers installed on the drum of a vibratory roller to measure roller drum accelerations in response to soil behavior during compaction operations. The use of machine drive power (MDP) as a measure of soil compaction is a concept originating from study of vehicle-terrain interaction. MDP, which relates to the soil properties controlling drum sinkage, uses the concepts of rolling resistance and sinkage to determine the stresses acting on the drum and the energy necessary to overcome the resistance to motion (White et al. 2007). Figure 2 presents an example of subgrade nonuniformity based on intelligent compaction maps with Compaction Meter Value (CMV) measurements.



CMV Statistics	Proof Area	Calibration Area
Mean, μ	49.8	47.6
Std. Dev, σ	7.6	5.6
COV (%)	15.2	11.7
IC-TV	42.0	42.0
>130% of IC-TV	25.9	6.0
90 – 130% of IC-TV	68.3	89.0
<90% of IC-TV	5.8	5.0

Figure 2. Nonuniformity observed in pavement foundation based on intelligent compaction measurements (from White et al. 2007)

Dynamic Cone Penetrometer

The dynamic cone penetrometer (DCP), shown in Figure 3, is a testing device that provides the stability characteristics of pavement layers. The test involves dropping an 8 kg hammer 575 mm and measuring the penetration rate of a 20 mm diameter cone. Penetration index, which typically has units of mm per blow, is inversely related to penetration resistance (i.e., soil strength). The DCP test has been correlated to CBR as presented in ASTM D 6951-03.

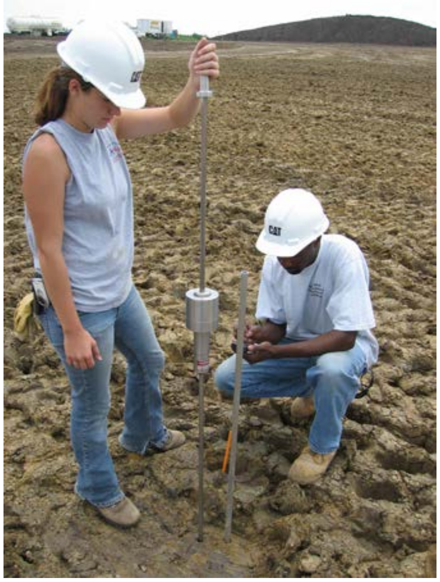


Figure 3. Strength determination using dynamic cone penetrometer (from White et al. 2007)

Clegg Impact Hammer

Clegg impact hammers, which were developed by Clegg during the late 1970’s and later standardized as ASTM D 5874-02 for evaluating compacted fill and pavement materials, are shown in Figure 4. The Clegg impact value is derived from the peak deceleration of a 4.5 kg or 20 kg hammer free falling 450 mm in a guide sleeve for four consecutive drops. Clegg impact values (CIV 4.5 kg or CIV 20 kg) have been correlated to CBR (Clegg 1986, White et al. 2007).

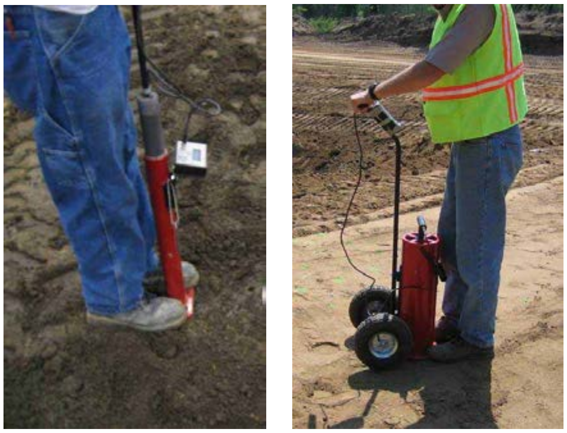


Figure 4. Strength determination using Clegg impact testers: 4.5-kg (left)and 20-kg (right) (from White et al. 2007)

Falling Weight and Light Weight Deflectometers

Falling weight deflectometers (FWD) and lightweight deflectometers (LWDs) can also be used to determine pavement layer stiffness. In performing the tests, a known weight is dropped to produce a dynamic load on a plate. The load and deflection are recorded during the test. From load and deflection data, a soil layer modulus can be calculated. One type of FWD is shown in Figure 5. Two different models of LWDs are presented in Figure 6 (White et al. 2007).

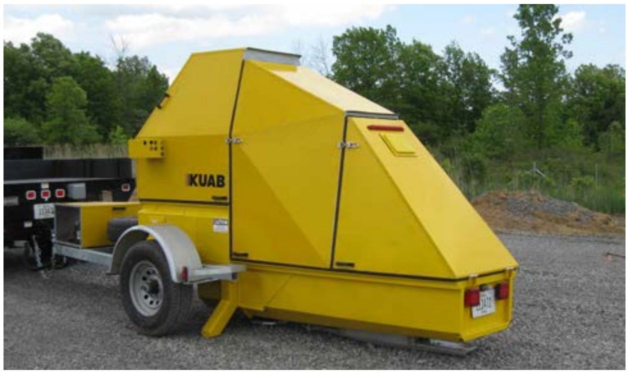


Figure 5. KUAB model FWD

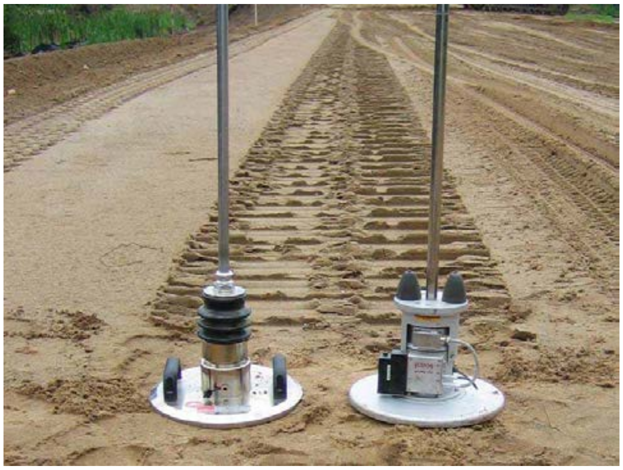


Figure 6. 300 mm LWDs: Zorn ZFG (left) and Keros (right) (from White et al. 2007)

CONCLUSIONS

Proper structural design, quality materials, and quality construction are the three essential components of a long-lasting pavement. With increased traffic loads and owner's desires to increase the life span of pavement structures, the current process of foundation layer construction and QA/QC must be improved. The current design-construction process has a mutual disconnect, with the pavement designer often being absent from the construction process and the construction and QA/QC personnel not fully comprehending the impact of field decisions on the long-term performance of the pavement. Owners should recognize that increased capital investment in the foundation layers is required to construct high-performing pavements.

Selecting design inputs for the foundation layers at first appears to be as simple as determining a representative stiffness from a correlation. However, the actual foundation layer stiffness parameters will change with moisture conditions and environment and will also be influenced by the quality of construction and drainage. Further, QA/QC procedures utilizing moisture and density control at widely spaced intervals do not provide direct measurement of the design parameters and the widely spaced intervals generally do not provide an acceptable level of reliability The process of excavation, compaction, and backfill in tandem with QA/QC from real-time measurements from the compaction equipment or rapid in situ testing will provide the degree of reliability required to construct long-lasting pavements (Lytton and Masad 2009). In situ testing devices, such as intelligent compaction, DCP, Clegg Impact Hammer, FWD, and LWD, are presently being studied in order to develop supplemental testing methods in addition to the method of moisture and density control. Griffiths and Thom (2007) state, "Confidence in design is an abstract concept, which is controlled by construction practice as much as it is by calculation and specification."

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A Large-Scale Traffic Simulation Model for Hurricane Evacuation of Hampton Roads, Virginia

Praveen Edara
Department of Civil and Environmental Engineering
University of Missouri, Columbia
E 3502 Lafferre Hall
Columbia, Missouri 65211
edarap@missouri.edu

EXTENDED ABSTRACT

Many coastal states in the United States have an evacuation plan to deal with the mass evacuation of people in the event of a hurricane. For Virginia, the hurricane evacuation study (HES) is in the form of an abbreviated transportation model (ATM) (U.S. Army Corps of Engineers 1992). The ATM determines the number of people (and vehicles) who will evacuate from each geographic zone, based on 2000 U.S. census data, under a hurricane threat within each jurisdiction in the Hampton Roads region of Virginia. An emergency evacuation plan has been developed for the Hampton Roads region of Virginia intended to facilitate the outbound movement of large numbers of people in vehicles from the region facing a hurricane threat.

A traffic control plan (TCP) (Virginia DOT 2001) has been developed as a part of the overall plan to provide detailed information on the procedures to be followed during the evacuation of traffic in the event of an approaching hurricane. The intent of the plan is to provide the most efficient movement of vehicles out of the region. The TCP uses a phased approach to ensure that those most at risk are given the opportunity to leave the region first. Phase 1 evacuation, which is assumed to take place 24 to 14 hours prior to the onset of hurricane, includes evacuating people from Virginia Beach, Norfolk, York, Poquoson, and parts of Hampton. Phase 2 evacuation, which takes place beginning 14 hours prior to and ending with the first contact of hurricane with the mainland, evacuates Portsmouth, Chesapeake, Suffolk, Newport News, and the remainder of Hampton.

The first attempt to analyze the TCP at a microscopic level was conducted by McGhee and Grimes (2006). Their study investigated the operational performance of only the interstate evacuation routes, namely, I-64, I-264, and I-664, as described in the TCP. The current study was undertaken to evaluate the TCP for all the evacuation routes—Interstate routes (I-64, I-264, and I-664) and arterial routes (Rt. 58, Rt. 460, Rt. 60, Rt. 17, and Rt. 10), including the roadways that carry the traffic up to these evacuation routes. The study area comprised of the following nine evacuation areas: cities of Virginia Beach, Norfolk, Chesapeake, Portsmouth, Suffolk, Hampton, Newport News, Poquoson, and York.

VISSIM (PTV 2007), a state-of-the-art microscopic, time-step and behavior-based simulation model was used to model the traffic operations during an evacuation. The geometrics of the evacuation road network were coded in VISSIM. The road network being studied in this research has a total length of approximately 2,000 miles (includes all evacuation routes and cross streets), and hence, manual coding of the network was not an option. A more efficient method to code the network was to use geographic maps in conjunction with VISUM software. The created network was then exported into VISSIM simulation tool. Traffic signal timing plan information was obtained from all the cities in the Hampton roads region and from Virginia DOT for state-maintained signals. There were 72 traffic-actuated signals (NEMA) and

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120 fixed-time signals in all in the entire network. Also, the ramp meters were coded using a fixed-time or a NEMA control depending upon the information from the TCP. Some of the ramp meters had different metering rates for the two phases of evacuation, and hence, a NEMA controller was used to code such an arrangement.

The temporal distribution of the total vehicle trips identified in the ATM for each origin is determined based on an earlier study (McGhee and Grimes 2006). The distribution lies between the COE response curves for slow and medium response from the public to the evacuation order. For Phase 1, 10% of the total volume evacuated are during the first 5 hours, 60% during the next 5 hours, 20% during the next 4 hours, and 10% during the last 10 hours. For Phase 2, traffic does not start to evacuate until the 14th hour. From the 14th to the 18th hour, 60% of the traffic evacuates, and the remaining 40% evacuates from the 18th to 24th hours. In VISSIM, these trips were coded in two steps. First, vehicle inputs were defined for each origin. There were 207 such inputs. The task of routing vehicles from each origin to the assigned evacuation route was performed manually by determining the set of preferred routes from an origin to each evacuation route. Preferred routes (to the evacuation routes) were established based on the shortest distance, geometry, and lane configuration of the roads and the expected traffic flows. The desired routes of all O-D flows were then defined in VISSIM as routing decisions. Due to the large size of the road network, this task of routing involved considerable amount of person hours as compared to the other aspects of the study. There were six evacuee scenarios obtained from the ATM, which reflected on the combinations of storm intensity classified based on Saffir-Simpson scale and hotel occupancy (L for low occupancy, H for high).

The base case simulation model corresponds to the conditions when a lane reversal is not ordered. The model was constructed based on the information from the TCP and the ATM. The base case was modeled for all storm categories (1 to 4) and hotel occupancies. This was done to compare the potential benefits of ordering a lane reversal and for identifying the storm category and scenario for which it would be warranted. The lane reversal case was modeled with the eastbound lanes of I-64 also going westbound. Data collection points were deployed in the simulation models to measure traffic characteristics at several critical locations on all evacuation routes and elsewhere in the network. Vehicle throughput (total number of vehicles), evacuation time, and average speeds of vehicles successfully evacuating the region were obtained. Additional data collection points were deployed on an imaginary screen line that cuts across all evacuation routes. The screen line was at a location before which most of the travel demand for the evacuation routes has entered the routes. Section measures such as travel time sections and queue counters were also added to the model.

Based on simulation results, the following conclusions were drawn:

1. For a Category 4 or higher storm, a lane reversal should always be implemented to achieve the best traffic performance.
2. For a Category 4 storm with high hotel occupancy, almost all vehicles (99%) were able to exit the network by the end of 27 hours.
3. Traffic demand on I-64 exceeds capacity for the section between I-64/Fort Eustis Blvd and I-64/I-295 interchanges. This significantly reduces the speed of evacuating vehicles that are able to leave the region.
4. Simulation results show that the reversed lanes have the potential to carry more vehicles than are currently assigned to them.
5. For a Category 3 storm, the throughput values for different evacuation routes are nearly the same with or without lane reversal; however, the travel times on few routes are considerably better if lane reversal is employed.

6. For Category 1 and 2 storms, the throughput on all evacuation routes was equal to the demand and the travel times were also consistent throughout the evacuation.

Key words: abbreviated transportation model—hurricane evacuation study—traffic control plan

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Construction Project Administration and Management for Mitigating Work Zone Crashes and Fatalities: An Integrated Risk Management Model

Daniel Enz
Department of Civil, Construction, and Environmental Engineering
Iowa State University
405 Town Engineering
Ames, IA 50011
enz@iastate.edu

Kelly Strong
Department of Civil, Construction, and Environmental Engineering
Iowa State University
454 Town Engineering
Ames, IA 50011
kstrong@iastate.edu

Jennifer Shane
Department of Civil, Construction, and Environmental Engineering
Iowa State University
498 Town Engineering
Ames, IA 50011
jsshane@iastate.edu

ABSTRACT

The goal of the research is to mitigate the risk of highway accidents (crashes) and fatalities in work zones. The approach of this research has been to address the mitigation of work zone crashes through the creation of a formal risk management model to be utilized during the construction management and administration of highway projects for all stages of the project life cycle. The result of these efforts is realized through the design of an integrated risk management model.

A standard risk management model has three components: risk identification, risk analysis, and risk response. The risks are identified by the factors that contribute to work zone crashes. The analysis of risk deals with understanding the tendency of a hazard to influence the frequency or severity of a loss, and the risk response relates to the appropriate countermeasures to the factors that contribute to work zone crashes. The number of hazards and mitigation strategies can be substantial.

The intent of this research is to develop a check list for the risk management team along with establishing scenario-based questions that will accompany the brainstorming sessions. These scenario-based questions are based on the established proximate cause (loss of control, loss of visibility, and confusion) to identify potential hazards on the plans, designs, or jobsite. The scenario-based questions that can cue the risk response that deals with mitigation strategies may possibly take the form of a mitigation "method" (alert motorist, assist worker/motorist, control motorist, inform motorist, and protect worker/motorist).

The results of this research will be a formal step-by-step methodology to be utilized by managers and decision makers. Each stage of the project life cycle (or project development process) will provide a checklist of hazards and mitigation strategies. This research will also provide a qualitative method to assess the likelihood and severity that a hazard or multiple hazards would pose on a roadway work zone.

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This research is intended to provide a holistic approach to risk management that is to be integrated into the existing corporate structure and not to be considered a stand-alone program. This integrated approach will allow a formalized procedure to be utilized by any member of an organization during all phases of the construction project life cycle.

Key words: project life cycle—project risk management—work zones

Modified Sheet Pile Abutment for Low-Volume Road Bridge

Ryan R. Evans
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
176 Town Engineering
Ames, IA 50011
ryevans@iastate.edu

David J. White
Earthworks Engineering Research Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
djwhite@iastate.edu

F. Wayne Klaiber
Bridge Engineering Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
klaiber@iastate.edu

Terry Wipf
Bridge Engineering Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
tjwipf@iastate.edu

Caleb Douglas
Earthworks Engineering Research Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
calebd@iastate.edu

ABSTRACT

Steel sheet piling, typically used for retaining structures in the United States, is a potential alternative for use as the primary component in low-volume road (LVR) bridge substructures. To investigate the viability of sheet pile abutments, a demonstration project was performed in Black Hawk County, Iowa. The project involved construction of a 40 ft single-span bridge utilizing axially loaded steel sheet piling as the primary foundation component. The site selected for the project had primarily silty clays underlain by shallow bedrock into which the sheet piling was driven. An instrumentation system (consisting of strain gages, deflection transducers, earth pressure cells, and piezometers) was installed on the bridge for obtaining service load test data as well as long-term performance data. This paper presents documentation of the design and construction of the demonstration bridge in Black Hawk County as well as an analysis of the design procedures used through information collected during load testing. Preliminary results indicate that steel sheet piling is an effective alternative for LVR substructures, and future demonstration projects are planned to investigate different sheet pile abutment alternatives for varying site conditions.

Key words: abutments—bridge—foundations—pile—sheet

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INTRODUCTION

Based on National Bridge Inventory data, 22% of the low-volume road (LVR) bridges in Iowa are structurally deficient, while 5% of them are functionally obsolete (Federal Highway Administration 2008). The substructure components (abutment and foundation elements) are known to be contributing factors for some of these poor ratings. In addition to timber piling, steel H-piling, and reinforced or prestressed concrete piling, steel sheet piling has been identified as a viable long-term option for LVR bridge substructures but needs investigation with regard to vertical and lateral load resistance, construction methods, design methodology, and long-term performance.

Iowa Highway Research Board project TR-568 was initiated in January 2007 to investigate the use of sheet pile abutments. A total of 14 different candidate sites were investigated in several counties. Three sites were selected based on site conditions for demonstration projects and are located in Black Hawk, Boone, and Tama Counties. Each of the demonstration projects utilizes a different experimental abutment system. This paper presents a case history of the demonstration project in Black Hawk County (BHC), which was completed in October 2008. The remaining demonstration projects are to be constructed, instrumented, and tested in the summer of 2009.

PROJECT DETAILS

The demonstration project in BHC was constructed at a site crossing a small creek on Bryan Road near La Porte City. This project was constructed to investigate the feasibility of axially loaded sheet piling for use as the primary foundation element for the bridge structure. The replacement bridge was completed by the end of October 2008. In coordination with the Iowa Department of Transportation, a load test of the bridge was subsequently performed using loaded trucks and data were collected and analyzed by Iowa State University (ISU). The following sections give an overview of the previous structure and the replacement bridge.

Old Bridge Structure

The structure that was replaced was a 40 ft single-span pony truss bridge supported on a timber pile foundation (see Figure 1). The bridge was approximately 10 ft above stream level. The structure was retrofitted at some point with steel H-piles, one per abutment. These retrofit piles are believed to have been driven into the existing shallow bedrock (about 8 ft below stream level) to provide reinforcement for the timber abutments; one of these piles can be seen in Figure 1.

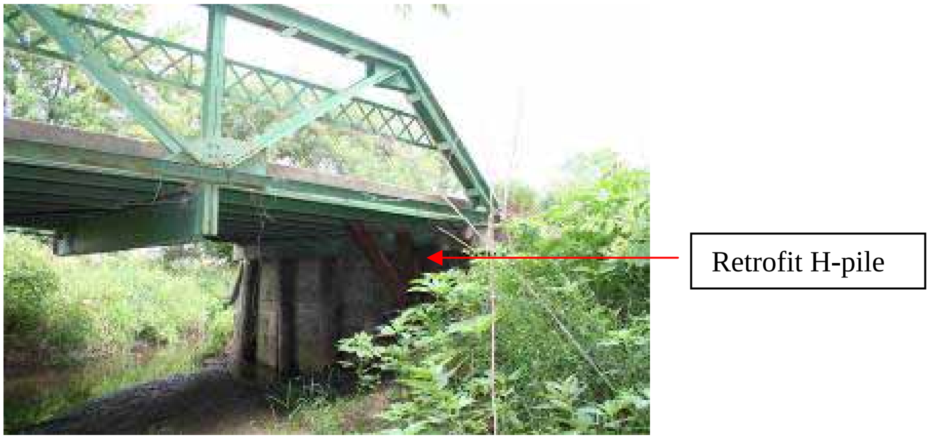


Figure 1. Previous bridge in BHC with retrofit pile shown

Replacement Bridge Overview

The replacement bridge (a two-lane 40 ft single-span beam-in-slab bridge) was a joint design effort between BHC and ISU. The design of the superstructure was performed by the BHC engineering department and utilized precast elements previously developed. The substructure, which was primarily designed by ISU, utilized axially loaded sheet piling as the bearing component of the foundation.

Superstructure

BHC used custom precast beam-in-slab units for the bridge superstructure. Each unit contained two W14x61steel beams. A total of six units were required for the bridge, each unit spanning the entire length. Between each unit there was a cast-in-place joint that was poured after the units were placed. Figure 2 shows a profile of the bridge deck and abutment.

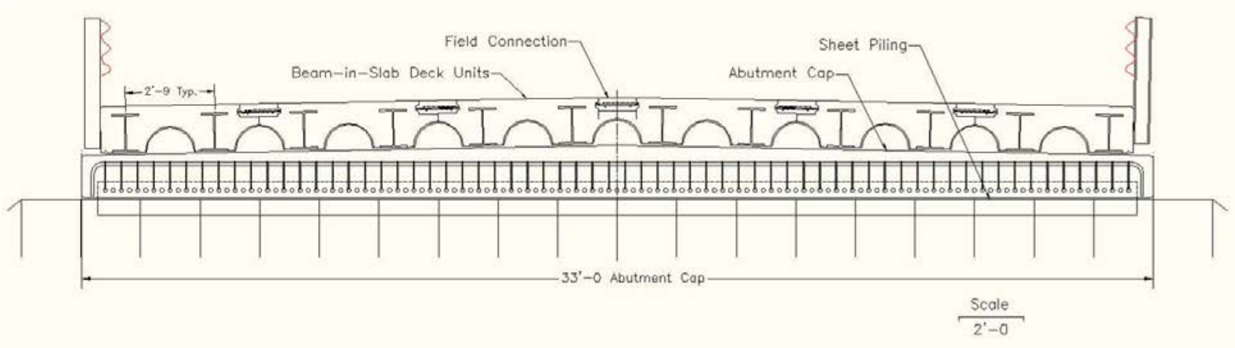


Figure 2. Cross section of replacement bridge deck

Substructure

Steel sheet piling was used for the foundation of the bridge structure. Each abutment consisted of a precast abutment cap bearing on sheet pile sections driven into shallow bedrock. A total of 64 PZ-22 sheet pile sections were required in each abutment.

The abutment cap was a precast element designed by BHC that consisted of a W12x65 steel beam cast in reinforced concrete. The web of the steel beam cast in the abutment cap beared directly on top of the driven sheet piling with no attachment between them. The bridge deck units were placed on the abutment cap using bearing pads between the deck units and the reinforced concrete abutment cap. Figure 3 shows a cross section of the sheet pile abutment.

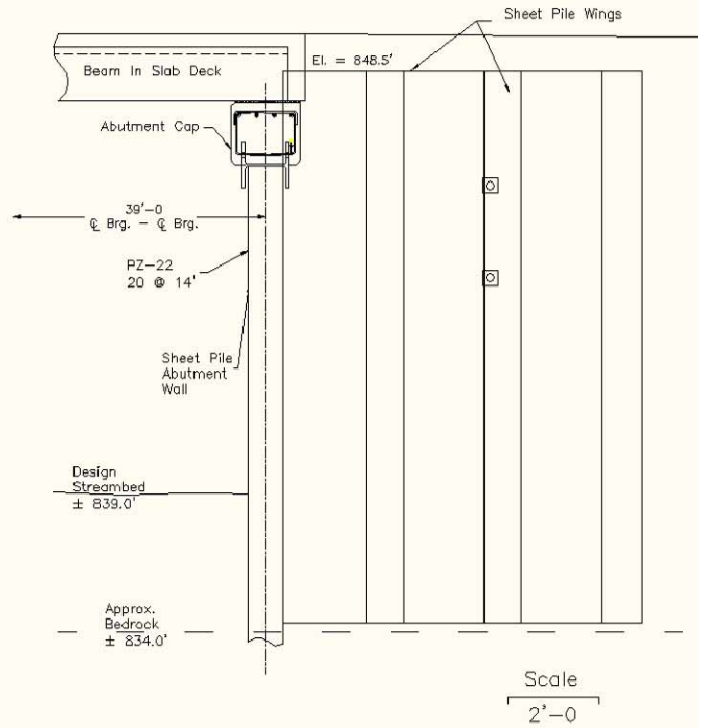
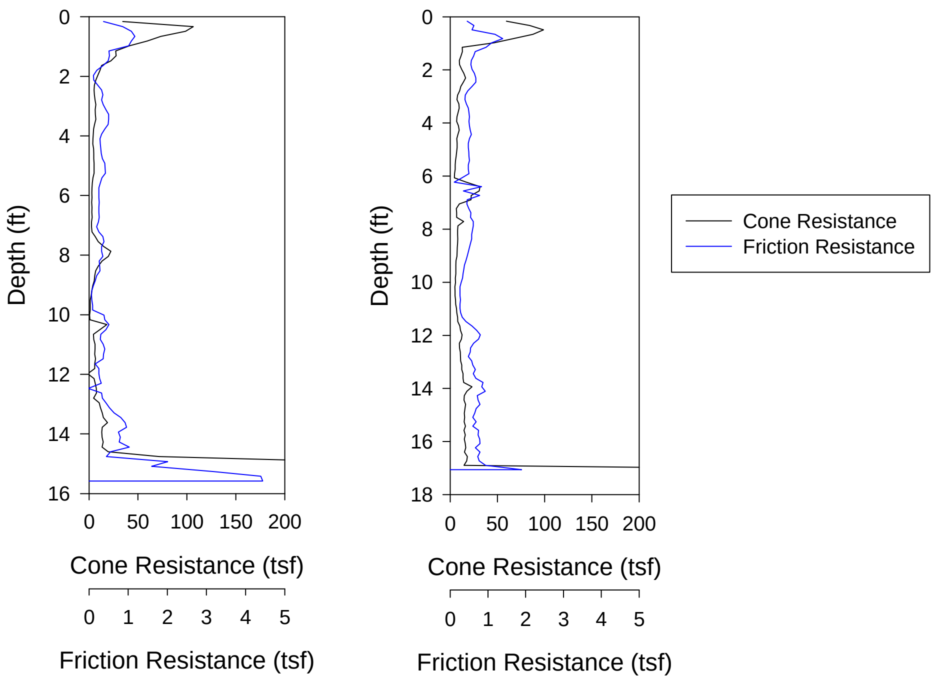

Sheet Pile Wings
El. = 848.5'
Beam In Slab Deck
Abutment Cap
39'–0
℄ Brg. – ℄ Brg.
PZ–22
20 @ 14'
Sheet Pile
Abutment
Wall
Design
Streambed
± 839.0'
Approx.
Bedrock
± 834.0'
Scale
2'–0

Figure 3. Cross-section of sheet pile abutment

SITE INVESTIGATION

Before designing the abutments, a subsurface investigation was performed that involved cone penetrometer testing (CPT) as well as laboratory analysis of soil borings. CPTs were performed at each abutment and provided subsurface profiles (see Figure 4) that showed predominately clay with sand seams and bedrock at approximately 15 ft and 17 ft depths for the east and west abutments, respectively. The soil borings (also performed at each abutment) involved drilling and sampling of the soil at various depths for analysis of strength and other characteristics.



(east abutment) (west abutment)

Figure 4. CPT results for BHC bridge site

DESIGN AND CONSTRUCTION

Design of Abutments

The design of the substructure was performed according to the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor (LRFD) Bridge Design Specification (1998). Substructure elements were designed to resist HL-93 loading. The 40 ft bridge was loaded with the design truck and lane load as per AASHTO (1998) Section 3.6.1.2 in order to determine live loads that needed to be resisted by the abutment. The design loads were determined using the critical load factors and load combinations in AASHTO (1998) Section 3.4. Although unused in this design (due to the presence of shallow bedrock), the Steel Construction Institute (1998) provides a design methodology for axially loaded sheet pile abutments that derive their bearing capacity through soil friction. Further design considerations for sheet piling are outlined by the American Society of Civil Engineers (1996).

Sheet Pile Wall Design

Due to the nature of the loading, the sheet pile sections were analyzed as beam columns. The combination of piling being driven into bedrock and restraint provided by wing walls was assumed to prevent translation at the base of the wall (but not rotation). Once in place, the bridge superstructure was assumed to provide restraint against translation at the top of the wall, and thus, the design element was assumed to be simply supported at both ends of the section.

Loads from the retained soil and surcharge were applied laterally to the element. For determining the transfer of vertical pressure to the wall, at-rest conditions were assumed due to the effect of the bridge structure in resisting lateral displacement at the top of the wall. The design parameters used for the backfill soil were a friction angle of 30˚, cohesion of 0 psf, and a unit weight of 125 pcf. For the underlying clay layer, the parameters used were a friction angle of 0˚, cohesion of 500 psf, and a saturated unit weight of 140 pcf.

Vehicular live loads on the retained soil were accounted for in design by using the equivalent surcharge loading outlined in AASHTO (1998) Section 3.11.6.2. Design axial loads in the piling were determined by assuming superstructure dead loads were distributed evenly amongst all piles and live loads were distributed over a 10 ft wide lane. As previously stated, the pile section required for the wall was the PZ-22.

The final design of the abutments required a total of 64 (32 per abutment), 15 ft PZ-22 piles (Grade 50 steel). Based on a market price of $20.80 per square foot of wall, the total cost of the sheet piling was approximately $36,600. The abutment caps were set directly on the top of the sheet pile wall after it was finished to grade. As stated previously, the superstructure was assumed to provide adequate lateral restraint once in place. During backfilling of the abutments, however, this was not the case. Because of the lack of lateral restraint, a reinforced concrete deadman anchor system was installed on each abutment. The system was designed by BHC and consists of a reinforced concrete deadman (approximately 8 x 3 x 2 ft) with two, 1 in. diameter tie rods connected to a waler channel on the exterior face of the abutment walls.

Construction of the Bridge

BHC used its own crew for construction of the entire project. The bridge crew consisted primarily of three construction workers employed each day that work was being done on the project. According to the BHC engineer, average labor costs amount to approximately $1,000 each day the bridge crew is on-site.

The total time required for construction of the replacement bridge was approximately 10 weeks. The demolition of the existing bridge structure, which was considered the beginning of project construction, began in the fall of 2008. A chronology of major construction events is shown in Table 1.

Table 1. Chronology of significant construction events for BHC project

Event Description	Start Date
Demolition	08/13/08
Sheet pile driving—East Abt.	08/25/08
Sheet pile driving—West Abt.	09/02/08
Abutment finishing	09/17/08
Deck unit placement	10/02/08
Bridge finishing	10/15/08
Open for service	10/20/08

Sheet Pile Driving

After demolition of the majority of the existing structure was completed, pile driving of the sheet pile walls was performed. For the main abutment sheet pile walls, pile driving was completed using both vibratory and impact hammers. The piles, after being placed in a guide rack to help ensure proper wall construction, were initially driven as far as possible by using an excavator equipped with a vibratory plate. The piles were then driven to a 25 ton bearing using a crane equipped with a drop hammer. Bearing was considered to be attained if after five consecutive blows (from a 4,250 lb. hammer dropped from six ft), less than two in. of penetration was observed. Wing walls were driven as deep as possible with the vibratory plate and then trimmed. The wing walls were placed at a 45˚ angle to the main wall using a custom connector.

The guide rack was built to have an opening that was one in. wider than the width of the sheet pile sections used in order to ensure the sheet piles would fit. This was unnecessary as a slightly rotated sheet pile section would be able to fit easily into a guide rack built to exact sheet pile width. A rack of exact width would also have ensured that adjacent piles would be flush with each other. Significant rotation between adjacent sheets occurred. This rotation resulted in extending the actual width of the wall by approximately 1 1/2 ft.

CPT results showed refusal at 15 ft below grade for the east abutment. During impact driving, practical refusal (the 25 ton bearing) was not reached until significantly below what was shown in the CPT results. This was not an issue for the east abutment since pile lengths ordered were longer than necessary.

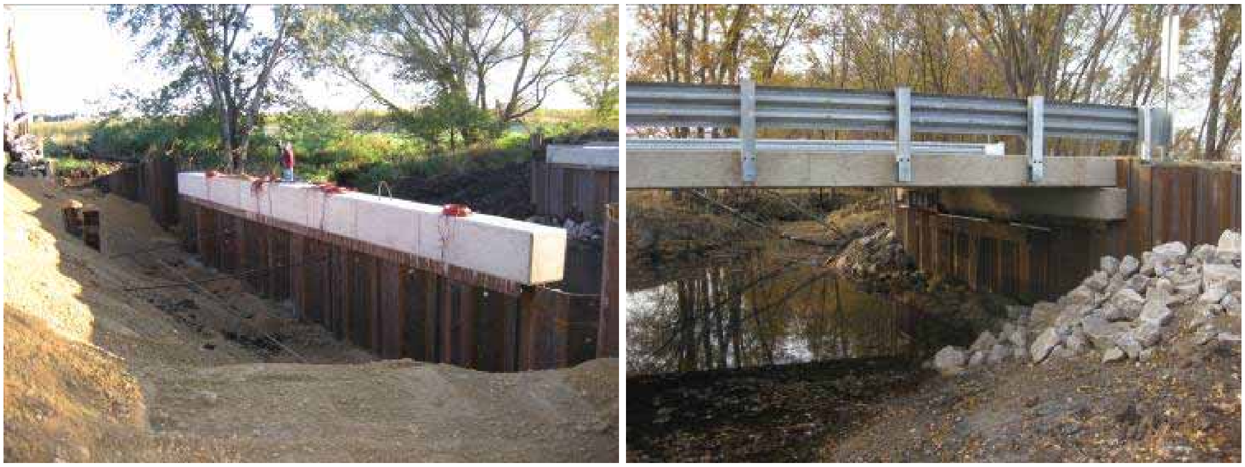
CPT results for the west abutment showed refusal at approximately 17 ft below grade. During the impact driving phase, it became evident that the piling lengths ordered were too short. The depth required for practical refusal on the west side of the stream was significantly lower than that predicted by the CPT results. All of the sheet piles along the main wall required splices to be added in order to achieve design elevations. In some cases, piles needed to be driven more than a foot lower than the adjacent section; this required splices to be made as well in order for the pile driving mechanism to fit in place.

Another issue encountered was fracturing of the pile driving cap. The BHC bridge crew had constructed a custom cap for driving of the sheet piles out of welded plates. After a few hammer blows, the welds would fracture and typically require the remainder of the work day to be redone. This occurred two separate times during construction of the west abutment wall.

Abutment Construction

After all sheet piling had been driven to specified bearing capacity, several tasks were performed to complete the abutments. The major tasks that needed to be performed before backfilling were placement of the subdrain, installation of the anchor system, placement of the abutment cap, and installation of instrumentation.

Before each abutment was backfilled, a layer of rip-rap was placed against the stream side face of the sheet pile walls. Both abutments were backfilled with ¾ in. roadstone within a short zone behind the sheet pile wall (shown in Figure 5a). Outside of these zones, existing material was left in place. On the east abutment, the existing material that was left consisted primarily of soil. On the west abutment, the majority of the abutment from the previous bridge was left in place. Backfill material was primarily used to fill the void between the sheet pile wall and the abutment from the previous bridge. The remainder of construction required placement of guardrails and finished grading of the roadway approaching the bridge. The bridge was opened for service on October 20, 2008. A view of the west abutment of the bridge after completion is shown in Figure 5b.



(a) (b)

Figure 5. (a) Backfilling of west abutment with abutment cap in place, (b) west abutment of completed bridge in BHC

LOAD TESTING

The bridge was instrumented with vibrating wire instruments as well as strain and displacement transducers. The vibrating wire instruments (strain gages, earth pressure cells, and piezometers) were permanently installed at the site and were used for long-term data recording. The strain and displacement transducers were installed for the one-time service load test of the structure.

Instrumentation Installation

Although strain gages were attached to the sheet pile sections before driving, earth pressure cells and tie rod strain gages needed to be installed both before and during the backfilling operations. Tie rod strain gages were attached to each tie rod and protected by welding angle iron sections around them. Earth pressure cells were placed at various depths along the abutment wall and required backfilling operations to be halted several times for placement. For placement of each earth pressure cell, a small trench was made in which the cell was placed and surrounded by fine silica sand and compacted.

Two piezometers were installed on the project to monitor the height of the water table. The instruments were placed at the centerline of the west abutment on opposite sides of the sheet piling (one on the backfill side and one on the stream side).

Bridge Load Testing

The instrumentation and monitoring system was used in conjunction with load tests to investigate the behavior of the structure under loading. The live load test involved driving loaded trucks over the bridge and taking readings when they were in predetermined locations. Axle loads of the test trucks are given in Table 2.

Table 2. Truck axle weights

Load	Truck #48 (lbs)	Truck #38 (lbs)
Front axle	17,460	16,980
Tandem axle	31,360	30,260
Total	48,820	47,240

Due to the unexpectedly high post-construction stress readings in the tie rods on the west abutment, it was decided that a test be performed on the south tie rod to verify the accuracy of the readings. The process of this test was to loosen the tie rod hex nut at specific intervals, taking readings of tie rod stress after each interval.

RESULTS AND ANALYSIS

In order to analyze the results of the bridge testing, expected stresses and deflections were estimated by using design analysis. The bridge test results were compared for the test run of Truck #48 in the south lane of the bridge for the case of the tandem axle load positioned 5 ft from the centerline of the sheet pile wall (putting the front axle load on the bridge). The expected and actual values for various load types are presented in Table 3. The total values are given as well as the values due to the live load test only. For the theoretical analysis, truck loads were assumed to distribute over a 10 ft width of the bridge. For the earth pressure cells listed, their locations are given relative to the top of the abutment cap (TOC).

Table 3. Comparison of actual to expected values for various bridge test results

Load or deflection	Expected		Measured	
	Total	Live load only	Total	Live load only
Pile axial stress	0.8 ksi	0.2 ksi	1.9 ksi	0.04 ksi
Pile flex. stress	9.1 ksi	-	8.7 ksi	-
Earth pressure (1’ below TOC)	440 psf	330 psf	35 psf	10 psf
Earth pressure (3’ below TOC)	500 psf	270 psf	27 psf	0 psf
Midspan flex. stress	-	2.2 ksi	-	1.3 ksi
Midspan defl.	-	0.2 in	-	0 in
Wall defl.	-	0.2 in	-	0 in

From the results in Table 3 it can be seen that earth pressures were significantly lower than estimated. Although more pressure cells were used for the test (all measuring unexpectedly low earth pressures), the cell one ft below the abutment cap showed the highest variation in stress during the live load test (a magnitude of 10 psf). The theoretical and measured loads were compared for the south tie rod as well and the results are given in Table 4. By analysis, the load test truck was expected to yield the tie rods (a stress of 50 ksi). The tie rods, however, were only intended for lateral resistance during abutment construction. Once in place, it was assumed that lateral restraint of the top of the wall would be provided by the superstructure.

Table 4. Comparison of tie rod stress results during construction and testing

	Stress (ksi)		
		Measured	
South tie rod	**Expected**	**During construction**	**During load test**
	50	45	0.5

As can be seen from the live load test results, a negligable amount of stress is developed in the tie rods from the truck loads. Coupled with the minimal wall displacements measured, the assumption of lateral restraint provided by the superstructure is considered accurate. After construction of the abutments, however, it was seen that high stresses had occurred during construction. As previously mentioned, a tie rod test was performed to confirm these readings. The conclusions drawn from the tie rod test were the following:

- The tie rod gage readings were reliable, and thus, unexpectedly high stresses were induced in the tie rods during construction of the abutments.
- Since initial readings in the south tie rod indicated stress levels near the yield stress of the steel, it is possible the tie rods experienced yielding at some point during compaction of the abutment backfill and construction of the bridge superstructure.

Long-term measurement of earth pressure (as well as temperature) in the cells one and three ft below TOC are presented in Figures 6 and 7, respectively. Although the monitoring system was destroyed in a flooding event in the spring of 2009, long-term data was recorded for 80 days after November 20, 2008. For the cell one ft below TOC, variations of earth pressure with time were recorded. In the cell just below it (three ft below TOC) the variation was less. Both cells experienced greater variations in stress during cold temperature cycles (perhaps attributed to stress development from ground freezing in the backfill behind the abutment).

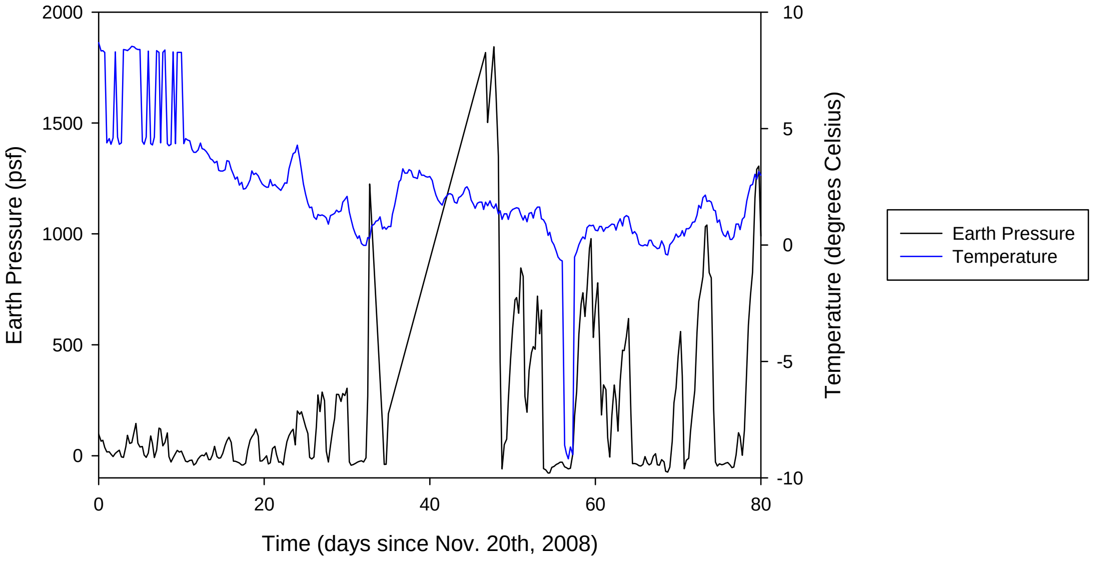


Figure 6. Long-term readings for pressure cell #9489 (located one ft below top of abutment cap)

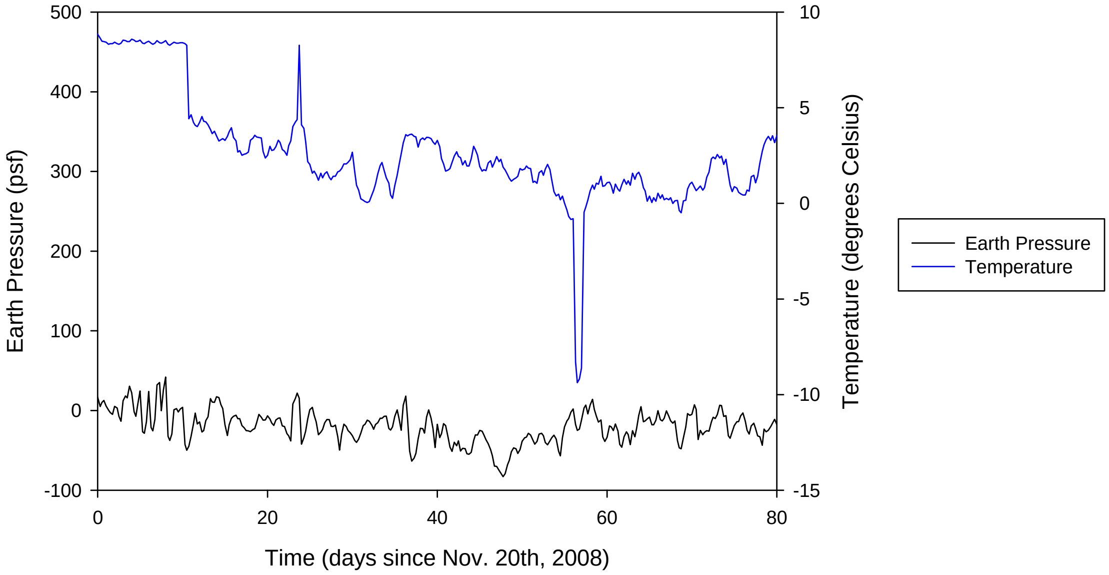


Figure 7. Long-term readings for pressure cell #8503 (located three ft below top of abutment cap)

Long-term groundwater table measurements, given as the distance from the bottom of the bridge deck (at the abutments) to the water level, are shown in Figure 8 for both sides of the abutment. Although the two piezometers measured different levels of groundwater, the offset is constant at about three to four in. (attributable to human error), suggesting that no significant pressure head developed behind the abutment wall.

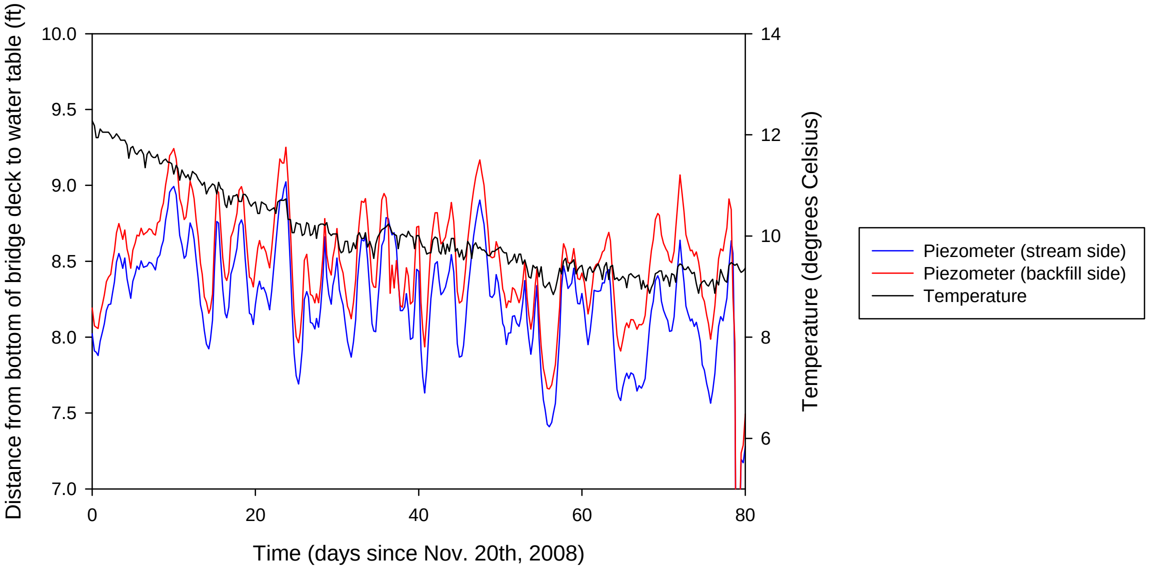


Figure 8. Long-term readings for piezometers (both sides of abutment)

CONCLUSIONS AND RECOMMENDATIONS

Through the construction and structural monitoring of the bridge project, steel sheet piling has been shown to be a feasible alternative for bridge abutments with site conditions similar to BHC. Although the BHC project required approximately 10 weeks for construction, in the future, potential for significant shortening of construction time exists if critical to the project timeline.

Several improvements for the sheet pile abutment system were determined during the project. Although the tie rods were shown to be unnecessary once the bridge is completed, the use of some form of lateral restraint is necessary to resist the loads developed during abutment construction. Tie rods are one alternative and will also provide overall system stability during large lateral loading events that may occur. The use of a forged pile driving cap is another recommendation as significant time and labor was spent repairing the custom-made, welded cap used by BHC.

Although the bridge test results showed significantly lower stresses and deflections than expected, further testing is recommended to determine the nature of earth pressure development behind sheet pile abutments. Two other tests are planned in the summer of 2009 during construction of the other demonstration bridges that are part of project TR-568; results from these tests will provide a more in-depth analysis of earth pressures as well as an investigation into the viability of steel sheet pile abutments for differing site conditions.

ACKNOWLEDGMENTS

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Determining the Effectiveness of Portable Changeable Message Signs in Work Zones

Umar Firman
Department of Civil, Environmental, and Architectural Engineering
University of Kansas
1530 W. 15th Street, 2160 Learned Hall
Lawrence, KS 66045
umar_firman@yahoo.com

Yong Bai
Department of Civil, Environmental, and Architectural Engineering
University of Kansas
1530 W. 15th Street, 2150 Learned Hall
Lawrence, KS 66045
ybai@ku.edu

Yue Li
Department of Civil, Environmental, and Architectural Engineering
University of Kansas
1530 W. 15th Street, 2160 Learned Hall
Lawrence, KS 66045
ylkx7@ku.edu

ABSTRACT

Due to the rising needs in highway maintenance and construction, the number of work zones is increasing nationwide. Highway work zones disrupt normal traffic flow and create safety problems. There were a total of 1,010 fatalities and more than 40,000 injuries in the United States in 2006. Even though there are some countermeasures developed to improve the safety of work zones, there is still a lot of room for improvements. To improve the effectiveness of existing countermeasures and to develop new countermeasures, evaluation of the existing countermeasures is essential. The objective of this research project was to evaluate the effectiveness of a Portable Changeable Message Sign (PCMS) on reducing vehicular speeds in the upstream of rural two-lane highway work zones. This objective was accomplished using field experiments conducted on US 36 located in Seneca, Kansas. During field experiments, the effectiveness of the PCMS was evaluated under two different conditions: (1) PCMS switched on and (2) PCMS switched off. Based on the data analysis results, the PCMS switched on condition reduced the vehicle speeds significantly compared to the PCMS switched off condition. Vehicles with slow speeds approaching a work zone are more likely to reduce the probability of having crashes. The major contribution of this research project was to quantify the effectiveness of PCMS in rural two-lane work zones which had not been studied in detail.

Key words: highway—Kansas—safety sign—work zone

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INTRODUCTION

Across the United States, most highways in the nation's highway system need maintenance; it is inevitable that more work zones will appear on the highways. Work zones on highways create serious disruptions to the normal flow of traffic, resulting in major inconveniences for the traveling public. In the 1960s, some researchers began studying work zone safety (Mohan and Gautam 2002). A significant number of relevant studies have been published that unveil safety problems and propose safety improvements in work zones (Li and Bai 2009). Over the last 10 years, the annual number of persons killed in motor vehicle crashes in work zones has increased by 45% (up to 1,010 in 2006) (FARS 2006). More than 40,000 people are injured each year as a result of motor vehicle crashes in work zones (FARS 2006).

Between 1960 and 1997, the average fatality rate declined at the rate of 3.3% per year per hundred million vehicle miles (hmvm) traveled (Mohan and Gautam 2002). With the growth of technology, the use of new equipment and adapting to new procedures has been implemented to improve the efficiency of work zones, nevertheless accidents still occur in highway work zones. Some factors have been extensively cited as the main causes of traffic crashes in highway work zones that include excessive vehicle speeds, variation of speeds between different vehicles, and driver inattention and erratic maneuvers (Zech et al. 2008). Researchers are most concerned with reducing the speed of traffic in work zones and believe a reduction in speed will ultimately be most effective for reducing crashes and fatalities (Li and Bai 2008a). Recently, work zones have begun to implement static regulatory and advisory signage but are known to be only low in effectiveness in decreasing number of crashes (Fontaine and Carlson 2001). These have led to enhancing the signage system and study the speed control measures. Research shows there are a number of ways to control the speed of vehicles in the work zones. Some examples are (1) police presence, (2) changeable message signs (CMSs), (3) rumble strips, (4) drone radar, (5) radar activated speed trailers, (6) temporary traffic control, (7) increased fines, and (8) detours or diversions. In addition, these examples of controlling the speed of vehicles in work zone could prevent some common human errors, such as overlooked traffic control, followed too closely, and exceeded speed limit or too fast for condition, from causing severe crashes. This paper reports the results of field study conducted on the Kansas rural highway work zone (US 36) to evaluate the effectiveness of Portable Changeable Message Signs (PCMSs) as a speed control measure in two conditions: (1) PCMS was switched on and (2) PCMS was switched off.

LITERATURE REVIEW

According to the *Manual on Uniform Traffic Control Devices* (FHWA 2003) for streets and highways, a Changeable Message Sign (CMS) is a sign that is capable of displaying more than one message, changeable manually, by remote control, or by automatic control. These signs are referred to as Dynamic Message Signs (DMSs) in the National Intelligent Transportation Systems (ITS) Architecture. DMSs are common used as an indication for drivers on traffic flow, weather, speed limits, individual speed, alternative-route guidance systems, and condition of the highway. DMSs could also be referred to as PCMSs if the DMSs are portable and are easily transferred from one location to another. Most research tests the effectiveness of DMSs under a simulated driving environment rather than in real-life situation (Miller et al. 2008). Few investigations focused on the effectiveness of DMSs based on the reduction of vehicle speed on a work zone environment. CMSs have become an integral part of work zone traffic control, advising motorists of unexpected traffic and routing situation. The following section briefly review the research reported in the literature using CMSs as a solution in reducing crash in a work zone or to improve the work zone safety.

Zech, Mohan, and Dmochowski measure the effectiveness of three commonly used CMS messages in reducing vehicular speeds and variance in highway work zone. The result of this research shows that, if properly selected, CMS messages can be significantly effective in reducing speeds of all classes of vehicles in highway work zone. A field study was conducted on Interstate 90 in western New York State, which included speed measurements of nearly 180,000 vehicles. The three types of CMS messages tested were the following: (1) RIGHT|LANE|CLOSED ~ KEEP|LEFT, (2) WORK ZONE|MAX SPEED|45 MPH ~ BE|PREPARED|TO STOP, and (3) LEFT|LANE|CLOSED ~ KEEP|RIGHT. Of the three CMS messages tested the second CMS message proved the most effective, significantly reducing vehicle speeds by 3.3 to 6.7 mph (5.3 to 10.8 km/h) (Zech et al. 2008).

Fontaine and Carlson evaluated the effectiveness of speed displays and portable rumble strips at reducing speeds in rural maintenance work zones. The field study was conducted in four test sites in the Childress District in Texas. All four sites were rural maintenance work zones on low-volume two-lane roads with 70 mph (112.7 km/h) speed limits and with work being completed within a single day. Fontaine and Carlson found that the speed display was effective. Car speeds were between 2 and 9 mph (3.2 and 14.5 km/h) lower in the advance warning area than with normal traffic control. Also, speed displays appeared to produce greater speed reduction in commercial trucks than in passenger cars. Speeds were 3 to 10 mph (4.8 to 16.1 km/h) lower with the speed display for trucks in the advance warning area (Fontaine and Carlson 2001).

Garber and Srinivasan conducted research using a CMS equipped with a radar unit on highways in Virginia. The CMS was placed within the work area at the beginning of the lane taper. Four different messages were evaluated during the course of the study, and the message “YOU ARE SPEEDING. SLOW DOWN” was the most effective. They concluded that the CMS with radar unit continued to be effective for long durations (Garber and Srinivasan 1998).

Benekohal and Shu observed the effectiveness of placing a single CMS in advance of work zones. Although the speed reductions were statistically significant, in general, they were not practically significant for trucks speed reduction. For some automobiles exceeding the speed limit, the CMS did reduce the speed by 20% (Benekohal and Shu 1992).

Ullman evaluated the effectiveness of using radar transmissions to reduce speeds without visible enforcement. Results showed that the radar signal, on average, reduced speeds by 3 mph (4.82 km/h) and had a greater effect on commercial trucks than on cars (Ullman 1991).

Jackels and Brannan conducted a similar study using a radar-controlled speed sign. The study revealed that the 85th percentile speeds were reduced from 68 to 58 mph (109.3 to 93.26 km/h) with the installation of just the static signs. The installation of the radar-controlled speed sign reduced the 85th percentile further to 53 mph (85.22 km/h) (Jackels and Brannan 1988).

OBJECTIVES AND SCOPE

The primary objective of this research was to conduct field experiments to determine the effectiveness of PCMS on reducing vehicle speed in rural highway work zones. Two different conditions were designed for the experiments: (1) PCMS was switched on and (2) PCMS was switched off.

DATA COLLECTION

The research team conducted the experiments in one rural highway work zone in Seneca, Kansas. These work zones are located on US 36; the traffic volume for US 36 is 3,630 vehicles per day (VPD). US 36 is a rural two-lane highway with a statutory speed limit of 65 mph and a posted work zone speed limit of 45 mph. The construction project was a paving operation in order to rehabilitate roadway surface. While construction operations were underway, the two-lane highways were reduced to one-lane two-way work zones. These operations required a temporary traffic control (TTC) to coordinate vehicles entering the work zones. When the normal function of the roadway is suspended, TTC provides for continuity of motor vehicle movement (FHWA 2003). Inside the TTC zone, temporary traffic signs (TTSs) guided the vehicles through and toward the flagger station, where the vehicle was halted to wait for the pilot car. The layout of work zone is shown in Figure 1. The experimental location was located 550 feet away from the first TTC either on the beginning or at the end in order not to disturb the TTC and to test the PCMS exclusively.

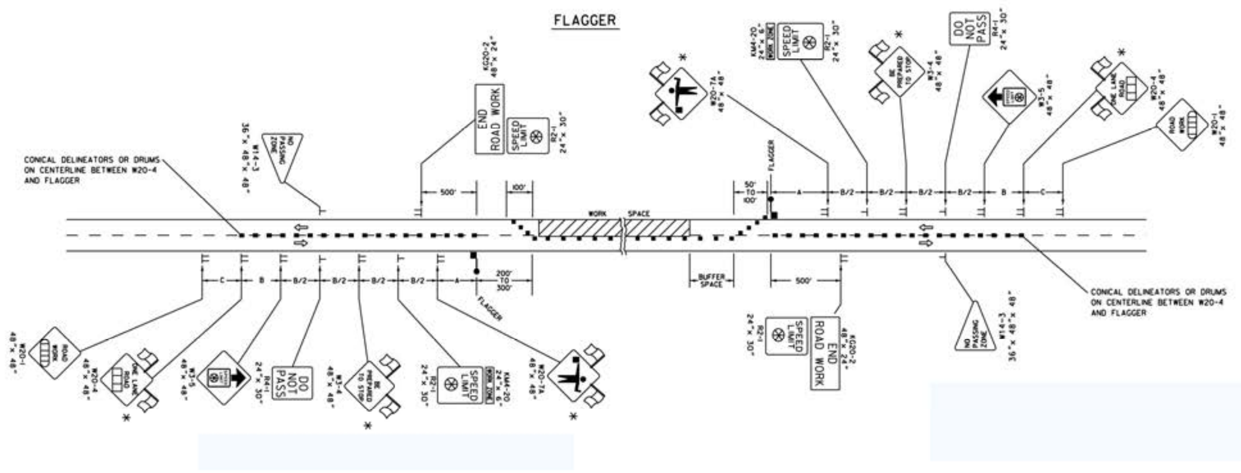

FLAGGER
CONICAL DELINEATORS OR DRUMS ON CENTERLINE BETWEEN W20-4 AND FLAGGER
W14-3
36"x 48"x 48"
NO PASSING ZONE
KG20-2
48"x 24"
END ROAD WORK
SPEED LIMIT
R2-1
24"x 30"
500'
100'
WORK
SPACE
BUFFER SPACE
50' TO 100'
200' TO 300'
FLAGGER
A
B/2
B
C
W20-1
48"x 48"
ROAD WORK
W20-4
48"x 48"
ONE LANE ROAD
W3-5
48"x 48"
R4-1
24"x 30"
DO NOT PASS
W3-4
48"x 48"
BE PREPARED TO STOP
KM4-20
24"x 6"
WORK ZONE
W20-7A
48"x 48"
*

Figure 1. Work zone layout on US 36

Vehicle speed was collected by two SmartSensor HD (Model 125) radar sensor systems. The SmartSensor HD is capable of collecting vehicle speeds up to 10 lanes and uses microwave radar technology to detect speeds with minimum influence from environmental condition (TxDOT 2007). Sensors were mounted on a 12-foot-tall tripod that was installed 8 to 12 feet away from the travel lane. This distance provided a relatively safe lateral clearance for passing traffic from the equipment and the researchers. In addition, this distance also complied with the manufacturer-recommended installation requirements. Field tests showed that this installation configuration enabled accurate speed collection especially when the speeds of the passing vehicles were greater than 20 mph. Figure 2 shows the setup of a SmartSensor HD system, and Figure 3 is the PCMS used for field experiments.

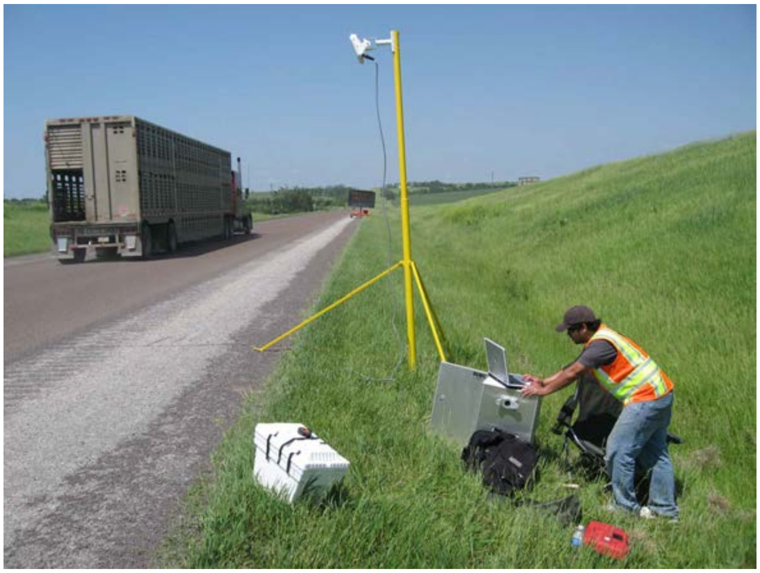


Figure 2. SmartSensor HD systems

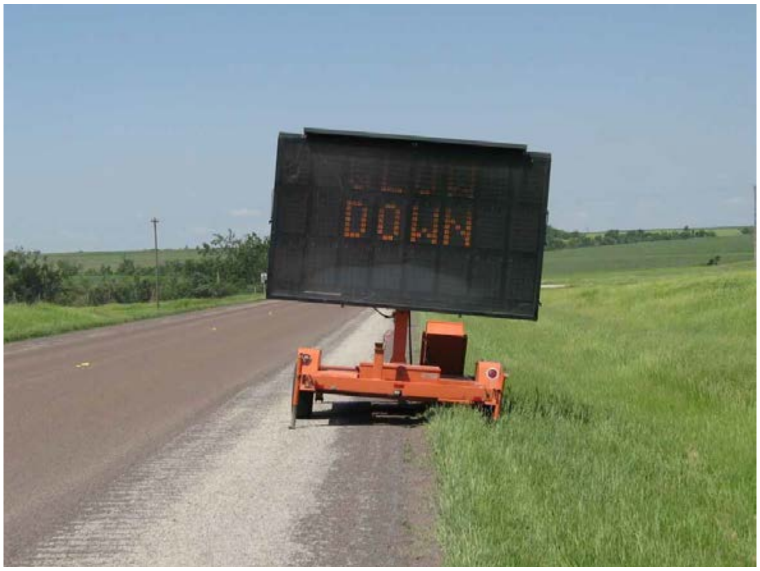


Figure 3. PCMS used for field experiments

In order to collect the speed data of the vehicles, two sensors were utilized. The first sensor (Sensor 1) was installed 1,050 feet from the first TTS (Road Work Ahead) and the second sensor (Sensor 2) was installed 550 feet from the first TTS. The PCMS was located within the two sensors and was 200 feet away from Sensor 2. The layout of sensors, PCMS, and the first TTS is shown in Figure 4. This layout was used for test condition 1 (PCMS ON) and 2 (PCMS OFF).

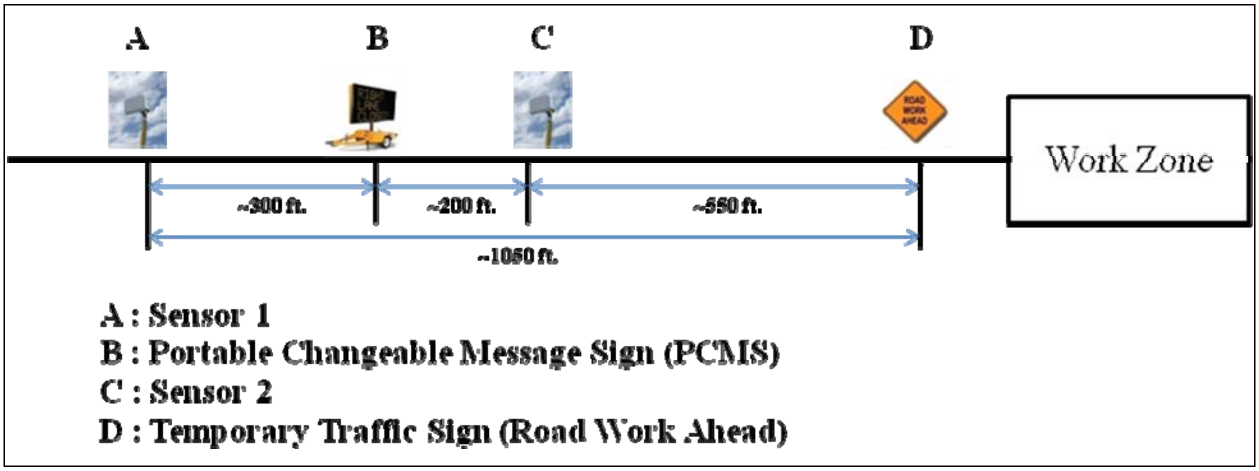

A
B
C
D
Work Zone
~300 ft.
~200 ft.
~550 ft.
~1050 ft.
A : Sensor 1
B : Portable Changeable Message Sign (PCMS)
C : Sensor 2
D : Temporary Traffic Sign (Road Work Ahead)

Figure 4. Experimental layout for test conditions 1 and 2

A successful experimental trial would depend on both sensors collecting the speed of vehicles during the experiment. Speed data of 793 vehicles were collected. Of these, 358 vehicle speed data were with the PCMS switched on and 435 were with PCMS switched off. Table 1 shows the list of data collected from US 36 on June 2,2008 through June 6, 2008.

Table 1. Speed data by different experimental conditions

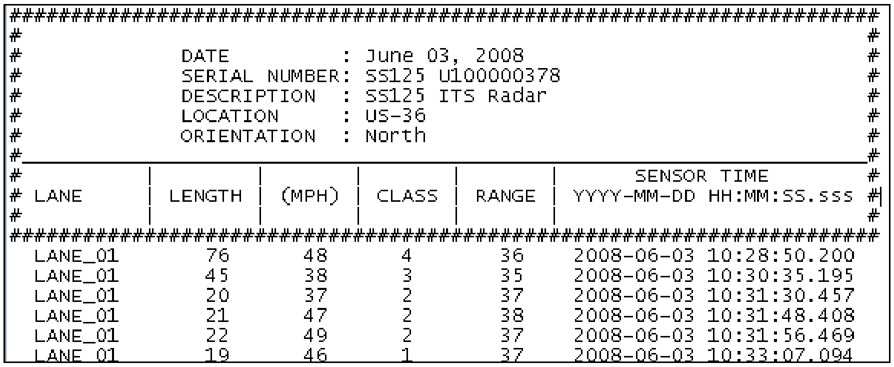
Work Zone	Speed Limit (mph)	PCMS ON	PCMS OFF
US 36	65	358	435

DATA ANALYSIS

The main objective of this research project was to evaluate the effectiveness of the PCMS on reducing vehicle speeds. The field experiments were conducted in one work zone that had speed limit of 65 mph. The effectiveness of the PCMS was measured based on the correlation of the vehicle speed change or the difference in speed from Sensor 1 and Sensor 2 under the two conditions. The important tasks that were accomplished in the analyses of speed data included analyses of the vehicle speed difference between Sensor 1 and Sensor 2 when the PCMS was turned on and off.

Although the SmartSensor has the capability of data storage and wireless data downloading, researchers had to be present in a real-time basis during the data collection to ensure accurate speed analysis. Two research assistants recorded the time when the PCMS was switched on and off and supervised the collected data by the sensors.

The sensors produced raw data files in a text file (.txt file) and classified the data by lanes, length of vehicle, speed, vehicle class, range, date and time, as shown in Figure 5. Researchers sorted the data that were collected from Sensor 1 and Sensor 2 based on individual vehicles. The data collected for each experimental condition followed the normal distribution from observation using statistical software called the Statistical Package for Social Science (SPSS). The normally distributed data for condition PCMS on and PCMS off can be seen in Figures 6 and 7.



```
########################################################################
#                                                                      #
#              DATE          : June 03, 2008                           #
#              SERIAL NUMBER: SS125 U100000378                         #
#              DESCRIPTION   : SS125 ITS Radar                         #
#              LOCATION      : US-36                                   #
#              ORIENTATION   : North                                   #
#______________________________________________________________________#
#           |          |         |        |        |     SENSOR TIME     #
# LANE      | LENGTH   | (MPH)   | CLASS  | RANGE  | YYYY-MM-DD HH:MM:SS.sss #
#           |          |         |        |        |                     #
########################################################################
  LANE_01        76        48       4        36     2008-06-03 10:28:50.200
  LANE_01        45        38       3        35     2008-06-03 10:30:35.195
  LANE_01        20        37       2        37     2008-06-03 10:31:30.457
  LANE_01        21        47       2        38     2008-06-03 10:31:48.408
  LANE_01        22        49       2        37     2008-06-03 10:31:56.469
  LANE_01        19        46       1        37     2008-06-03 10:33:07.094
```

Figure 5. Example of the text file

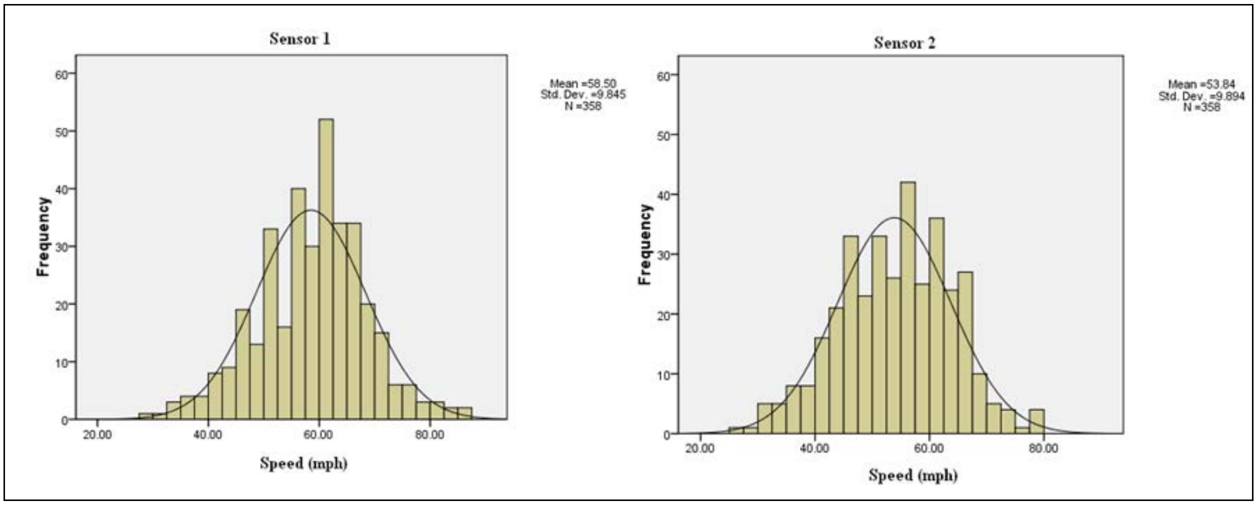


Figure 6. Data distributions of Sensors 1 and 2 when PCMS on

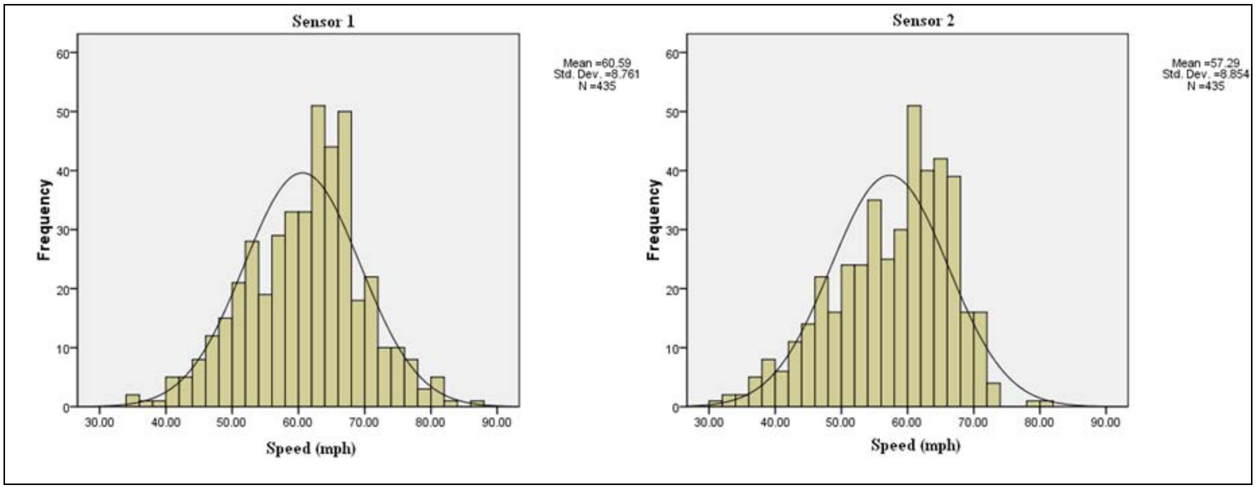


Figure 7. Data distribution of Sensors 1 and 2 when PCMS off

SPSS was used to calculate and analyze statistical values of the entire data sample, including the standard deviation, mean, and standard deviation error mean. Table 2 shows the computed values of the statistical analysis. The mean showed in Table 2 is the average vehicular speed reduction from the position of Sensor 1 to the position of Sensor 2.

Table 2. Statistical analysis values

Conditions	Population	Mean	Standard Deviation	Standard Error Mean
PCMS ON	358	4.7	6.44934	0.34086
PCMS OFF	435	3.3	5.69533	0.27307

Based on the comparison analysis that was mention above, one null hypothesis (H0) and alternative hypothesis (H1) were defined. The null hypothesis and the alternative hypothesis tested are as follows:

$$H_0 : (\mu_{O1} - \mu_{O2}) \leq (\mu_{F1} - \mu_{F2}), \quad \text{(Case 1)}$$

$$H_1 : (\mu_{O1} - \mu_{O2}) > (\mu_{F1} - \mu_{F2}),$$

where μ_{O1} or μ_{O2} = mean vehicle speed at Sensor 1 or Sensor 2, respectively, when the condition of the PCMS is on, and μ_{F1} or μ_{F2} = mean vehicle speed at Sensor 1 or Sensor 2, respectively, when the condition of the PCMS is off. If the probability of the conditions is below the threshold for a statistical significance of 5% level, then the null hypothesis (H_0) is rejected in favor of an alternative (H_1) hypothesis.

The normally distributed sample data and equality variances allowed researchers to test the significances using the t-test within each case. Using the SPSS software to calculate the significance by the independent two-sample t-test (unequal sample size and equal variance), the result was 0.002. Table 3 shows the computed values generated by SPSS. This value is significantly less than 0.05. As a result, researchers concluded that the null hypotheses are confidently rejected. Thus, the alternative hypotheses were statistically true.

Table 3. Independent sample t-test

Cases	Significant	Effectiveness? α = 0.05
1	0.002	YES

CONCLUSIONS

Highway statistics data indicated that 91% of the Kansas public roadway miles are rural, and approximately 97% of the major rural roadways (interstates, principal and minor arterials, and major collectors) are two-lane highways (Li et al. 2009). Preserving, rehabilitating, expanding, and enhancing these highways requires the setup of a large number of work zones. In Kansas, 63% of the fatal crashes and 33% of the injury crashes took place in two-lane highway work zones (Li and Bai 2008b). This research project evaluated the effectiveness of PCMS in rural highway work zones under the following situations: (1) PCMS was switched on and (2) PCMS was switched off. The results showed that the PCMS was effective on reducing vehicle speeds in two-lane work zones. The PCMS turned on was significantly effective comparing to the PCMS turned off. When the PCMS was turned on, it reduced vehicle speeds by 4.7 mph over 500 feet distance on average. When the PCMS was turned off, the vehicle speeds reduced 3.3 mph over 500 feet distance. Based on the results of data analyses, researchers concluded that a visible and active PCMS in a work zone significantly reduced the speed of vehicles approaching the work zones. A reduction in vehicular speed allows for greater reaction time to avoid crashes and potentially creates a safer environment for drivers and construction workers in the work zones.

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Update on Iowa's 2009 Roadway Departure Strategic Action Plan and Synthesis of Neighboring State Practices to Address Roadway Departure Crashes

Eric J. Fitzsimmons
Center for Transportation Research and Education
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010
efitz@iastate.edu

Tom Welch
Office of Traffic and Safety
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Tom.Welch@dot.iowa.gov

Shashi Nambisan
Institute for Transportation
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010
shashi@iastate.edu

Shauna Hallmark
Center for Transportation Research and Education
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010
shallmar@iastate.edu

ABSTRACT

Roadway departure crashes contribute to a significant percentage of the total number of motor vehicle crashes each year. Government agencies have reported that 61% of the 41,059 vehicle crashes in the United State in 2007 were due to a vehicle departing the roadway. It has also been reported that 25% of these crashes have occurred on horizontal curves. Roadway departure crashes can encompass a variety of types of crashes, including single-vehicle run-off-road crashes, multi-vehicle cross-centerline head-on crashes, and crashes involving fixed objects in the clear zone. Countermeasures deployed to reduce the number of roadway departure crashes include shoulder and centerline rumble strips, high-crash curve treatments, cable median barriers, and shoulder treatments. These countermeasures are typically located near the road and serve to alert the driver departing the roadway through visual, auditory, or vibratory warnings. Many of these countermeasures have been implemented by agencies in the midwestern region of the United States to address roadway departure crash problem areas. However, there is sometimes a lack of explicit guidance on where, how, and what designs should be used.

This paper provides a synthesis of countermeasures adopted to mitigate roadway departure crashes with a focus on the practices in the Midwest United States. It also includes guidance on implementation of countermeasures and discusses their effectiveness. These serve as a basis to develop a strategic action plan for the deployment of countermeasures to reduce roadway departure crashes in Iowa.

Key words: lane departure crashes—roadway departure crashes—safety countermeasures—strategic action plan

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Linking Highway Improvements to Patterns of Regional Growth and Land Use with Quasi-Experimental Research Design

Richard G. Funderburg
Graduate Program in Urban and Regional Planning
The University of Iowa
Iowa City, IA 52242-1316
richard-funderburg@uiowa.edu

Marlon G. Boarnet
Department of Planning, Policy, and Design
University of California, Irvine
Irvine, CA 92697-7075
mgboarne@uci.edu

Hilary Nixon
Department of Urban & Regional Planning
San Jose State University
San Jose, CA 95192-0185
Hilary.Nixon@sjsu.edu

EXTENDED ABSTRACT

Understanding linkages of new highway construction or capacity expansion to regional growth patterns is crucial for transportation planners and policymakers. Particularly important will be the ability of new projects to avoid or sustain challenges to Environmental Impact Statements (EIS) based upon forecasts of regional growth. A legal precedent for such challenges was established in 1997 when a U.S. district court judge ruled that the EIS for a proposed Illinois toll road was deficient because the growth projections were the same in the build and no-build scenarios (Sierra Club vs. United States DOT 1997). Despite considerable research on the topic, a fundamental debate in urban and regional planning remains as to whether new highway infrastructure induces growth or whether the new infrastructure merely follows the path of development to service regions that would have grown with or without the new investment. In this study, we incorporate popular regional growth forecasting models into a quasi-experimental research design that directly relates new highway investments in Merced, Orange, and Santa Clara Counties to changes in population and employment location, while controlling for no-build historical counterfactuals. We study this mix of urban, small-town, and ex-urban highway projects to examine the possibility of differential effects.

Recent empirical studies confirm effects from transportation infrastructure improvements on intrametropolitan location choices of people and employers to be both statistically and economically significant. Baum-Snow's (2007) examination of the suburbanization effects from the U.S. Interstate Highway System (IHS) estimates that building the first new highway through a central city reduces central city population growth by 17 percent, while increasing suburban population growth. Nationally, Baum-Snow (2007) estimates that building the IHS resulted in central city population growth that was 8 percent lower than what would have otherwise occurred, again shifting growth to the suburbs. In this study, we examine highway-related growth effects at a finer scale to address language and nuances posed by the National Environmental Protection Act regarding projections under "build" and "no build" scenarios and judicial decisions thereof. In short, given the recent evidence that the IHS contributed to the decentralization of U.S. metropolitan areas, what is the land use/growth impact of specific highway projects?

Our study cases (all from California) are in Orange County, where 51 new centerline miles of highway were added in a rapidly growing ex-urban area; Santa Clara County, where 19 new centerline miles of highway

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were added in an urban area; and Merced County, where a 1.5 mile highway bypass was built in a rural community. All of the new highway investments opened in the early 1990s. We use census data from 1980, 1990, and 2000 for our empirical test. The research design is a "before and after" comparison, looking at population and employment changes from 1980 to 1990 (the before period) compared to population and employment changes from 1990 to 2000 (the after period).

Our basic approach involves identifying the group of treatment census tracts or other spatial units that gain access to a new highway, identifying the superset of geographic units from which to select the control group, implementing alternative matching methods, analyzing changes in population and employment growth as difference in differences (treatment from matched control and over time), and incorporating the selection of matched controls into models of simultaneous population and employment growth to examine temporal changes in growth before and after the investments. The selection of controls is designed to incorporate the appropriate no-build counterfactuals into the forecast model.

Propensity score matching is the quasi-experimental technique used to select, as controls, regions similar in every respect to those receiving (or in proximity to) transportation improvements, except that the controls lacked any similar sort of intervention. Quasi-experimental techniques have been used in a variety of settings to find and match the cases among the set of potential controls that are most similar in every respect to the treatment group, except that the control group did not experience the intervention, thus preserving the intent behind random assignment in experimental design (Cook and Campbell 1979; Rosenbaum and Rubin 1983; Dehejia and Wahba 1999 and 2002; Holzer, Quigley, and Raphael 2003; O'Keefe 2004; Smith and Todd 2005).

With the matched control areas, we implement simple difference-in-differences tests for the impact of the highway infrastructure on population and employment change. In addition, we use a simultaneous model of population and employment change as a second test (e.g., Steinnes and Fisher 1974; Carlino and Mills 1987; Boarnet 1992 and 1994). Building on the recent specification of the endogenous growth model in Boarnet, Chalermpong, and Geho (2005), our contribution is to directly incorporate a selection of controls into the system of simultaneous equations so as to devise natural experiments for each of the three study counties.

We build on the considerable amount of research that has been conducted on regional growth forecasting models to explore how transportation infrastructure improvements have led to changes in population and employment location. We estimate that, on average, 338 to 11,103 new Orange County jobs shifted to a typical census tract that gained highway access when compared to a nearly identical census tract that did not gain highway access. This employment gain for tracts near the new highways is statistically and economically significant as it represents upwards of an 18 percent addition from 1990 levels to these tracts on average. We estimate that a new highway bypass built outside the small town of Livingston in Merced County had an opposite effect and instead resulted in a statistically significant 12 to 83 job losses per square kilometer as a result of the new bypass, a loss upwards of 11 percent of new jobs that the town otherwise might expect if the bypass had not been built. We find no significant effects on population or employment growth that can be attributed to the new highway investment near the urban center of Santa Clara County. The differential effects from highway investments in the three contexts illustrate the importance of choosing appropriate comparison groups in forecasts of population and employment growth for build and no-build scenarios. Thus, the key finding of the study is that while improvements in surface transportation infrastructure tend to have large impacts on growth patterns, the nature of the effects is materially dependent on the context of the highway investment.

Key words: highways—land use—quasi-experimental design

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Curling of New Concrete Pavement and Long-Term Performance

Daba S. Gedafa
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
daba@ksu.edu

Kirk Fredrichs
Kansas Division
Federal Highway Administration
6111 SW 29th St.
Topeka, KS 66614
kirk.fredrichs@fhwa.dot.gov

Mustaque Hossain
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
mustak@ksu.edu

Dave Meggers
Materials and Research Center
Kansas Department of Transportation
300 Van Buren
Topeka, KS 66611
dmeggers@ksdot.org

Zahidul Q. Siddique
Infrastructure Division
ARCADIS LNW
2081 Vista Parkway
West Palm Beach, FL 33411
zahidul.siddique@arcadis-us.com

ABSTRACT

Curling results from the temperature differential across the concrete slab thickness and may induce undue stresses in newly placed slab. This study deals with the finite element (FE) analysis of curling, curling stresses, field measurement of curling on a newly built jointed plain concrete pavement, and comparison of its long-term performance using both the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) and HIPERPAV II software. The FE analysis was performed with a software program ANSYS. The test section was modeled as a three-layer system with 12 in. (300 mm) concrete slab, 4 in. (100 mm) treated drainable base, and 6 in. (150 mm) lime-treated subgrade. All layers were assumed to be linear elastic. Temperature data were collected at five different depth locations across the concrete slab with digital data loggers. Curling was measured on five different days with a simple setup. The effect of temperature nonlinearities across the slab thickness was also examined. The results show that both upward and downward curling increase as the temperature differential increases. The maximum stress resulting from the combined effect of curling and traffic loading due to positive temperature differential is higher than that due to the negative temperature differential of the same magnitude. Since temperature differential has a significant influence on curling, both curling and curling stresses can be mitigated at an early age with temperature control, namely via enhanced curing. Both MEPDG and HIPERPAV II (HIgh PERformance concrete PAVing) showed approximately the same performance for the PCC thickness ranging from 12 in. (300 mm) to 8.5 in. (215 mm) for this project. Performance prediction from HIPERPAV II is very sensitive to the change in PCC thickness less than 9 in. (230 mm), whereas MEPDG prediction is not as sensitive to the thickness change as with HIPERPAV II.

Key words: curling—HIPERPAV II— long-term performance—MEPDG

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INTRODUCTION

The American Concrete Institute (ACI 1978) defined curling as “the distortion of any essentially linear or planar member into a curved shape such as the warping of a slab due to creep or to differences of temperature or moisture content in the zones adjacent to its opposite faces.” Curling is caused by the temperature gradient across the thickness of the concrete slab. It induces stresses in the pavement concrete slab since the slab is restrained by its weight and the support pressure from the foundation layer. The thermally induced stresses caused by such interaction can be a significant factor in contributing to early pavement cracking (Tang, Zollinger, and Senadheera 1993). This may be critical, particularly a few hours after placement of the slab, since concrete at an early stage of hydration may have insufficient strength to prevent cracking. Temperature rise caused by hydration does not immediately produce thermal stresses because of the process of stress relaxation or creep in the concrete (Emborg 1989). The temperature gradient in the newly placed slabs may cause the slabs to curl and even if the cracks do not develop, this gradient can cause curling, and permanent set of the concrete slabs in a non-planar fashion. This phenomenon is popularly known as “as-built” curling (Beckemeyer, Khazanovich, and Yu 2002). Recent studies have shown that this as-built curling adversely affects the future performance of jointed plain concrete pavement (JPCP) (Beckemeyer, Khazanovich, and Yu 2002; Huang 2004).

Traditionally, in the design and analysis of concrete pavements, the temperature distribution across the slab thickness is assumed to be linear. Unless actual field measurements are made, it is reasonable to assume a maximum temperature gradient of 2.5°F to 3.5°F/in (0.055°C to 0.077°C/mm) of slab during the day and about half of these values at night (Byrum 2000). Studies have shown that for the temperature range encountered in a temperate region, the assumption of linear temperature profile could lead to errors of 30% or more in the computed peak warping stresses (Choubane and Tia 1992). Zhang et al. (2003) reported that assumption of linear temperature distribution overestimated tensile stresses during certain periods of the day, while underestimated during other periods. Differences between the peak tensile stresses corresponding to the linear and nonlinear analyses reached as high as 75%. From field measurements, it is known that the temperature distribution is nonlinear. Typically, the nonlinear profile is featured with a relatively rapid change of temperature within the top quarter of the slab thickness, followed by a more gradual change towards the bottom face (Yoder and Witczak 1975). Researchers have represented non-linear temperature profile by a quadratic equation or by a third-order polynomial (Choubane and Tia 1992; Zhang et al. 2003).

OBJECTIVES

The main objective of this study was to evaluate curling and curling stresses of JPCP by the finite element (FE) method as well as to compare these results with the curling measured in the field. Comparison of the long-term performance of this JPCP based on the MEPDG software and HIPERPAV II (HIgh PERformance concrete PAVing) has also been done.

TEST SECTION AND DATA COLLECTION

The concrete pavement section in this study is located on Interstate route 70 (I-70). The section is a dowelled JPCP with 15 ft (5 m) joint spacing. The cross section consists of a 12 in. (300 mm) concrete slab, a 4 in. (100 mm) treated drainable base, also known as bound drainable base (BDB), and a 6 in. (150 mm) lime-treated subgrade (LTSG). The BDB layer has a minimum permeability of 1,000 ft/day (330 m/day). The concrete mixture was composed of 40% coarse and 60% fine aggregates with a water-cement ratio of 0.45. The entrained air was 5.8%. Average 28-day core compressive strength and 3-day modulus of rupture were 5,210 psi (35.9 MPa) and 580 psi (4 MPa), respectively.

Temperature data were collected by the digital data logger, iButton® (2003). iButton® is a computer chip enclosed in a 0.63 in. (16 mm) stainless steel can. The assembly was installed near the right wheel path, which was about 39 in. (1 m) away from the edge of the driving lane. Temperature data were collected at five different depths across the slab thickness: the top surface; 3 in. (75 mm); 6 in. (150 mm); and 9 in. (225 mm) below the top surface; and the bottom surface. Data was collected at 10 min intervals. The advantage of using five buttons is that they capture the actual temperature distribution across the slab thickness. Figure 1 presents a typical hourly pavement temperature distribution curve. It is apparent from the figure that the temperature distribution is nonlinear. The nonlinearity for the bottom half of the slab is not as pronounced as it is for the top half. For the bottom half, the distribution is almost linear. For the top half, the distribution for the hours of positive temperature differential (i.e., temperature of the top surface is higher than that of the bottom) is steeper than that for the hours of negative temperature differential (i.e., temperature of the bottom is higher than that of the top). The hourly variation of temperature at the bottom of the slab is not very prominent. The difference between the maximum and the minimum temperatures is about 10°F (4.5°C) for this particular case. However, this difference is more pronounced at the top surface, which is about 43°F (19°C).

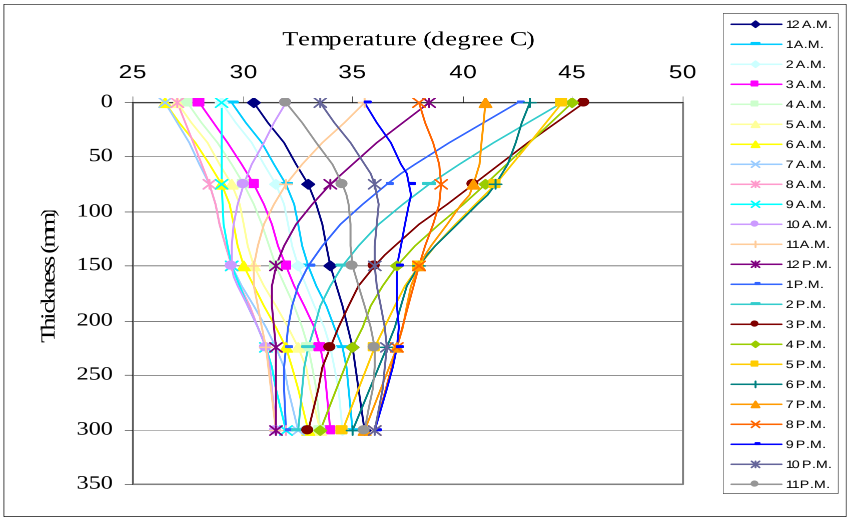


Figure 1. Typical temperature variation across the slab thickness

FINITE ELEMENT (FE) MODELING

In this study, ANSYS 7.0 (ANSYS 2003) was used to simulate curling. The model was built with actual geometric and material properties of the JPCP section described earlier. The FE model was built for a three-layer system. Each lane is 12 ft (3.7 m) wide, whereas the widths of the inside and outside shoulders are 6 ft (1.8 m) and 10 ft (3 m), respectively. All lanes and shoulders are separated by longitudinal joints with a width of 0.37 in. (9.5 mm) and a depth equal to the quarter of the slab thickness. Transverse joints in the model are located at 15 ft (5 m) intervals and the dimensions are the same as those of the longitudinal joints. Cracks developing along the slab edge under the transverse joints were also modeled. Dowel bars, located at the mid-depth of the slab with a bar diameter of 1.5 in. (37.5 mm) and length of 18 in. (450 mm), were placed at 6 in. (300 mm) intervals. Because of the symmetry in the longitudinal (driving) direction, one half of the slabs on both sides of a transverse joint were used as the model geometry.

Element selection is important to obtain reasonable results in the FE analysis. Based on the experiences of previous researchers (Channnakeshava, Barzegar, and Voyiadjis 1993; Davis, Turkiyyah, and Mahoney 1998; Kuo 1994), a 3-D 20 node brick element was selected as the candidate element for this study. In the ANSYS library, this element is known as SOLID186 (ANSYS 2003). The interaction between concrete and dowel bars is a complex one. This interaction was modeled as a contact problem. Rigid-to-flexible type of contact, available in the ANSYS library, was used in "surface-to-surface" contact mode. Target element TARGE170 and contact element CONTA174 were selected to model the target and contact surfaces, respectively.

Concrete layer material was modeled as linear elastic. The effect of concrete slab on curling is overwhelming compared to the effects due to the base and subgrade layers. Hence, base and LTSG layers were also modeled as linear elastic. Material properties needed for the FE model include the elastic properties such as modulus of elasticity and Poisson's ratio of different layers. Modulus of elasticity of concrete slab, drainable base, and subgrade layers used in this study were 4,206 ksi (29 GPa), 957 ksi (6.6 GPa), and 40 ksi (276 MPa), respectively. Poisson's ratios for the concrete, BDB, and LTSG layers were assumed to be 0.15, 0.15, and 0.20, respectively. The modulus of elasticity and the Poisson's ratio for the dowel bars were assumed as 29,000 ksi (200 GPa) and 0.25, respectively. Since this study deals with curling, which is caused by temperature, another important material property that was needed is the coefficient of thermal expansion. The typical values of 5 microstrains/°F to 6.67 microstrains/°F (9 microstrains/°C and 12 microstrains/°C) were used as the coefficients of thermal expansion for concrete and steel, respectively.

Generating an FE mesh is an important part of FE modeling. Finer meshes produce better results. However, several factors, such as size and complexity of the geometry, use of contact elements, and product limitation of the ANSYS version used in this study, restricted the creation of a very fine mesh. Figure 2 shows the meshed geometry of the study section. In general, the mesh is coarse. However, because of discontinuities created by the joints in the slab, areas near the joints were refined to obtain better results. The total number of elements generated for each model was approximately 30,000. Temperature was used as the main load. Temperature data were applied with both linear and non-linear temperature distribution across the slab thickness. The bottom of the subgrade layer was assumed to be fixed in all directions. The edges of the base and subgrade layers were fixed in the z-direction (direction of traffic). Pavement edge was allowed to move in all directions. Both translation and rotation of the dowel bar were restrained in all directions on one side of the joint.

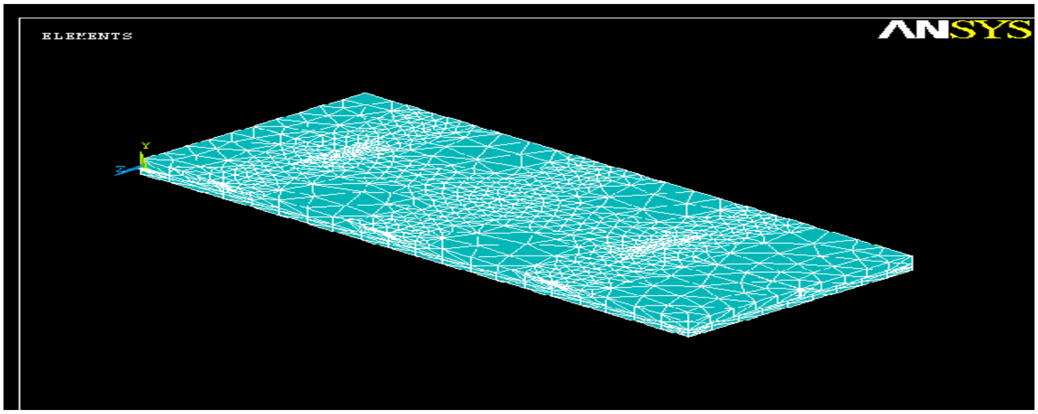


Figure 2. Meshed geometry

FE SIMULATION RESULTS

Six different positive and negative temperature gradients simulating daytime and nighttime temperature differentials were applied to the FE model. In this study, positive temperature differential refers to the daytime condition when temperature of the slab surface is higher than that of the bottom surface. The opposite is true for the negative temperature differential. The applied temperatures were: 22°F (-5.6°C), 27°F (-2.8°C), 37°F (+2.8°C), 42°F (+5.6°C), 47°F (+8.3°C), and 52°F (11.1°C). As expected, pavement slabs curled downward and upward for the positive and negative temperature differentials, respectively. Figure 3 shows the curled profiles of the section for these temperature gradients assuming linear temperature distribution across the slab thickness. Both upward and downward curling deflections increased with an increase in temperature differential. Maximum deflection of 0.012 in. (0.30 mm) was obtained when the temperature differential was the maximum, which is 52°F (11.1°C). However, curling deflection from a temperature differential of 52°F (11.1°C) was not twice that due to a temperature differential of 42°F (5.6°C), but was about 75% higher. The magnitude of the curling deflection for the same positive and negative temperature differential was not the same. A positive temperature differential resulted in higher magnitude of curling. The magnitudes of the upward curling were about 74% and 80% of downward curling for temperature differentials of 37°F (+2.8°C), and 42°F (+5.6°C), respectively.

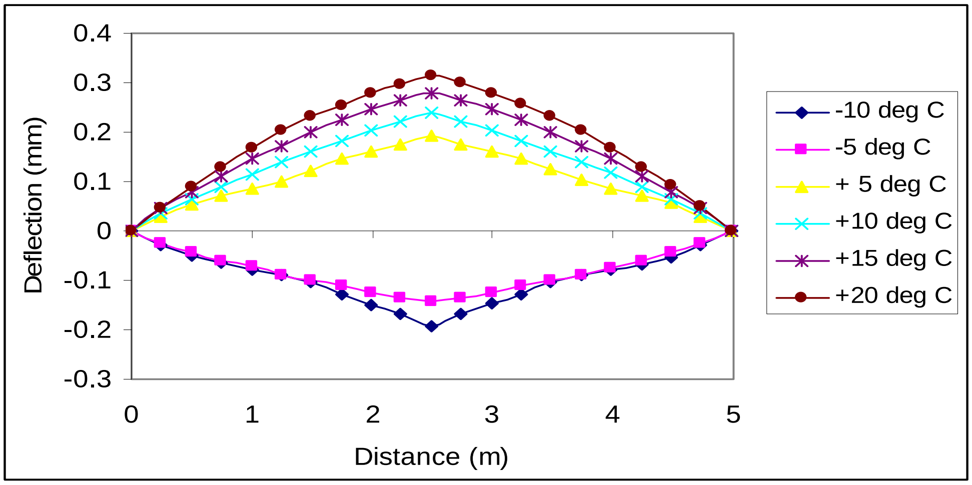


Figure 3. Deflection profiles of the study section

Curling Stress Analysis

It has long been recognized that critical stresses in concrete pavements result from the combined effects of curling and traffic loads. The location of traffic load also affects the critical stresses. In this study, the effect of truck loading in conjunction with the temperature loading was examined. An 18 kip (80 kN) static load (two 9 kip [40 kN] loads at 6 ft [1.8 m] apart) was used in the analysis. Three different load positions on the pavement slab were investigated: (1) center, (2) edge, and (3) corner. A preliminary analysis showed that for positive temperature differential, the critical load position is at the edge of slab. On the other hand, for negative temperature differential, corner loading is the critical load position. Table 1 tabulates the maximum stresses due to the combined effect of temperature and traffic loading for the critical load locations. The results show that for both positive and negative temperature differentials, maximum curling stresses increase with an increase in temperature differential. Maximum stresses resulted from the combined effect of temperature and traffic loading due to a positive temperature differential is higher than those due to the same negative temperature differential. The difference between the combined stress due to curling and traffic, and the curling stress increases with an increase in the temperature differential.

Table 1. Maximum stresses due to combined effect of temperature and traffic load

Temperature Differential (°C)	Maximum Stress (kPa)		
	Curling Only	Curling + Traffic Loading	Difference
-2.8	442	1090	648
-5.6	559	1366	807
2.8	462	1290	828
5.6	607	1628	1021
8.3	683	1911	1228

Linear Versus Non-Linear Temperature Distribution

In this study, the effect of linear and non-linear temperature distributions on curling was examined by the finite element method. Applied temperatures at the top and bottom for both analyses were the same. For nonlinear analysis, iButton temperatures were used to model nonlinear temperature distribution. Different temperature gradients ranging from 22°F (–5.6°C) to 57°F (+13.9°C) were used to compare the effects of linear and non-linear temperature distributions. Figure 4 shows the maximum curling deflections due to different temperature differentials for linear and non-linear temperature distributions. For both positive and negative temperature gradients, maximum deflections resulting from linear temperature distribution are lower than those obtained from non-linear distribution. The difference in curling for linear and non-linear temperature distribution is about of 3% to 5%.

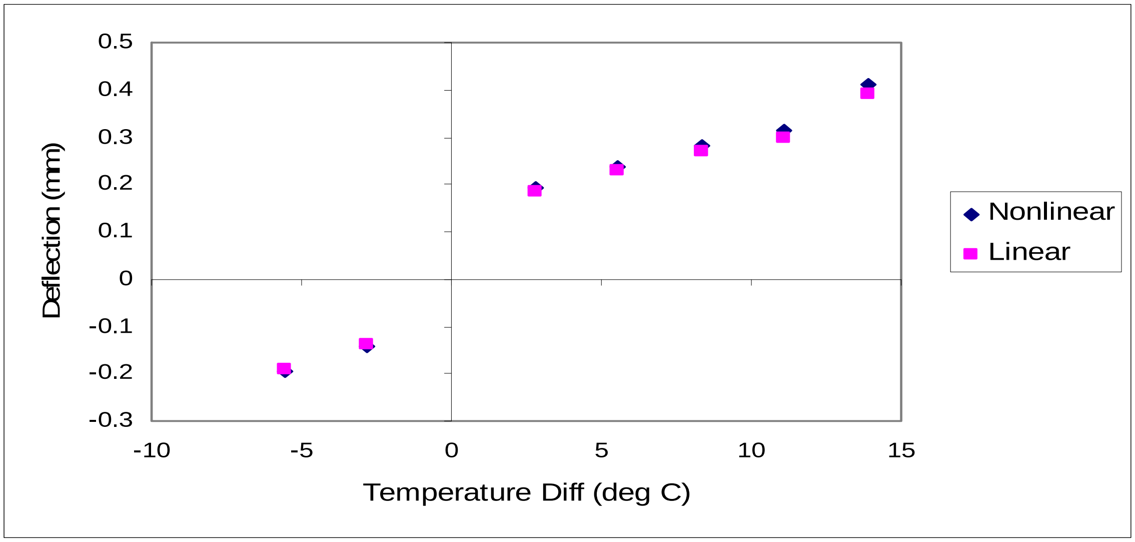


Figure 4. Variation of curling with temperature differential

Field Measurement of Curling

Curling deflection of the study section was also measured by a simple setup. The schematic of this setup is shown in Figure 5. It consists of an extensometer placed at the center of a lightweight aluminum frame. The length of the frame is approximately 15 ft (5 m), which represents the length of the concrete slab. The frame is positioned on steel pins attached to the bottom of the frame. These pins ensure correct and repeatable positioning, and serve to form a reference level. The pavement surface moves vertically with time because of the temperature differential between the pavement top and bottom surfaces. The extensometer, which is in contact with the top surface of the pavement, moves with the vertical movement of the concrete slab, thus measuring both upward and downward movements. This measurement represents the curling or mid-slab deflection of the pavement slab with respect to the reference plane established by the pins.

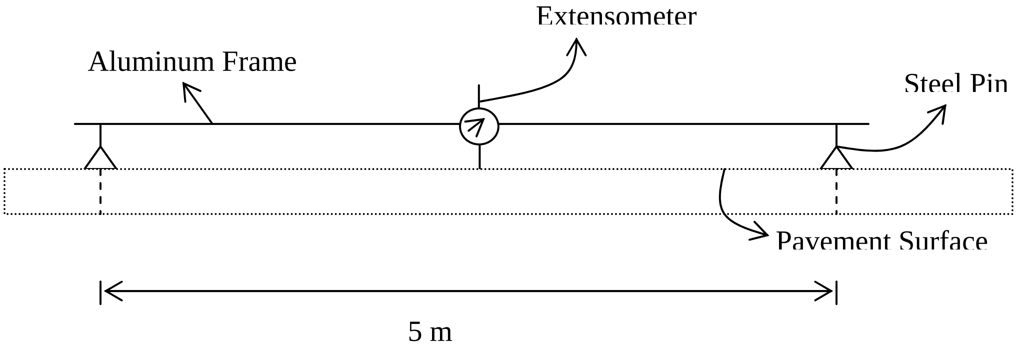


Figure 5. Curling measurement

Data were collected on five different days: August 7, September 2, September 25, October 19, and November 6 of 2003. For a particular day, hourly curling measurements were taken throughout the day. Table 2 shows measured curling deflections as well as corresponding temperature differential between the slab top and bottom.

Table 2. Curling deflection and temperature differential values

Day	**August 7**		**September 2**		**September 25**		**October 15**		**November 6**	
Time	**Curling (mm)**	**Temp. Diff. (^{0}C)**	**Curling (mm)**	**Temp. Diff. (^{0}C)**	**Curling (mm)**	**Temp Diff. (^{0}C)**	**Curling (mm)**	**Temp. Diff. (^{0}C)**	**Curling (mm)**	**Temp. Diff. (^{0}C)**
8 A.M	-	-5.0	-0.051	-5.0	-0.025	-6.5	-	-5.5	-	-4.5
9 A.M.	-0.025	-3.0	-0.013	-3.0	-0.025	-4.0	-0.076	-4.5	-0.051	-4.5
10 A.M.	-0.076	0.5	-0.010	-0.5	-0.051	-1.5	-0.051	-1.0	-0.076	-2.5
11 A.M.	-0.051	4.0	-0.051	1.5	-0.051	1.5	-0.051	2.5	-0.024	0.0
12 P.M.	0.025	7.0	0.051	5.5	0.051	4.5	0.076	5.5	-0.025	2.5
1 P.M.	0.279	10.5	0.203	9.0	0.229	6.5	0.229	7.0	0.076	5.0
2 P.M.	0.356	12.0	0.330	11.5	0.381	8.0	0.330	8.0	0.178	6.5
3 P.M.	0.635	12.5	0.584	13.0	0.457	8.5	0.432	8.5	0.356	7.0
4 P.M.	0.533	11.5	0.508	13.5	0.508	8.0	0.406	7.5	0.330	6.0
5 P.M.	0.457	10.0	10.406	12.0	0.381	6.5	0.406	6.0	0.279	4.0
6 P.M.	0.381	8.0	0.330	9.5	0.356	4.0	0.305	4.0	0.229	1.0
7 P.M.	0.330	5.5	0.330	6.5	0.279	1.5	-	2.0	-	-1.5

* Negative sign indicates upward curling

COMPARISON OF RESULTS

Comparison of FE and Field Measurement

Figure 6 shows the comparison of curling deflections obtained from the field measurements and the FE simulation. As mentioned earlier, during curling measurement on any given day, it was assumed that the slab was flat during the first measurement. Subsequent measurements were done hourly. Hence, these measurements showed the curling deflection of the section with respect to the first measurement. The finite element analysis was performed using the actual temperature condition during curling measurements.

Results from both methods show similar trend, although actual values are different. Deflections obtained by the FE simulation were lower than those measured in the field. Better agreements were observed for lower temperature differentials. However, the difference increased with an increase in temperature differential.

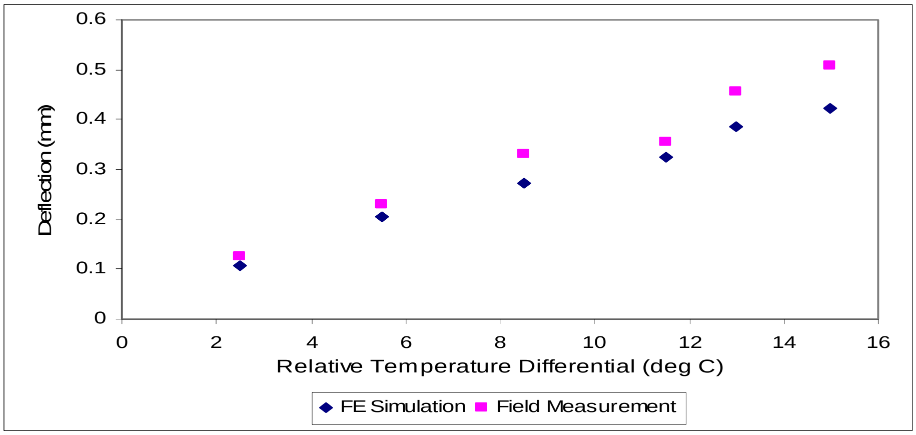


Figure 6. Comparison of FE simulation and field measurement

COMPARISON OF LONG-TERM PERFORMANCE USING MEPDG AND HIPERPAV II

MEPDG

The design methodologies in all versions of the AASHTO guide are based on the empirical performance equations developed using the AASHO Road Test data from the late 1950s. Due to the limitations of earlier guides, a design guide, based as fully as possible on mechanistic principles, was developed under the National Cooperative Highway Research Program (NCHRP) (NCHRP 2004). The guide is popularly known as *Mechanistic Empirical Pavement Design Guide* (MEPDG). The procedure is capable of developing mechanistic-empirical design while accounting for local environmental conditions, local materials, and actual highway traffic distribution by means of axle load spectra. The designer first considers site conditions (traffic, climate, and material properties) in proposing a trial design for a new pavement. The trial design is then evaluated for adequacy against some predetermined failure criteria. Key distresses are predicted from the computed structural responses of stress, strain, and deflection due to given traffic and environmental loads. If the design does not meet desired performance criteria, it is revised, and the evaluation process is repeated as necessary (NCHRP 2004).

The hierarchical approach is used primarily for traffic, materials, and environmental inputs in MEPDG. This approach provides the designer with several levels of "design efficacy" that can be related to the class of highway under consideration or to the level of reliability of design desired. In general, three levels of inputs are provided. Input data used for the MEPDG analysis of concrete pavements are categorized as the following: (a) General information, (b) Traffic, (c) Climate, (d) Pavement structures, and (e) Miscellaneous. The key outputs are the individual distress quantities. The outputs for JPCP are roughness in terms of international roughness index (IRI), percent slabs cracked, and joint faulting at the required level of reliability for a given design period

HIPERPAV II

HIPERPAV II is a Windows-based concrete paving software. It was originally developed by the Transtec Group, Inc. for the Federal Highway Administration (FHWA) to serve as a tool in the proper selection and control of the factors affecting concrete pavement behavior at early age. By controlling these factors properly, one can ensure good performance of concrete pavement throughout its design life. It is the first software of its kind to provide control over the concrete pavement design and construction. It deals with the analysis of behavior of concrete pavement in the first 72 hour period after placement of both JPCP and continuously reinforced concrete pavement (CRCP). In addition, the software determines the effect of early-age behavior factors on long-term performance of JPCP (McCullough and Rasmussen 1999).

Some of the main inputs in HIPERPAV II for analysis of JPCP are design (geometry, dowels, and slab support), materials and mix design (type of cement, portland cement concrete [PCC] mix, PCC properties, and maturity data), construction, environment, and traffic loading. Traffic loading is used only for long-term performance analysis. Each of these inputs has its own detailed components. The outputs for early-age JPCP are critical stress, evaporation rate, and dowel analysis results. The outputs for the long-term performance are joint faulting, transverse cracking, longitudinal cracking, ride (in terms of the International Roughness Index [IRI]), and serviceability. Longitudinal cracking and serviceability results have not been used in this study since there are no corresponding MEPDG outputs.

Comparison of Long-Term Performance

The thickness of PCC has been varied from 6 in. (150 mm), which is the minimum allowable thickness using MEPDG, to original PCC thickness of 12 in. (300 mm) to compare the long-term performance at the end of 20 years using both MEPDG and HIPERPAV II. Long-term performance has been compared in terms of IRI (with minimum and maximum values of 63 in/mi (0.99 m/km) and 164 in. (2.59 m/km), respectively), percent slabs cracked (maximum 15%), and mean joint faulting with terminal value of 0.12 in. (3 mm). The maximum limits have been indicated on the figures. Comparison based on these performance criteria has been done separately.

IRI

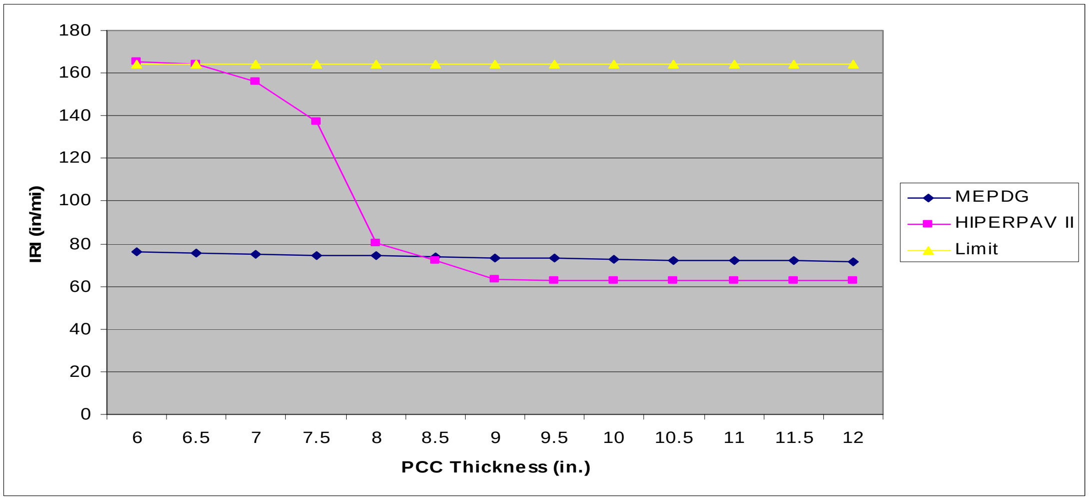
Figure 7 (a) illustrates the roughness development using both MEPDG and HIPERPAV II. The change in roughness due to change in thickness from 12 in. (300) to 6 in. (150 mm) is 4.6 in/mi (73 mm/km) and 103 in/mi (1619 mm/km) for MEPDG and HIPERPAV II, respectively. The increase in roughness due to decease in thickness is minimal for MEPDG. PCC thickness less than 7 in. (177.8 mm) is not recommended since roughness exceeds the limit while using HIPERPAV II. MEPDG and HIPERPAV II give approximately the same roughness at PCC thickness of 8.5 in. (216 mm). In general, roughness prediction from MEPDG is not sensitive to the change in PCC thickness.

Percent Slabs Cracked

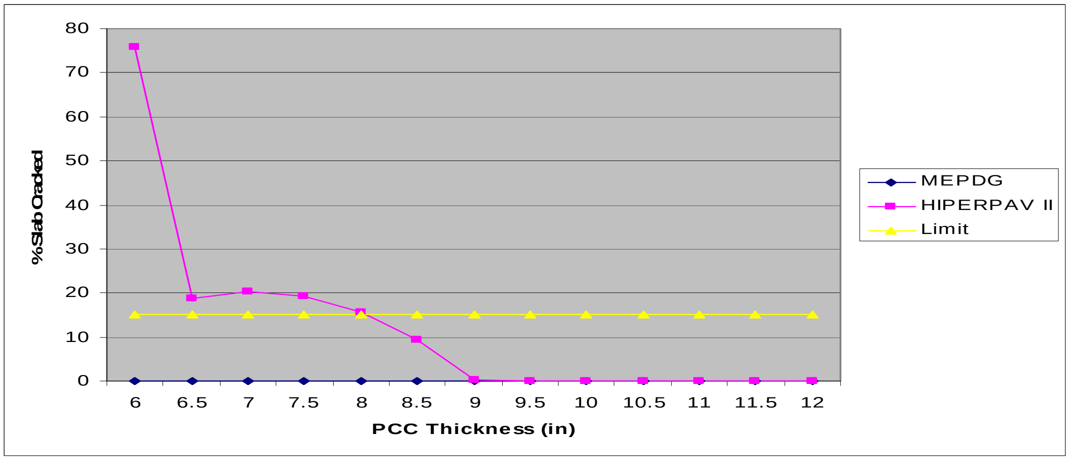
Comparison of long-term performance in terms of percent slabs cracked is shown in Figure 7 (b). No crack has been observed in prediction by MEPDG, but a significant increase in cracking was observed in HIPERPAV II. Both procedures have shown no cracking up to PCC thickness of 9 in. (230 mm). The increase in percent slabs cracked due to decrease in thickness from 9 in. (230 mm) to 6 in. (150 mm) is 76%. A significant increase in cracking was observed at a thickness lower than 6.5 in. (165 mm) for HIPERPAV II. PCC thickness less than 8.5 in (216 mm) is not recommended as per the results by HIPERPAV II.

Mean Joint Faulting

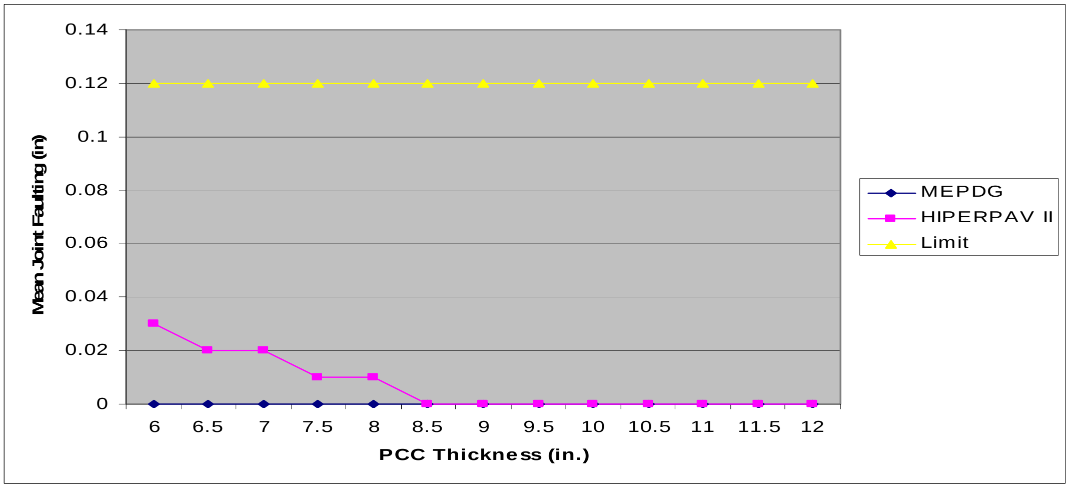
As shown in Figure 7 (c), no joint faulting was observed for both procedures up to PCC thickness of 8.5 in. (216 mm). There is no faulting for the range of thickness for MEPDG, but there is an increase of faulting from 0 to 0.03 in. (0.76 mm) when the thickness was decreased from 8.5 in. (216 mm) to 6 in. (150 mm) using HIPERPAV II.



(a) IRI



(b) Percent slabs cracked



(c) Mean joint faulting

Figure 7. Comparison of long-term JPCP performance

CONCLUSIONS

Based on this study, the following conclusions can be made:

- Both upward and downward curling increase as the temperature gradient increases. Based on the results, a maximum temperature gradient of 54°F (12°C) appears to be reasonable for typical PCC pavements in Kansas.
- Curling resulting from a particular positive temperature differential is slightly higher than that resulting from the negative temperature differential of the same magnitude.
- The maximum stresses resulting from the combined effect of curling and traffic loading due to a positive temperature differential are higher than those due to the negative temperature differential of same magnitude. This should be considered in the pavement design as is done in the new NCHRP MEPDG.
- Assumption of linear temperature distribution across the slab thickness results in lower curling deflection than that due to nonlinear distribution. The difference between the deflection values is less than 10%.
- Curling deflections measured in the field are in close agreement with the deflections obtained from the FE simulation for lower temperature differentials.
- MEPDG and HIPERPAV II analysis showed about similar performance for the PCC thickness ranging from 12 in. (300 mm) to 8.5 in. (215 mm).
- The optimum PCC thickness for this project has been found to be 8.5 in. (215 mm) from both procedures.

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Simulation of Flexible Pavement Design in Kansas

Daba S. Gedafa
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
daba@ksu.edu

Stefan A. Romanoschi
Department of Civil Engineering
University of Texas at Arlington
UTA Box 19308
Arlington, TX 76019-0308
romanoschi@uta.edu

Mustaque Hossain
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
mustak@ksu.edu

Andrew J. Gisi
Kansas Department of Transportation
Materials & Research Center
2300 Van Buren
Topeka, KS 66611
agisi@ksdot.org

ABSTRACT

The Kansas Department of Transportation (KDOT) is currently adopting the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) to replace the 1993 American Association of State Highway and Transportation Officials (AASHTO) design method. It would be valuable for the designer to know whether the MEPDG design simulation analysis would predict the actual distresses observed in Kansas. Thus, five newly built Superpave pavements, designed using the 1993 AASHTO design guide, were selected as test sections for the design-simulation study. Deflection data were collected approximately 8 to 10 weeks after construction using a falling-weight deflectometer (FWD). The FWD deflection data were used to backcalculate the pavement-layer moduli using three different backcalculation programs. All mixture data were obtained from the KDOT mix-design database. Required parameters for use in asphalt concrete (AC) mixture modulus prediction equation were then calculated. The existing pavement structures were analyzed for a 10-year analysis period. The maximum numbers of years the existing pavement structures will be in a serviceable condition as well as the minimum thicknesses of different layers to serve for ten years were determined. Effects of changing subgrade modulus, target distress, and reliability were also investigated.

The MEPDG design analysis shows that the 1993 AASHTO guide-designed flexible pavements do not show the distresses currently observed in Kansas for the 10-year design period. Thus, local calibration of the design models appears to be essential. The MEPDG design simulation shows that the thinner the pavement sections, the higher the AC layer and total permanent deformation. The existing pavement structures can serve for more than 20 years as per the MEPDG design analysis if the default failure criteria and nationally calibrated models are used. The total AC thickness varies from 3 to 6 inches for a 10-year design period if the effect of AC surface-down cracking (longitudinal cracking) is ignored. The lowest thickness is observed on a pavement that has 11 inches of aggregate base. The minimum total AC thickness to serve for a 10-year period, considering the longitudinal cracking, varies from 6 to 9 inches. The lowest International Roughness Index (IRI) is observed on a pavement that has the highest total AC thickness and vice versa. Longitudinal cracking does not depend on the thickness of AC layers. Backcalculated subgrade moduli obtained from various backcalculation programs result in variable predicted distresses for different projects. Distress target values and reliability do not show significant effects on the maximum service life of the existing pavements. Verification of these results by the in-service pavement performance is recommended.

Key words: backcalculation—MEPDG—simulation

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INTRODUCTION

The most widely used procedure for the design of flexible pavements is specified in the *Guide for Design of Pavement Structures* published in 1986 and 1993 by the American Association of State Highway and Transportation Officials (AASHTO 1986, AASHTO 1993). A few states use the 1972 AASHTO interim guide procedure, their own empirical or mechanistic-empirical procedures, or a design catalog (Hall 2000). The design methodologies in all those versions of the AASHTO guide are based on the empirical performance equations developed using the AASHO Road Test data from the late 1950s.

Due to the limitations of earlier guides, a design guide, based as fully as possible on mechanistic principles, was developed under the National Cooperative Highway Research Program (NCHRP) (NCHRP 2004). The procedure is capable of developing mechanistic-empirical design, while accounting for local environmental conditions, local materials, and actual highway traffic distribution by means of axle-load spectra. Since the resulting procedure is very sound and flexible and it considerably surpasses the capabilities of any currently available pavement design and analysis tools, it has been adopted by AASHTO as the new AASHTO design method for pavements structures.

The design method adopted in the *Guide for Mechanistic–Empirical Design of New and Rehabilitated Pavement Structures* (NCHRP) is popularly known as the *Mechanistic-Empirical Pavement Design Guide* (MEPDG). In MEPDG, prediction of pavement response and performance must take into account fundamental properties of layer materials. Among these, the most important property of hot mix asphalt (HMA) is the dynamic modulus of asphalt concrete. This property represents the temperature- and time-dependent stiffness characteristics of the HMA material. Significant amount of effort has been devoted to developing a test protocol to determine the dynamic modulus of HMA (Witczak et al. 2002). This effort has resulted in a standard test protocol that can be used for the "Simple Performance Test for Superpave® Mix Design" (NCHRP 2002). This test protocol calls for the use of axial-compression testing for measuring the dynamic modulus. One of the issues related to the dynamic modulus is its use in forensic studies and pavement rehabilitation design.

In the hierarchical design approach proposed in MEPDG for new HMA pavements, direct measurements of dynamic modulus are required for the highest design reliability (Level 1), which is intended for pavements with very high traffic volumes. However, dynamic modulus is used as the primary stiffness property for HMA at all three levels of hierarchical inputs in MEPDG.

The dynamic modulus test is relatively difficult and expensive to perform. Therefore, numerous attempts have been made to develop regression equations to estimate the dynamic modulus from mixture volumetric properties. The predictive equation developed by Witczak et al. (2002) is one of the most comprehensive mixture dynamic modulus models available today that can predict the dynamic modulus of dense-graded HMA mixtures over a range of temperatures, rates of loading, and aging conditions. These inputs are available from conventional binder tests and the volumetric properties of the HMA mixture. A revised version of this model has been recommended in the design of intermediate- and low-volume roadways (design Levels 2 and 3) in MEPDG (NCHRP 2007).

PROBLEM STATEMENT

The Kansas Department of Transportation (KDOT) is currently considering adopting MEPDG to replace the 1993 AASHTO design method that is in use now. However, work is needed to determine whether MEPDG gives results similar to that of the AASHTO design method and/or predict the distresses that match the measured/observed distresses.

OBJECTIVES OF THE STUDY

The main objectives of this study are to:

- Investigate the AASHTO flexible design method in Kansas using M-EPDG
- Investigate the effect of subgrade modulus on the predicted distresses using M-EPDG
- Investigate the effect of failure criteria and reliability while using M-EPDG

TEST SECTIONS AND DATA COLLECTION

Test Sections

Five newly built Superpave pavements, designed using the 1993 AASHTO Design Guide, were selected as test sections in this study. Each test section was 1,000 ft. long. Table 1 tabulates the layer types and thicknesses of these sections.

All pavement sections have Superpave 9.5 mm nominal maximum aggregate-size mixture (known as SM-9.5A and SM-9.5T in Kansas) with PG 64-28 binder in the surface course of 1.5 in. thick. Layers 2 and 3 consist of fine-graded Superpave 19 mm nominal maximum aggregate-size mixture, SM-19A, with PG 64-28 and PG 64-22 binders, respectively. The base layer thickness varies from 5 to 8.5 in. The K-7 project in Doniphan County has the thinnest asphalt base (5 in.), since it also has 11 in. crushed-stone base, designated as AB-3 in Kansas.

All projects have lime-treated subgrade except K-7 in Doniphan County, where subgrade was modified with a Class C fly ash.

Table 1. Layer type and thickness

Layer No.	Layer Type	Material Type	Thickness (in.)				
			US-54	US-77	US-283	K-7	K-99
			Butler	Butler	Grah.	Don.	Elk
1	Surface	SM-9.5A (PG 64-28)	1.5[a]	1.5	1.5	1.5	1.5
2	Binder	SM-19A (PG 64-28)	2.5	2.5	2.5	2.5	2.5
3	Base	SM-19A (PG 64-22)	8.5	8	7	5	7
4	Aggregate Base	AB-3	N.A.	N.A.	N.A.	11	N.A.
5	Subgrade	Modified Subgrade	6[b]	6[b]	6[b]	6[c]	6[b]

a: SM-9.5T PG 64-28; b: Lime-treated subgrade (LTSG); c: fly ash modified subgrade (FASG).

Data Collection

Deflection Data

Deflection data were collected approximately 8 to 10 weeks after construction. Multiple-target loads were used on most test sections. The target loads used in the falling-weight deflectometer (FWD) testing were 9, 12, and/or 15 kips for all sections. Deflection measurements were made in the outside wheel path of the travel lane at 11 stations at 100 ft. intervals. The geophone spacing was 0, 8, 12, 18, 24, 36, and 48 inches for US-54, US-77, and US-283. The last sensor was located at 60 in. for the K-7 and K-99 projects. FWD test locations, test dates, and target loads for the sections are shown in Table 2.

Table 2. FWD test locations, dates and target loads

Routes	County	District	Test Date	Load	
				Target Load (kips)	Repetitions
US-54	Butler	5	10/31/05	9, 12, 15	1
US-77	Butler	5	07/13/05	9, 12, 15	1
US-283	Graham	3	07/11/05	9, 12, 15	1
K-7	Doniphan	1	08/07/06	9, 12	3
K-99	Elk	4	07/18/07	9, 12, 15	3

Volumetric Properties

Most of the mixture data required have been obtained from the mix-design database of KDOT. Information includes gradation of aggregates (cumulative percent retained on ¾ in., 3/8 in., No. 4 sieves, and percent-passing No. 200 sieve), physical properties of the aggregates (bulk and effective specific gravities), asphalt content and asphalt specific gravity, and theoretical maximum specific gravity of the mixture.

Bulk specific gravities of the compacted samples and the cores were determined in the laboratory following the Kansas Standard Test Method KT-15, Procedure III. KT-15 closely follows AASHTO T 166. From these pieces of information, the air void (%), effective binder content (% by volume), voids in the mineral aggregates (VMA%), and percent of VMA filled with binder (%) were calculated. The original, mix/lay-down, surface aging, and aging at different viscosities have been determined at different temperatures and frequencies. Temperature data for different locations have been obtained from Kansas State University (KSU) weather data library.

ANALYSIS METHODOLOGY

Backcalculation of Modulus

FWD deflection data have been normalized to 9 kip load. These normalized deflection data were used to backcalculate the pavement-layer moduli based on the multilayered elastic theory. The moduli of thin surface or sandwiched layers are usually difficult to obtain because surface deflections are often insensitive to changes in the moduli of these layers. Changes in the moduli of subgrade or other thick layers may mask changes in thin layers (Chou and Lytton 1991). Flexible pavements are usually analyzed as three-layered systems, having an asphalt-concrete surface layer, a mechanically or chemically stabilized base layer, and a subgrade (Meier, Alexander, and Freeman 1991). In this study, all pavement sections were modeled as three-layer systems by combining all asphalt-concrete layers into one layer. Comparison of solutions from different programs gives an idea of the range of solutions that can be expected (Chou and Lytton 1997). Thus, three backcalculation computer programs—EVERCALC, MODCOMP 5, and MODULUS—were used in this study. It is to be noted that only backcalculated, subgrade moduli have been used in this study.

In the backcalculation of pavement-layer moduli, the objective is to identify a set of pavement-layer moduli that would produce a deflection basin matching the measured deflection basin. Since only a finite number of sensor data points are available from the deflection measurements, the objective function in the backcalculation analysis typically involves the minimization of the root-mean-square difference (D_{rms}) of the measured and computed deflections. A solution that has a smaller D_{rms} value is considered to be a better fit, and thus, a better solution (Fwa, Tan, and Chan 1997).

$$Minimize\, D_{rms} = \sqrt{\frac{1}{m}\sum_{i=1}^{m}\left(\frac{d_i - D_i}{D_i}\right)}, \tag{1}$$

where m = number of deflection-measurement points, d_i = backcalculated deflection at point *i*, and D_i = measured deflection at point *i*.

Analysis Using MEPDG

MEPDG software version 1.0 was used to do the design analysis. Four cases have been considered at Level 3 using default distress targets. The existing pavement structure was analyzed for a 10-year analysis period as Case 1. Case 2 considered the maximum number of years the existing pavement structure will be in a serviceable condition. Minimum thickness of different layers to serve for 10 years has been found by ignoring and considering longitudinal cracking as Cases 3 and 4, respectively.

MEPDG Design Inputs

Hierarchical approach is used for the design inputs in MEPDG. This approach provides the designer with several levels of "design efficacy" that can be related to the class of highway under consideration or to the level of reliability of design desired. The hierarchical approach is primarily employed for traffic, materials, and environmental inputs (NCHRP 2004). In general, three levels of inputs are provided.

Level 1 is an advanced design procedure and provides the highest practically achievable level of reliability and is recommended for design in the heaviest traffic corridors or wherever there are dire safety or economic consequences of early failure. The design inputs are also of the highest practically achievable level and generally require site-specific data collection and/or testing.

Level 2 is the input level expected to be used in routine design. Level 2 inputs are typically user selected, possibly from an agency database. The data can be derived from a less-than-optimum testing program or can be estimated empirically.

Level 3 is typically the lowest class of design and should be used where there are minimal consequences of early failure. Inputs typically are user-selected default values or typical averages for the region.

Input data used for the MEPDG analysis of flexible pavements are categorized as General Information, Site/Projection Identification, Analysis Parameters, Traffic, Climate, Pavement Structures, and Miscellaneous. Each of them is discussed below.

General Information

The general information inputs include design life, construction month, traffic-opening month, and pavement type. All pavement sections in this study were flexible pavements and analyzed for different design periods.

Site/Project Identification

Project location, project identification, and functional class of the pavements are included under this input category. Project location defines the climatic conditions for the pavement design. The functional class influences the default design criteria, helps determine the default vehicle classification, and aids in the selection of the vehicle-operating speed input.

Analysis Parameters

Flexible pavement design is based on the surface-down and bottom-up fatigue cracking of the asphalt surface, HMA thermal cracking, fatigue cracking in chemically stabilized layers, permanent deformation for both asphalt layers and the whole pavement, and smoothness. Since there are no stabilized layers in this study, fatigue cracking in chemically stabilized layers is not applicable. Default and modified criteria have been used in this study. Distress targets have been changed for some of the distresses as tabulated in Table 3. Reliability was also changed to 50% keeping the default distress targets (not included in the table).

Table 3. Performance criteria for the study

Distress Type	**Criteria 1/Default**		**Criteria 2**	
	Distress Target	**Reliability Level (%)**	**Distress Target**	**Reliability Level (%)**
Terminal IRI (in/mi)	164	90	164	90
AC Surface Down Cracking (Long. cracking) (ft/mi)	1,000	90	500	90
AC Bottom Up Cracking (Alligator cracking) (%)	25	90	10	90
AC Thermal Fracture (Transverse cracking) (ft/mi)	1,000	90	500	90
Chemically Stabilized Layer (Fatigue Fracture)	25	90	10	90
Permanent Deformation (AC only)(in.)	0.25	90	0.25	90
Permanent Deformation (Total pavement) (in.)	0.75	90	0.75	90

Traffic

Traffic data are one of the key elements required for the design and analysis of pavement structures. The basic required information are annual average daily truck traffic (AADTT) for the base year, percent trucks in the design direction, percent trucks in the design lane, and operational speed of the vehicles. Three functions are available to estimate future truck traffic volumes: no growth, linear growth, and compound growth. Linear growth rate was used in this study.

Project-specific linear traffic growth rates varied from 0.9% to 1.7%. Directional- and lane-distribution factors for trucks were taken as 60% and 100%, respectively. Percent of trucks varied from 13 to 26% as indicated in Table 4. For this study, some other required traffic inputs were derived from the MEPDG Level 3 or default values.

Table 4. Summary of traffic data

	US-54	**US-77**	**US-283**	**K-7**	**K-99**
Initial two-way AADT	3,959	1,217	1,046	1,251	1,862
Percent of Trucks	13	26	20	14	16
Linear growth rate (%)	1.5	1.4	0.9	1.4	1.7
Operational speed (mph)	70	60	65	55	55
No. of lanes in each direction	1	1	1	1	1
Directional distribution (%)	60	60	60	60	60
Lane distribution (%)	100	100	100	100	100

Climate

Environmental conditions have significant effects on the performance of flexible pavements. The seasonal damage and distress accumulation algorithms in the MEPDG design methodology require hourly data for six weather parameters, such as air temperature, precipitation, wind speed, percentage sunshine, relative humidity, and seasonal or constant water table depth at the project site (NCHRP 2004). The design guide recommends that the weather inputs be obtained from weather stations located near the project site. At least 24 months of actual weather station data are required for the computations.

The design guide software includes a database of appropriate weather histories from 851 weather stations throughout the United States. This database is accessed by specifying the latitude, longitude, and elevation of the project site. The design guide software locates the six closest weather stations to the site. Specification of the weather inputs is identical at all three hierarchical input levels in MEPDG. In this study, project specific virtual weather stations were created by interpolation of climatic data from the selected physical weather stations.

Pavement Structures

Input values for pavement-structure properties are organized into drainage and surface characteristics, layer properties, and distress potential. Flexible pavement-design procedure allows a wide variety of asphalt, base, and layer thicknesses. The original pavement structure defined by the user usually has 4 to 6 layers. However, MEPDG may subdivide the pavement structure into 12 to 15 sublayers for modeling of temperature and moisture variations. Sublayering depends on material type, layer thickness, and the location of the layer within the pavement structure. A maximum of 19 layers can be analyzed.

The inputs required for the AC layer were thickness, PG binder grade, gradation, Superpave mixture volumetric properties, Poisson’s ratio, reference temperature, etc. The software-computed dynamic modulus (E*) uses the default Witczak’s predictive equation that takes into account gradation, volumetric properties, asphalt-binder grade, and reference temperature (NCHRP 2004).

Miscellaneous

The thermo-hydraulic properties required as inputs into MEPDG are groundwater depth, infiltration and drainage properties, physical/index properties, hydraulic conductivity, thermal conductivity, heat capacity, etc. (Barry and Schwartz 2005). The recommended calibrated values of 1.25 BTU/hr-ft-°F and 0.28 BTU/lb-°F were used for thermal conductivity and heat capacity, respectively. Physical and index properties were derived based on the gradation of the unbound materials. Surface-shortwave absorptivity and drainage-path length were chosen based on the default inputs and were 0.85 and 12 ft., respectively.

Performance Models

All performance models used in this study are the nationally calibrated default ones. Only the inputs represent local conditions or projects in Kansas.

RESULTS AND DISCUSSIONS

The existing pavement structures were analyzed for a 10-year analysis period as Case 1. Case 2 considered the maximum number of years the existing pavement structures will be in a serviceable condition. Minimum thickness of different layers to serve for 10 years has been found by ignoring and considering longitudinal cracking as Cases 3 and 4, respectively. Default criteria have been used for these cases. Effects of changing subgrade modulus, target distress, and reliability have also been presented.

Case 1: 10-Year Analysis Period

The predicted distresses for the existing pavement structures are tabulated in Table 5. These predicted distresses are far less than the target distress limits. The initial International Roughness Index (IRI) is 64 in/mi for all projects. The lowest and highest IRI are 86.1 and 89.3 in/mi, respectively. Longitudinal and transverse crack are zero and one, respectively, for all projects. The thinner the pavement sections, the higher the AC and total permanent deformation.

Table 5. Distress predicted using 10-year analysis period

Distresses	**Predicted Distress**					**Distress Target**
	US-54	**US-77**	**US-283**	**K-7**	**K-99**	
IRI (in/mi)	86.7	86.1	87.5	89.3	89	164
Long. Cracking (ft/mi)	0	0	0	0	0	1,000
Alligator Cracking (%)	0	0	0	0.1	0.1	25
Transverse Cracking (ft/mi)	1	1	1	1	1	1,000
AC permanent deformation (in.)	0.08	0.05	0.07	0.09	0.09	0.25
Total permanent deformation (in.)	0.29	0.28	0.31	0.33	0.34	0.75

Case 2: Maximum Design Life

Table 6 tabulates the maximum number of years the existing pavement structures will be in a serviceable condition. K-7 has the thinnest total AC thickness, and as a result, it has the least service period. US-77 has the highest service period. The lowest and highest IRI is observed on US-283 and US-77, respectively. Insignificant amount of longitudinal cracking is observed on only K-99. The lowest and highest AC and total permanent deformation are observed on US-77 and K-99, respectively.

Table 6. Maximum design life for existing pavement structure

Distresses	Predicted Distress					Distress Target
	US-54	US-77	US-283	K-7	K-99	
Maximum Design Life (years)	24	25	23	22	23	
IRI (in/mi)	120.1	121.7	118.3	119.9	120.3	164
Long. Cracking (ft/mi)	0	0	0	0	0.1	1,000
Alligator Cracking (%)	0.1	0	0.1	0.1	0.4	25
Transverse Cracking (ft/mi)	1	1	1	0	1	1,000
AC permanent deformation (in)	0.12	0.08	0.1	0.13	0.14	0.25
Total permanent deformation (in)	0.37	0.35	0.38	0.41	0.44	0.75

Case 3: Minimum Thickness Ignoring Longitudinal Cracking

The causes and effects of AC surface-down cracking (longitudinal cracking) are not well understood yet, and it was ignored (there has been failure in longitudinal cracking) in this case to determine the minimum thicknesses of different layers to serve for 10 years, as shown in Table 7. The lowest to highest total AC thickness to serve for 10 years are 3, 4.5, 4.5, 5.5, and 6 inches for K-7, US-283, US-77, US-54, and K-99, respectively. K-7 has the lowest total AC thickness, since it has 11 in. of AB-3. K-7 and US-54 have the lowest and highest alligator cracking, respectively. US-283 and US-54 have the lowest and highest AC permanent deformation, whereas K-99 and US-54 have the lowest and the highest total permanent deformation, respectively.

Table 7. Minimum layer thicknesses ignoring longitudinal cracking

		Predicted Distress				
		US-54	US-77	US-283	K-7	K-99
Layer Thickness	Surface (in.)	1.5	1.5	1.5	1	1.5
	Binder (in.)	2	2	2	1	2.5
	Base (in.)	2	1	1	1	2
	AB3 (in.)	-	-	-	11	-
Distresses	IRI (in/mi)	100.4	102.3	100.9	101.7	99.1
	Long. Cracking (ft/mi)	839	1,010	959	63.1	4260
	Alligator Cracking (%)	6.1	4.7	3.8	1.4	5.7
	Transverse Cracking (ft/mi)	1	1	1	1	1
	AC permanent deformation (in.)	0.15	0.14	0.11	0.13	0.13
	Total permanent deformation (in.)	0.54	0.6	0.58	0.59	0.52

Case 4: Minimum Thickness Considering Longitudinal Cracking

Minimum layer thicknesses of different layers to serve for a 10-year period considering the longitudinal cracking are tabulated in Table 8. The lowest to highest total AC thickness are 6, 6.5, 7.5, 7.5, and 9 inches for K-7, US-77, US-54, US-283, and K-99, respectively. The lowest total AC thickness is observed for K-7, which has 11 inches of AB-3. The lowest IRI is observed on a pavement that has the highest total AC thickness and vice versa. The highest longitudinal cracking is observed on US-77 and K-99. This shows that longitudinal cracking does not depend on the thickness of AC layers. Transverse cracking amount is constant for all projects. US-283 has the lowest AC and total permanent deformation.

Table 8. Minimum layer thicknesses considering longitudinal cracking

		Predicted Distress				
		US-54	**US-77**	**US-283**	**K-7**	**K-99**
Layer Thickness	Surface (in.)	1.5	1.5	1.5	1.5	1.5
	Binder (in.)	2	2	2.5	2.5	2.5
	Base (in.)	4	3	3.5	2	5
	AB3 (in.)	-	-	-	11	-
Distresses	IRI (in/mi)	92.3	92	91.4	93.1	91.4
	Long. Cracking (ft/mi)	2.9	8.8	2.6	4.1	8.8
	Alligator Cracking (%)	0.7	0.4	0.2	0.4	0.5
	Transverse Cracking (ft/mi)	1	1	1	1	1
	AC permanent deformation (in.)	0.12	0.08	0.08	0.1	0.11
	Total permanent deformation (in.)	0.42	0.42	0.4	0.42	0.4

Effect of Subgrade Modulus on Predicted Distress

Backcalculation programs(EVERCALC, MODCOMP 5 and MODULUS) have been used to backcalculate the subgrade modulus. One-third of the backcalculated subgrade modulus was used as the subgrade design modulus to run the MEPDG software for a 10-year design-analysis period. Default failure criteria have been used for this analysis. The results for different projects have been presented and discussed below.

US-54 Project

The design subgrade modulus varies from 2,121 to 27,345 psi for US-54, as indicated in Table 9. Transverse and longitudinal cracking remain the same at all subgrade moduli. IRI, alligator cracking, and total permanent deformation decrease with an increase in subgrade modulus, whereas the AC permanent deformation increases/remains constant with an increase in subgrade modulus. No failure has been observed at any subgrade modulus level.

Table 9. Effect of subgrade modulus on predicted distresses for US-54

	Design Subgrade Modulus (psi)				
	2,121	**3,704**	**4,107**	**5,158**	**27,345**
Distresses	**Predicted Distress**				
IRI (in/mi)	98.7	92.3	91.4	89.5	82.3
Long. Cracking (ft/mi)	0	0	0	0	0
Alligator Cracking (%)	0.2	0.1	0.1	0.1	0
Transverse Cracking (ft/mi)	1	1	1	1	1
AC permanent deformation (in.)	0.07	0.08	0.08	0.08	0.08
Total permanent deformation (in.)	0.59	0.43	0.41	0.36	0.18

US-77 Project

Table 10 shows that the design subgrade modulus for US-77 varies from 1,145 to 9,694 psi. Transverse and longitudinal cracking remain the same at all subgrade moduli. IRI, alligator cracking, and total permanent deformation decrease with an increase in subgrade modulus, whereas AC permanent deformation increases/remains constant with an increase in subgrade modulus. There is a failure in total pavement deformation when subgrade modulus is 1,145 psi. The result shows that the stronger the subgrade, the higher the AC permanent deformation.

Table 10. Effect of subgrade modulus on predicted distresses for US-77

	Design Subgrade Modulus (psi)			
	1,145	**2,056**	**3,070**	**9,694**
Distresses	**Predicted Distress**			
IRI (in/mi)	108.90	98.60	93.5	84.5
Long. Cracking (ft/mi)	0	0	0	0
Alligator Cracking (%)	0.1	0.1	0	0
Transverse Cracking (ft/mi)	1	1	1	1
AC permanent deformation (in.)	0.04	0.05	0.05	0.05
Total permanent deformation (in.)	0.84*	0.59	0.46	0.24

* indicates failure

US-283 Project

The design subgrade modulus varies from 1,124 to 9,694 psi for US-283, as indicated in Table 11. Transverse and longitudinal cracking remain the same at all subgrade modulus levels. IRI, alligator cracking, and total permanent deformation decrease with an increase in subgrade modulus, whereas the AC permanent deformation increases/remains constant with an increase in subgrade modulus. There is failure due to total pavement deformation when the subgrade modulus is 1,124 psi.

Table 11. Effect of subgrade modulus on predicted distress for US-283

	Subgrade Modulus (psi)				
	1,124	**2,714**	**6,572**	**7,788**	**8,667**
Distresses	**Predicted Distress**				
IRI (in/mi)	113.3	97.3	88.3	87.3	88.9
Long. Cracking (ft/mi)	0	0	0	0	0
Alligator Cracking (%)	0.1	0.1	0	0	0
Transverse Cracking (ft/mi)	1	1	1	1	1
AC permanent deformation (in.)	0.06	0.06	0.07	0.07	0.09
Total permanent deformation (in.)	0.95*	0.55	0.33	0.3	0.32

* indicates failure

K-7 Project

The design subgrade modulus varies from 6,340 to 10,053 psi for K-7, as indicated in Table 12. Transverse cracking, longitudinal cracking, and AC permanent deformation remain the same at all subgrade moduli. IRI, alligator cracking, and total permanent deformation decrease with an increase in subgrade modulus. There is no failure at any subgrade modulus level.

Table 12. Effect of subgrade modulus on predicted distress for K-7

	Subgrade Modulus (psi)			
	6,340	**7,583**	**9,348**	**10,053**
Distresses	**Predicted Distress**			
IRI (in/mi)	90.8	89.7	88.5	88.1
Long. Cracking (ft/mi)	0	0	0	0
Alligator Cracking (%)	0.1	0.1	0	0
Transverse Cracking (ft/mi)	1	1	1	1
AC permanent deformation (in.)	0.09	0.09	0.09	0.09
Total permanent deformation (in.)	0.37	0.34	0.31	0.3

K-99 Project

For K-99, the design subgrade modulus varies from 6,564 to 10,133 psi, as indicated in Table 13. Transverse cracking remains constant at all subgrade modulus. IRI, alligator cracking, and AC permanent deformation decrease with an increase in subgrade modulus, whereas longitudinal cracking and total permanent deformation decrease with an increase in the subgrade modulus. There is no failure at any subgrade modulus level.

Table 13. Effect of subgrade modulus on predicted modulus for K-99

	Subgrade Modulus (psi)		
	6,564	**6,709**	**10,133**
Distresses	**Predicted Distress**		
IRI (in/mi)	90	89.8	87.3
Long. Cracking (ft/mi)	0	0	0.1
Alligator Cracking (%)	0.2	0.2	0.1
Transverse Cracking (ft/mi)	1	1	1
AC permanent deformation (in)	0.09	0.09	0.1
Total permanent deformation (in)	0.37	0.37	0.3

Effect of Reliability

The effect of reliability has been investigated by keeping the distress target at default values and changing the reliability to 50%. All projects failed in total permanent deformation, and there has not been any increase in maximum service life due to the change in reliability under existing traffic condition. Higher traffic values have been used to investigate the effect of reliability, and maximum service life is higher using 50% reliability than that of 90%. The results have not been included since the results are the same under the existing condition.

Effect of Distress Target

The effect of distress targets on the maximum service of projects in this study has been investigated using Criteria 2 in Table 3. There has not been a change in the maximum design life due to the changes in distress target under the existing conditions.

CONCLUSIONS

Based on this study, the following conclusions have been made:

- MEPDG design analysis shows that the 1993 AASHTO design for flexible pavements does not show the default distresses currently observed in Kansas in the 10-year design period. Currently, by the end of 10-year design period, over 50% of pavements have had some form of structural rehabilitation. The MEPDG analysis results also show that the thinner the pavement sections, the higher the asphalt-concrete layer moduli and total permanent deformation.
- The existing pavement structures can serve for more than 20 years as per the MEPDG design analysis if the nationally calibrated models and default failure criteria are used. This is contrary to the current Kansas experience that shows that by year 18, nearly all pavements have had some kind of structural rehabilitation.
- Total AC thickness varies from 3 to 6 inches for a 10-year design period if the effect of AC surface-down cracking (longitudinal cracking) is ignored. The lowest thickness is observed on K-7, which has 11 inches of AB-3.
- The minimum total AC thickness to serve for 10-year period, considering the longitudinal cracking, varies from 6 to 9 inches. The lowest IRI is observed on a pavement that has the highest total AC thickness and vice versa. Longitudinal cracking does not depend on the thickness of AC layers.
- Backcalculated subgrade moduli obtained from various backcalculation programs result in variable predicted distresses for different projects.
- Distress target and reliability have not shown significant effects on the maximum service life of the existing pavements. This need to be verified by performance observation of in-service pavements.

ACKNOWLEDGMENTS

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Safety Analysis for Older Drivers at Signalized Intersections in Kansas

Daba S. Gedafa
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
daba@ksu.edu

ABSTRACT

The proportion of older drivers continues to increase since baby boom generation is becoming old. The level of mobility of older drivers is also increasing. Older drivers begin to noticeably be overinvolved in fatal crashes. Intersections appear to be hazardous to older drivers, particularly left turns due to one or more of sensory, perceptual, cognitive, physical, and general driving-knowledge deficiencies. The safety concern of older drivers is also becoming significant. Ten years of crash data have been extracted from the Kansas Accident Report System. It has been analyzed using statistical analysis software (SAS). Five different age groups have been considered. Comparisons have been made between different age groups of the same gender and the same age group for different gender. It has been found that proportion of older drivers in "through movement" involved in accidents decreases as age increases, whereas the proportion of left-turn accidents increases with age; left turns are harder for females in all age groups; proportion of right-turn accidents has no specific trend. Effects of light condition, weather condition, surface type, surface condition, road character, and construction/maintenance zone on older drivers' safety have also been considered in this study. Most of the accidents have taken place during daylight when there are no adverse weather conditions on dry surface, straight and level road, and blacktop surface type.

Key words: older drivers—safety—signalized intersections

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INTRODUCTION

The proportion of older drivers continues to increase since baby boom generation is becoming old. The level of mobility of older drivers is also increasing. Older drivers are the fastest growing segment of the driving population. In 2000, the elderly accounted for 12.4 percent of the U.S. population. It is estimated that persons 65 and over will make up more than 15 percent of the total drivers (Carr 2000); the number will approximately be 50 million in 2020 and 20 percent of the total drivers in 2030 (Burkhardt and McGavock 1999). According to the Federal Highway Administration (FHWA), if design is controlled by 85th percentile performance requirements, individuals over 65 years will be the design driver in the early 21st century (Staplin et al. 2001).

Older drivers begin to noticeably be overinvolved in fatal crashes. In the next 30 years, due to the increased number of older drivers, the number of fatalities involving older drivers is expected to increase three to four times if there is no change in the current crash rates (Burkhardt and McGavock 1999; Yaw et al. 2003). Some of the factors for higher fatality rates are related to body frailty and environmental factors (Braver and Trempel 2004; Li, Braver, and Chen 2001). The higher crash propensity of older drivers is often attributed to typical aging-related deterioration (Staplin et al. 1999; Deason 1998; Bailey and Sheedy 1988). A recent written report identified five main deficiencies present in older drivers. These include sensory, perceptual, cognitive, physical, and general driving knowledge deficiencies (Ballard et al. 1993).

Intersections appear to be hazardous to elderly drivers, particularly left turns. Studies have indicated that a large majority of accidents involving elderly drivers take place at intersections. The main difficulties elderly drivers face at intersections are estimation of a safe gap and speed of other vehicles, fast execution of driving maneuvers, failure to sense and comprehend traffic signals and signs, inability to perceive and process information about high-traffic volumes, failure to signal their turns, poor positioning when turning left along with a general lack of caution, frequent failure to stop, and jerky or abrupt stops (Yaw et al. 2003).

The safety concern of older drivers is becoming significant. In this study, 10 years of crash data at signalized intersections in Kansas has been analyzed, and comparison have been made for drivers in five different age groups for men and women. Different severity levels have been considered separately, and finally, overall crash data has been analyzed for through, left-turn, and right-turn movements.

RESEARCH OBJECTIVES

The main objectives of this study are

- Analyze crash data at signalized intersections in Kansas for old drivers.
- Compare the result within the same age group and different gender and different age group of the same gender.
- Determine effect of light condition, weather condition, surface type, surface condition, road character, and construction/maintenance zone on the safety of older drivers.

DATA AND RESEARCH METHODOLOGY

Ten years of crash data, 1993–2002, has been extracted from the Kansas Accident Report System database for drivers older than 29 years and has been used for this study. Through (TH), right-turn (RT), and left-turn (LT) crashes have been considered separately. The various injury severities considered are disabled-incapacitating (D), fatal injury (F), injury-not incapacitating (I), not injured (N), and possible injury (P).

Many researchers commonly classify the elderly population into three subgroups as “younger old,” ages 65–74; “older old,” ages 75–84; and “oldest old,” ages 85+. Another study classified drivers from ages 30 to 50 as middle-aged, ages 65 to 74 as young elderly, and 75+ as old elderly. Five age groups have been considered in this study as shown in Table 1.

Table 1. Five age groups for this study

Sr. No.	Age Group	Remark
1	30-49	Comparison group
2	50-64	
3	65-74	
4	75-84	
5	85+	

Statistical analysis software (SAS) has been used to analyze the data in this study. Then, comparison has been made for different age groups of the same gender and the same age group for different gender.

RESULTS AND DISCUSSIONS

Linear Regression

Intercept, slope, and coefficient of determination (R^2) have been determined by using SAS based on age group as independent variable and severity level as dependent variable. The results are shown in Table 2. The higher the coefficient of determination, the better the linear model compared to intercept only model. T-test has been done to determine the significance of the intercepts and slopes at 5% significance level. The slopes and intercepts that are not significant are indicated by (*) in the table. If slope is not significant, there is no linear relationship between severity level and age group. The intercepts for TH are higher for males than females at all age groups, as shown in Figure 1. The slope shows a decrease in proportion of TH accidents for a unit increase in age group. The magnitude of slopes for females is higher than that of males in “disabled” and “possible injury” cases, whereas it is higher for males when “fatal,” “injury (not incapacitated),” and “not injured.” As a result, there is no specific trend in slope unlike the intercept.

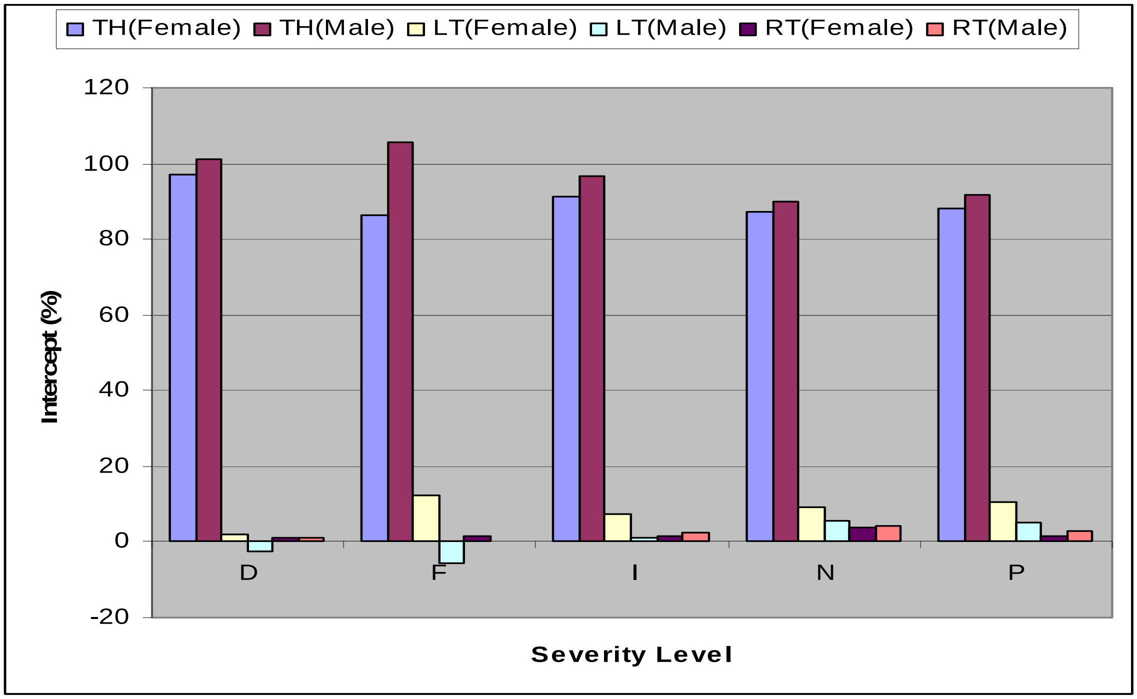
Table 2 also shows the linear model coefficients for LT accidents. Unlike that of TH accidents, the intercepts for females are higher than males. The slopes for males are higher than females except in “disabled” case. Both the slope and intercept are not significant for “fatal injury” for females.

Statistical estimates for linear model of RT accidents are also indicated in Table 2. There is no defined trend for RT accident cases unlike LT and TH. The slopes and intercepts change in magnitude and sign at different severity levels. The intercepts for males are higher at all severity levels except for “fatal injury,” as shown in Figure 1. Intercepts are not significant at severity levels of “disabled” and “fatal injury” for both females and males and “possible injury” for females. Slopes are not significant for both females and males at severity levels of “disabled,” “fatal injury,” and “possible injury” and “injury” for males. Slopes for females have higher magnitude than males except for “fatal injury.”

Table 2. Summary statistics of linear models for various severity levels

Severity	Statistics	Gender					
		Female			Male		
		TH	LT	RT	TH	LT	RT
D	Intercept	97.20	1.80	1.00*	101.36	-2.64*	1.28*
	Slope	-6.96	6.55	0.41*	-5.27	5.46	-0.19*
	R^2	0.93	0.94	0.62*	0.94	0.96	0.24*
F	Intercept	86.24	12.38*	1.38*	105.59	-5.68*	0.07*
	Slope	-0.78*	1.04*	-0.26*	-6.23	6.01	0.23*
	R^2	0.02*	0.04*	0.25*	0.94	0.97	0.28*
I	Intercept	91.13	7.47*	1.40	96.60	1.10*	2.30
	Slope	-4.84	4.51	0.34	-5.09	5.10	-0.01*
	R^2	0.78	0.75	0.89	0.79	0.79	0.01*
N	Intercept	87.06	9.34	3.60	89.86	5.80	4.35
	Slope	-4.53	3.73	0.80	-4.98	4.39	0.59
	R^2	0.93	0.92	0.95	0.98	0.98	0.88
P	Intercept	88.04	10.36	1.60*	91.79	5.32*	2.89
	Slope	-4.49	3.81	0.68*	-4.05	3.93	0.12*
	R^2	0.98	0.95	0.55*	0.94	0.95	0.08*

*not significant at 5% significance level.



D: Disabled-incapacitating, F: Fatal injury, I: Injury-not incapacitating, N: Not injured, P: Possible injury

Figure 1. Intercept at different severity levels

Comparison of the Same Gender in Different Age Groups

The data have been analyzed using different severity levels as replicates for both females and males. Table 3 shows comparison for the same gender in different age groups. The letters in the parenthesis show whether the means are significantly different from each other or not. If they are the same letter, they are not significantly different and vice versa. There is no significant difference for females in age group 1, 2, and 3 for both TH and LT accidents. Age groups 4 and 5 are not significantly different from each other but significantly different from groups 1, 2, and 3 for both TH and LT accidents. There is no significant difference for all age groups for RT.

TH in age groups 1 and 2, 3 and 4, 4 and 5 are not significantly different from each other for males. LT in age groups 1 and 2, 2 and 3, and 4 and 5 are not significantly different. There is no significant difference for all age groups in the case of RT for males like that of females.

Table 3. Comparison of the same gender in different age groups

Age Group	Female			Male		
	TH	LT	RT	TH	LT	RT
30–49	82.18 (a)	15.34 (f)	2.48 (k)	90.52 (c)	7.12 (h)	2.36 (l)
50–64	83.96 (a)	13.74 (f)	2.30 (k)	87.99 (c)	9.59 (hi)	2.41 (l)
65–74	80.25 (a)	16.75 (f)	3.00 (k)	82.88 (d)	14.49 (i)	2.64 (l)
75–84	71.86 (b)	25.12 (g)	3.02 (k)	76.10 (de)	21.10 (j)	2.79 (l)
85+	66.63 (b)	29.28 (g)	4.09 (k)	70.83 (e)	26.26 (j)	2.91 (l)

Note: The same letters indicate that the means are not significantly different for different age group for the same gender

Comparison of the Same Age Group for Different Gender

Comparison has been made for females and males in the same age group for TH, LT, and RT accidents. TH and LT for females and males are not significantly different except for age group 1. RT for females is not significantly different from that of males for all age groups as shown in Table 4.

Table 4. Comparison of the same age group of different gender

Age Group	TH			LT			RT		
	Female	Male	Similar	Female	Male	Similar	Female	Male	Similar
30–49	82.18	90.52	No	15.34	7.12	No	2.48	2.36	Yes
50–64	83.96	87.99	Yes	13.74	9.59	Yes	2.30	2.41	Yes
65–74	80.25	82.88	Yes	16.75	14.49	Yes	3.00	2.64	Yes
75–84	71.86	76.10	Yes	25.12	21.10	Yes	3.02	2.79	Yes
85+	66.63	70.83	Yes	29.28	26.26	Yes	4.09	2.91	Yes

Effect of Light Condition

Most of the accidents have taken place during daylight as shown in Figure 2. The proportion of the accidents increase as age increases, and the magnitude is higher for females in all age groups during daylight. Since most of the accidents take place during daylight and dark while street lights are on, light condition is not the main cause of accidents for older drivers.

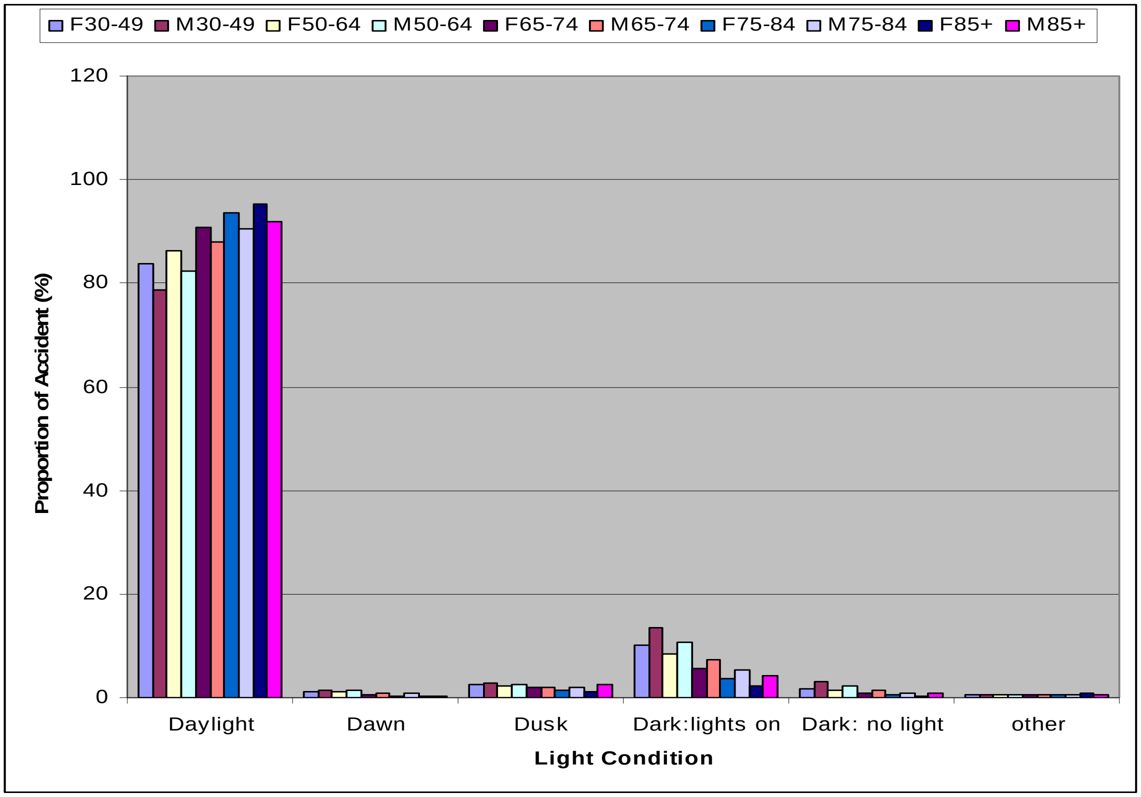


Figure 2. Effect of light condition

Effect of Weather Condition

Most of the accidents have taken place when there are no adverse conditions. The proportion of the accidents increase as age increases, and the magnitude is higher for females in all age groups when there are no adverse conditions, as shown in Figure 3. The second highest proportion of accidents have happened when there is rain, mist, or drizzle, whereas the magnitude reduces as age increases and is higher for females, unlike when there are no adverse conditions. The proportion of accidents during the rest of weather conditions is not significant. Since most of the accidents have taken place when there are no adverse conditions, weather condition is not the main problem for older drivers.

Effect of Surface Type

Effect of surface type is indicated in Figure 4. The highest proportion of accidents have taken place on blacktop, followed by concrete. The magnitude is the same on blacktop and concrete surface for almost all age groups. Accidents on the rest of surface types are not significant.

Effect of Surface Condition

The magnitude of the accident increases with age and is higher for female in all age groups on dry surface conditions, as indicated in Figure 5. Accidents decrease as age increases on wet surface conditions and are lower for females, unlike on dry surface condition. The remaining surface conditions have less contribution. Surface condition is not the main cause of accidents since most of the accidents have taken place on dry surface conditions.

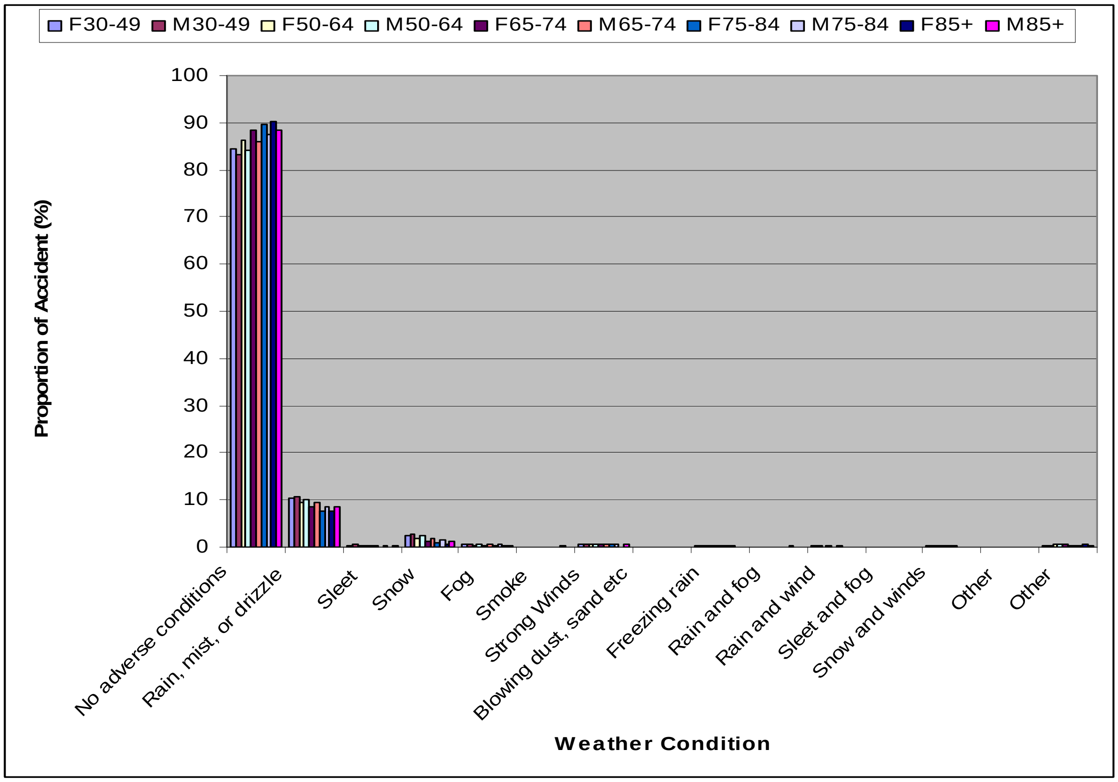


Figure 3. Effect of weather condition

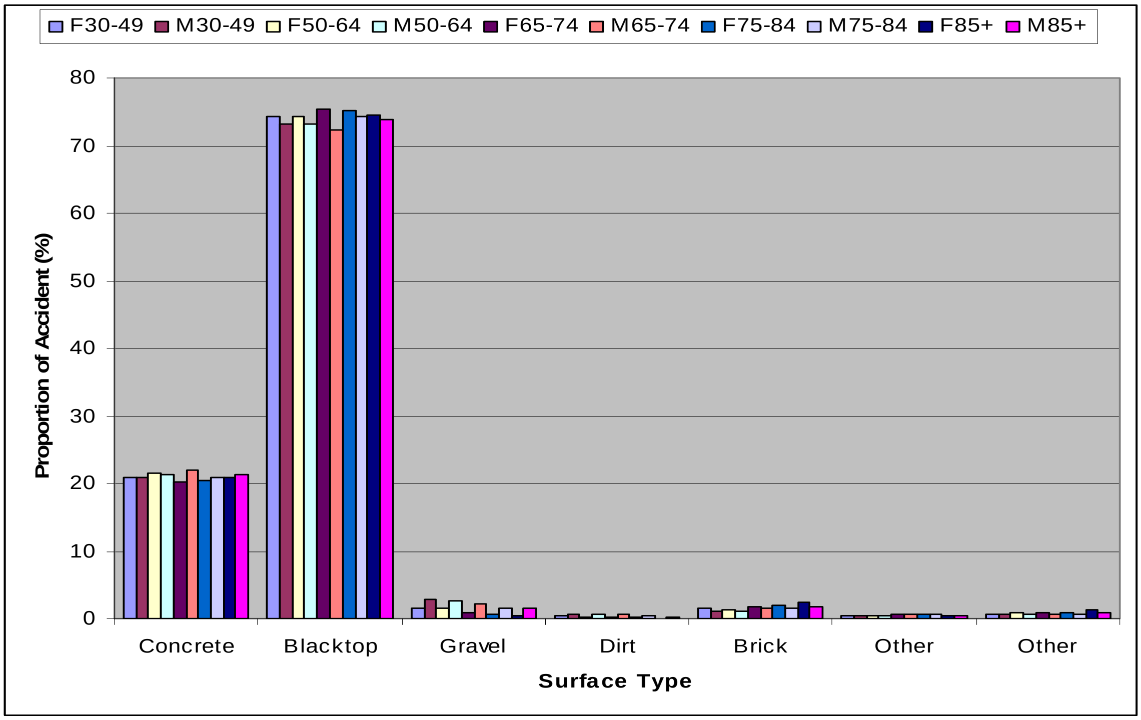


Figure 4. Effect of surface type

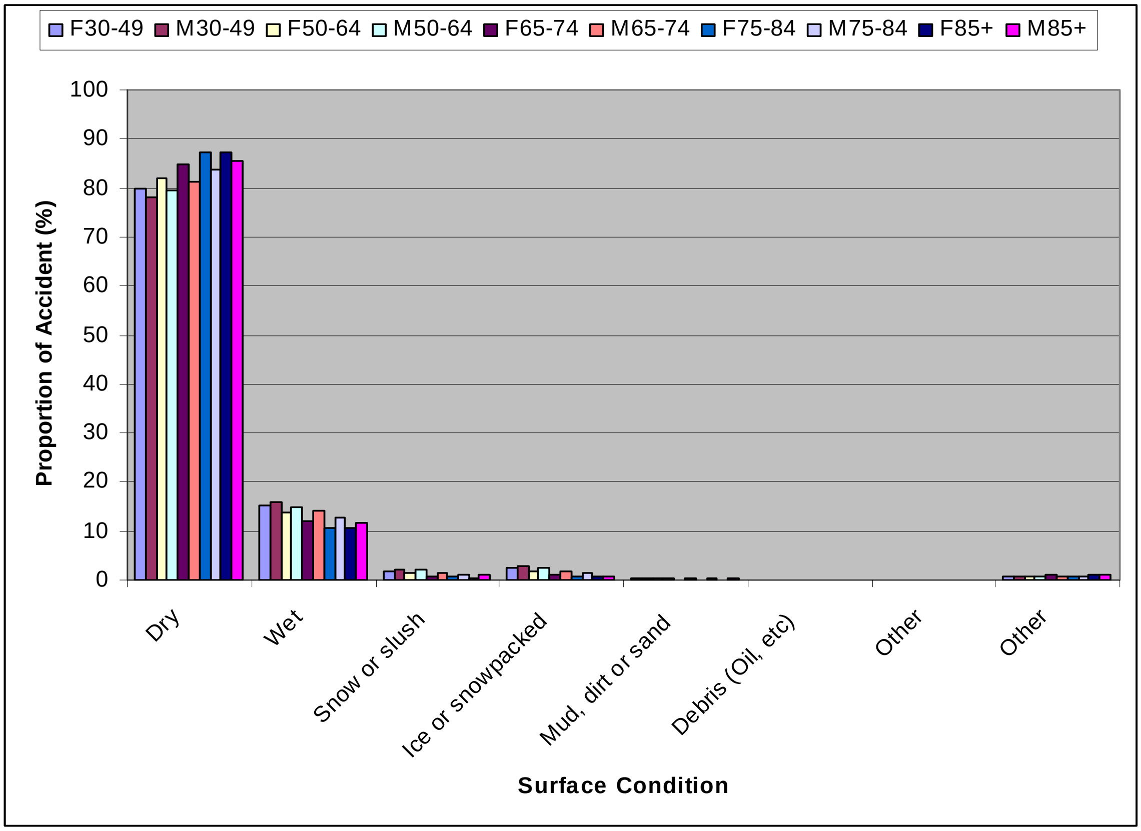


Figure 5. Effect of surface condition

Effect of Road Character

Most of the accidents have happened on straight and level roads, as shown in Figure 6. Magnitude slightly increases as age increases, followed by straight on grade roads. The magnitude for females is higher on straight and level roads, whereas it is equal on straight on grade except for the last age group. Accidents on the remaining road characteristics are insignificant.

Effect of Construction/Maintenance Zone

Figure 7 shows the effect of construction/maintenance zone. About 98% of the accidents have taken place on no construction/maintenance zone. The magnitude is the same in all age groups for both gender. The remaining zones cause insignificant accidents.

CONCLUSIONS

Ten years, 1993–2002, of accident data has been extracted from Kansas Accident Report System database and analyzed using SAS software for five different age groups. Five severity levels have been considered for both females and males. Based on this study, the following conclusions have been made:

- Proportion of old drivers in “through accidents” involved in accidents decreases as age increases, whereas the proportion of left-turn accidents increases with age.
- Left turn is harder for females in all age groups.

- Proportion of right turn accident has no specific trend, unlike through and left-turn accidents at signalized intersections.
- Most of the accidents have taken place during daylight when there are no adverse conditions on dry surface, straight and level road, and blacktop surface type.

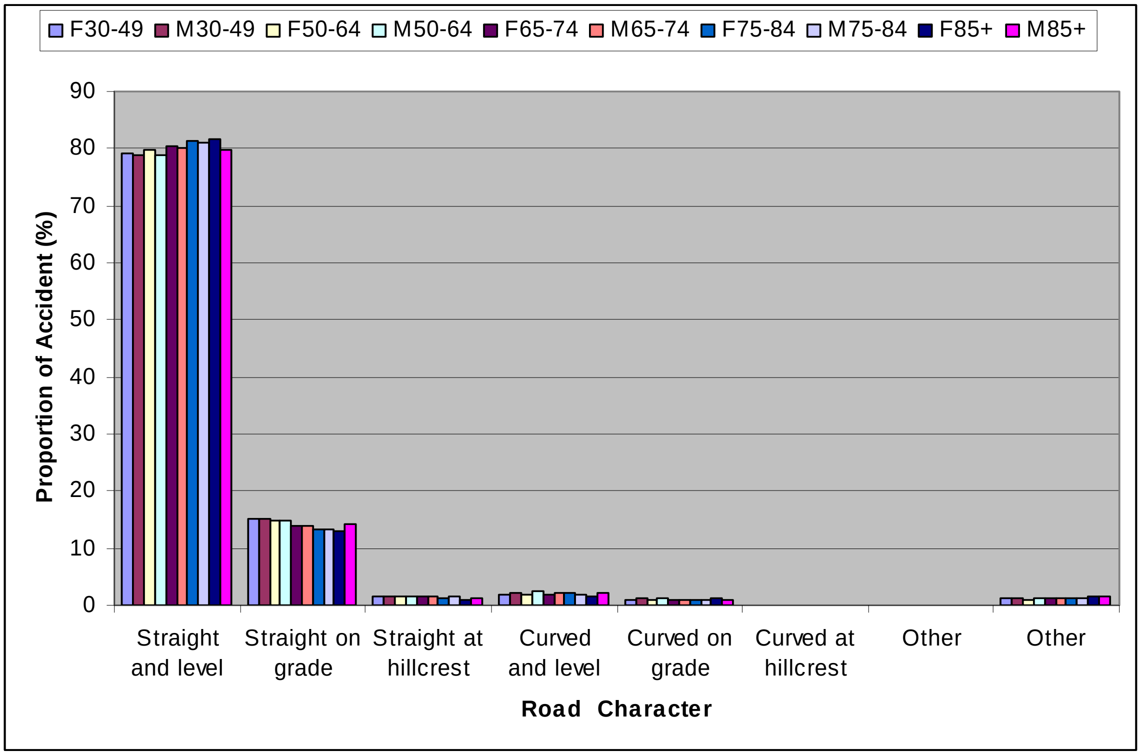


Figure 6. Effect of road character

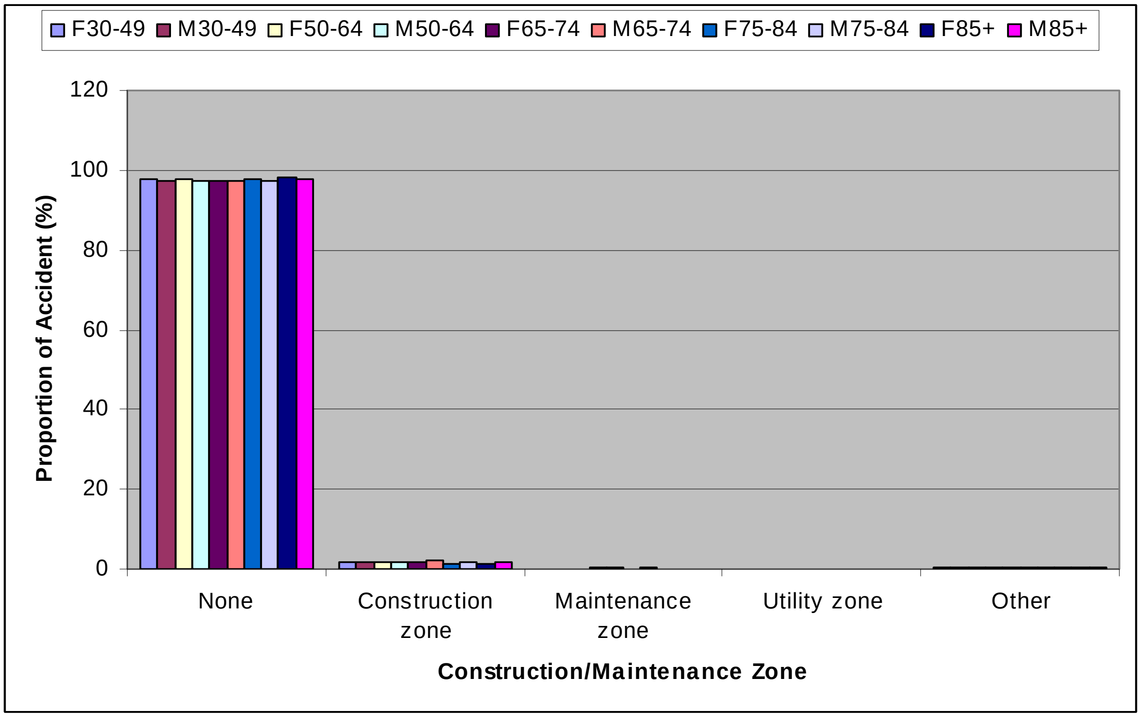


Figure 7. Effect of construction/maintenance zone

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Rider and Non-Rider Opinions of Rural Public Transportation

Brian Geiger
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
GeigerBC@ksu.edu

Sunanda Dissanayake
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
sunanda@ksu.edu

ABSTRACT

Public transportation in rural areas has been in existence for decades. Ridership levels, however, are quite low, often because many of the providers are still perceived as only for the elderly or disabled. Ridership is a factor of many things, including age, disability, income, car ownership, and other. Other intangible factors include quality of service, personal opinions, and issues and concerns about using public transportation. This paper primarily discusses the personal opinions of the riders and non-riders that may account for the lower ridership levels in rural Kansas.

In order to acquire the information about the characteristics and opinions of riders and non-riders of public transportation, two similar surveys were sent out with 445 rider and 557 non-rider surveys returned. The questions are either the exact same or as close as possible in order to compare the answers and opinions of the riders and non-riders. Findings of these two surveys can then be used to suggest transit service enhancements to increase the ridership of demand-response transportation.

Riders of demand-response transit systems in rural Kansas are pleased with the service provided as a whole. The majority of the riders fell between being content with the service to being ecstatic with the service provided. Non-riders are ambivalent toward demand-response transit service. They appreciate the fact that in many cases, general public transportation services exist, but they are also generally unwilling to use it themselves.

Key words: demand-response—public transportation—rural transit—transit ridership

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PROBLEM STATEMENT

Public transportation in rural areas has been in existence for decades. Ridership levels, however, are quite low, often because many of the providers are still perceived as only for the elderly or disabled. Ridership is a factor of many things, including age, disability, income, and car ownership. Other intangible factors include quality and type of service, personal opinions, attitudes, and issues and concerns about using public transportation. This paper primarily discusses the personal opinions of the riders and non-riders that may account for the lower ridership levels of demand-responsive transit services in rural Kansas.

Public transportation is generally perceived as urban fixed-route bus lines or subways. These public transportation facilities are typically centered in large population, high-density cities. However, bus lines and subways are not the only public transportation available. In less-populated and less-dense areas, public transportation providers exist that adapt to the rural nature of providing public transportation. These systems, often called demand-response systems, function differently than fixed routes.

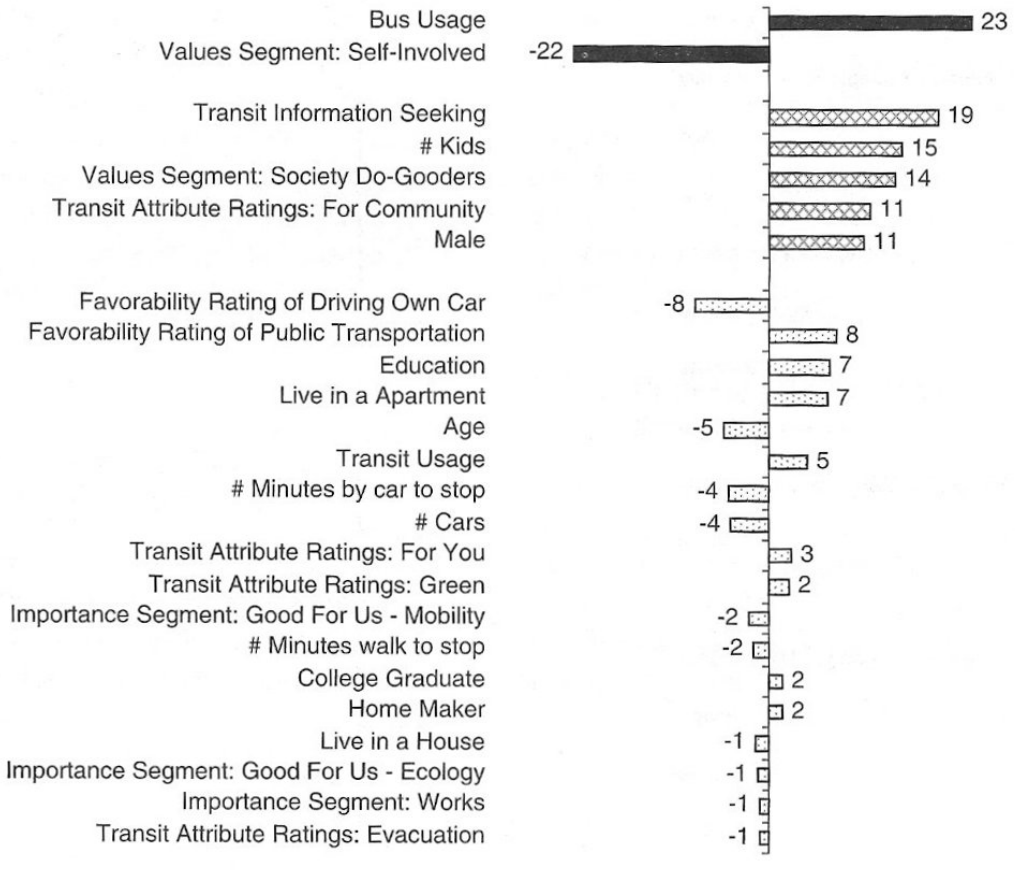
Demand-response transportation (also called paratransit or dial-a-ride) is transportation consisting of passenger cars, vans, or small buses that will show up at a location after a person has called in a request to the dispatcher. The buses are often wheelchair-lift equipped in order to be more accessible for those who cannot use the stairs. These vehicles do not run on a fixed route or timetable but rather will pick up and drop off people at requested origins and destinations as quickly and efficiently as they can. This often means that unlike taxis, they will pick up or drop off other passengers before continuing on to a rider’s intended destination (Silver 2007). These shared rides operate typically either door-to-door or curb-to-curb and from many origins to many destinations (Transportation Research Board 2003). Door-to-door means that drivers may assist passengers all the way up to, and sometimes inside of, the building the passenger is destined for. Curb-to-curb is where the drivers pull up to the curb of the building to pick up and drop off passengers, similar to a conventional taxi service.

The federal government provides financial assistance to rural transportation agencies through 49 U.S.C. Section 5311, often called Section 5311. Rural areas are defined as having a population smaller than 50,000. Federal funds are provided to the individual states who then distribute the funds to transportation agencies in rural areas. These funds cover up to 80% of the capital costs and up to 50% of operating expenses of the agency (Federal Transit Administration). A provider is not required to operate a demand-response service if it receives Section 5311 funds, but often that is the method used to operate cost effectively and serve the riders as efficiently as possible in rural areas. Section 5311 funds are for general public transportation providers as opposed to similarly distributed Section 5310 funds, as provided through 49 U.S.C. Section 5310, which focus on elderly and disabled riders. This report will primarily focus on Section 5311 providers who serve the general public through a demand-responsive operation system. Although these Section 5311 providers are for the general public, often their ridership consists mainly of the elderly and disabled and, therefore, will be discussed in more detail further in the report.

Many people who use the demand-response transit systems in rural areas are elderly or handicapped. Twenty-one percent of Americans older than 65 do not drive and more than 600,000 people age 70 and over stop driving each year nationwide (El Nasser 2007). This will only become more pronounced in the near future with the increasing age of the large baby boomer section of the U.S. population. The U.S. will be experiencing a significant increase in population of the elderly in the coming years. The elderly population age 65 and over was 34.5 million in 1999, and by 2030, there will be about 70 million elderly. (Sungyop and Gudmundur 2004). This significant shift in demographics will increase the number of potential drivers with reduced driving abilities. Abilities related to driving are a decrease in vision, an increase in response time, and other functions necessary for driving. Demand-response transportation should be available and widely publicized in order to be an encouraged alternative option to driving.

Transit agencies in rural areas are often focused on the elderly and disabled riders. This is due to the fact that most people in rural Kansas prefer to drive if possible and given the choice. It has been shown, however, that the elderly prefer modes other than public transit (Rhindress 2008). An AARP (formerly known as the American Association of Retired Persons) study found that “seniors aged 75 or older widely preferred driving. Elderly who are no longer drivers almost universally considered riding with friends or family the next best alternative” to driving themselves (Rhindress 2008). This preference for travel modes other than public transportation is something providers must overcome to increase their ridership.

Many factors affect whether or not a person will chose to ride transit in rural areas. Transit Cooperative Research Program (TCRP) Report 122: *Understanding How to Motivate Communities to Support and Ride Public Transportation*, created the chart shown in Figure 1 for low-density areas showing the relative power in driving support for transit from different attributes. Solid dark bars represent having more effect than those with cross hatching, which in turn have more effect than those with dotted bars. The most obvious finding is that those who use transit support it (Rhindress 2008). It also shows that if one rates driving his or her own car favorably or owns more cars, the less he or she supports transit. Age is also on the list, showing that the older you are the less you support transit. This may be due to transit being perceived as more difficult to use than other modes of transportation.



Relative power in driving support, positively or negatively.

Source: Rhindress, M. *Understanding how to Motivate Communities to Support and Ride Public Transportation*. Transit Cooperative Research Program, Report 122, 2008, pp. 15

Figure 1. Net impact on transit support in low-density markets

Potential public transportation clients' lack of information about public transportation is a common issue for transportation providers. A study in Washington, D.C., found that even when transit service is available for seniors, 47% stated they did not have enough information to use the services (Burkhardt et al. 1999). Even those who are part of city commissions and approve the budgets for public transportation do not always realize that public transportation is for everybody, not just the elderly or disabled (Smith 2008). Rural and small town residents though, tended to be more aware of alternative transportation modes than urban and suburban residents (Kostyniuk and Shope 2000).

Although one would like to think the elderly will stop driving when they no longer feel safe to drive, this isn't true in all cases. During a focus group with the elderly and their adult children, some of the elderly stated that "even if they knew they should stop, they would not and would keep driving 'until the end'" (Kostyniuk and Shope 200). Another stated, "I don't want to give up my license. Someone will have to take it before I give it up." While this lack of concern for others' safety is serious, perhaps it could be avoided with prior planning and encouragement to try public transportation modes. Discussions with adult children found that 75% of them feel that their relatives, typically their parents, either "do not know or will not admit when it is time to stop driving" (Kostyniuk and Shope 2000). Another study looked into older drivers age 65 and above that thought they would be stopping soon and could only find a single driving senior who thought they would stop driving within the next two years (Schatz, Stutts, and Wilkins 1999). They therefore had to re-write their criteria to include those who thought they might not be driving five years from then, as the elderly were highly unlikely to think about stopping driving on their own in the near future.

Thinking about ceasing or curtailing driving for the elderly is not something the elderly enjoy. The focus group comments from seniors or their adult children about older driver cessation conducted in five locations across the continental United States show how hard it is to get even those, who by most accounts should be using public transportation, to ride public transportation (Schatz, Stutts, and Wilkins 1999). Increasing ridership for public transportation is not just the responsibility of the transportation provider, it is a social and safety issue for all.

RESEARCH OBJECTIVES AND METHODOLOGY

To achieve the objective of this study, two surveys were created for the public. One was for riders of public transportation and the other for non-riders. The surveys collected demographic and opinion data for each survey in order to compare the two groups. Survey responses were then tabulated and conclusions were drawn from the data and opinions of the public.

Questions for both rider and non-rider surveys were developed based on knowledge gathered through the literature review. The starting point of the questions for both surveys was the U.S. Census Bureau's proposed 2010 questionnaire and its American Community Survey questionnaire (U.S. Census Bureau; U.S. Census Bureau 2008). In addition, questions from the Transit Performance Monitoring System (TPMS) results reports were used (American Public Transit Association 2002; American Public Transit Association 2004). By using similar-worded questions to previous large studies, it was assumed that it would decrease problems with the survey questions. Other questions were then added related specifically to transit and rural areas from perceived issues from the literature review (Sungyop and Gudmundur 2004; Schmocker et al. 2005; Crain and Associates 2000; Cherrington 2007).

Distributing the rider survey was fairly straightforward. The Kansas University Transportation Center (KUTC) contains a list of all transit providers in Kansas by county. The pertinent information from each listing was then recorded into spreadsheet format to be filtered. Criteria for transit providers to be contacted consisted that they must serve towns with a population less than 50,000 and have the "General

Public" clientele box checked on the KUTC site. From this shortened list, providers were contacted by phone and asked to distribute the surveys through their drivers to users of their system. The surveys were inside pre-paid, self-addressed envelopes to make it as easy as possible for riders to return them and to enable the highest possible response rate. There were 3,260 surveys distributed to transit providers in rural Kansas. There were 445 valid rider responses and 24 invalid rider responses received, generating a response rate of 14.4% for the rider survey. There were 1,735 surveys distributed to those willing to hand out non-rider surveys in rural Kansas. There were 557 valid non-rider responses and 28 invalid non-rider responses received, generating a response rate of 33.7% for the non-rider survey.

RIDER AND NON-RIDER COMMENTS AND OPINIONS

There are notable differences in the demographics and opinions of riders and non-riders who responded to the surveys. Table 1 shows that females were over-represented in both types of surveys as compared to the general population of the state of Kansas. Figure 2 shows that many of the riders of public transportation were elderly. These transit using elderly nearly 50% of the time do not have a current driver's license and nearly 40% of the time they have a handicapped parking permit.

Table 1. Gender of rider and non-rider survey respondents

	Riders		Non-Rider		State of Kansas
Gender	Frequency	Percentage (%)	Frequency	Percentage (%)	Percentage (%)
Male	96	21.6	167	29.98	49.6
Female	335	75.3	382	68.58	50.4
Invalid	1	0.2	0	0.0	-
Blank	13	2.9	8	1.44	-
Total	445	100.	557	100.	100

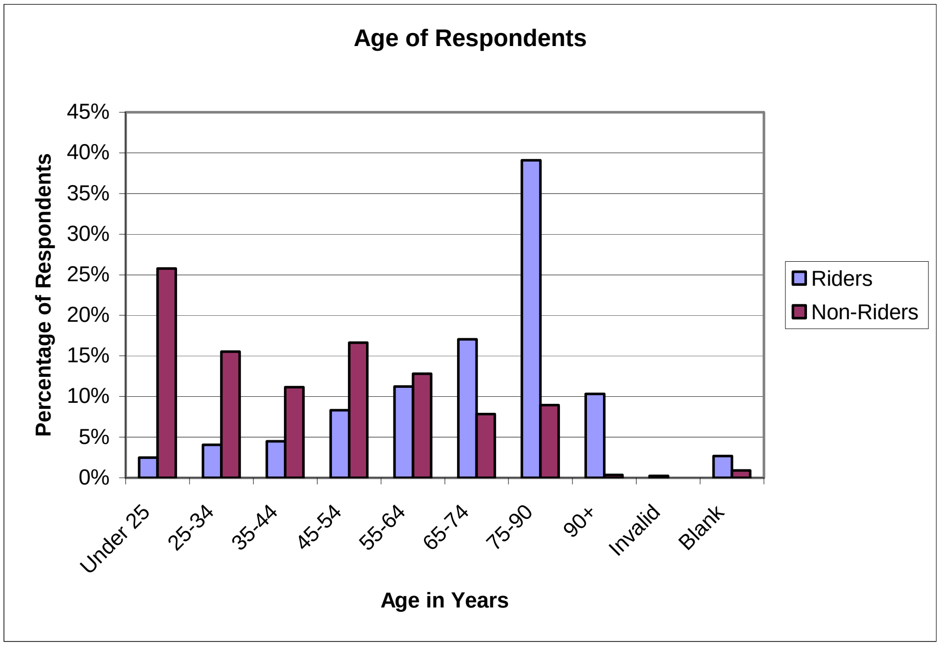


Figure 2. Age of rider and non-rider survey respondents

When non-riders had vehicle problems, they are not likely to turn to public transportation for mobility due to several other options, primarily the availability of personal vehicles that could be used instead (see Figure 3). Riders, however, were unlikely to turn to driving if transit was unavailable, as seen in Figure 4. Only 15% of riders said they would drive if transit was unavailable, with 25% of riders not taking their previous trip at all. Twenty-five percent of riders said they would ride with somebody else, implying that either they do not have a vehicle to drive or were unable to drive themselves. This was supported by further data that showed that 60% of riders of public transportation did not have a personal vehicle they could have used to make the trip in the first place.

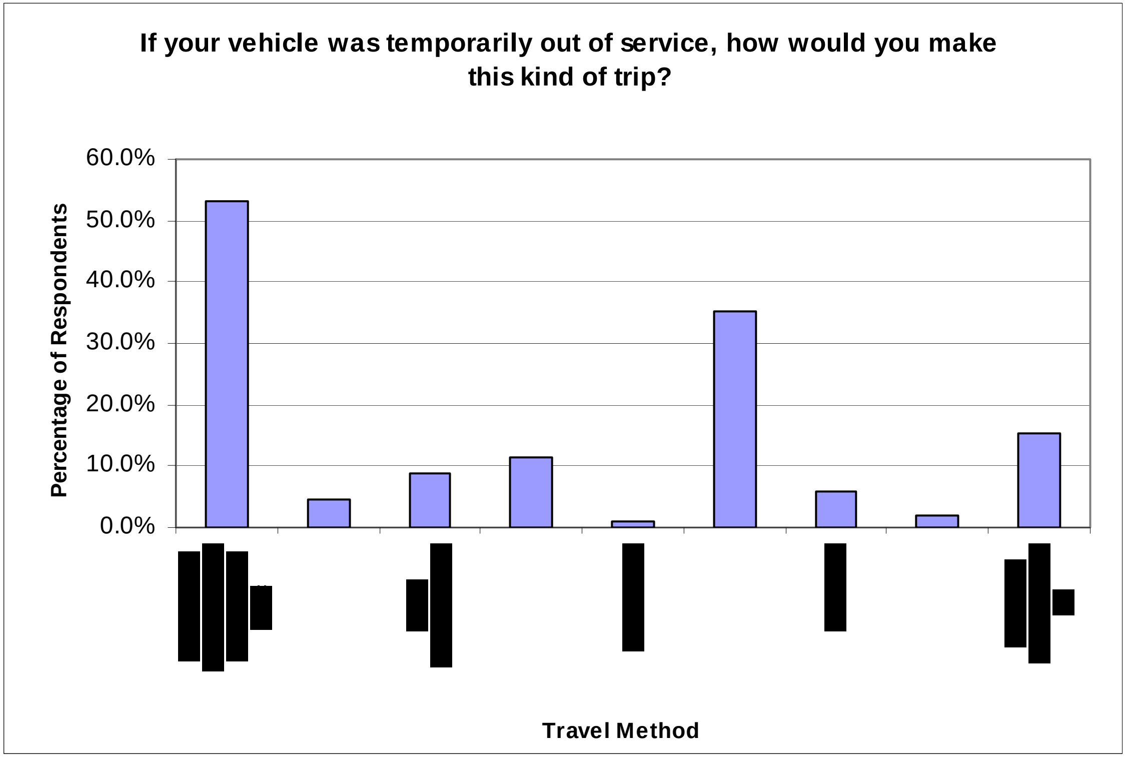


Figure 3. Non-rider alternative modes of transportation

Non-riders were asked if they knew a public transportation provider existed in their area. The majority (43%) said that no public transportation existed in their area, 20% said they didn't know, and 35% said that public transportation does exist in their area. The percentage that said that no public transportation exists in their area seemed higher than it should be considering almost every county in Kansas has a demand-response transit provider. Therefore, it is assumed that while non-riders do not know whether a service exists in their area, it may exist. It was later found that with increasing age of non-riders, about 55% of non-riders age 75 realized that public transportation exists in their area much more than those who were younger, where only about 25% of them realized public transit was available.

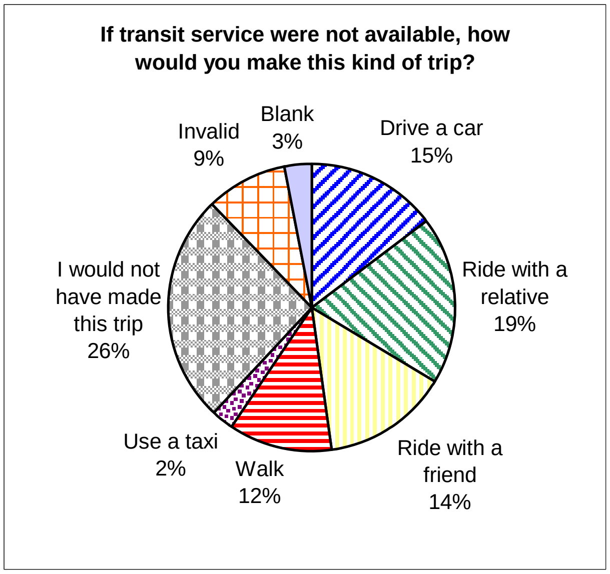


Figure 4. Rider alternative mode of transportation

Rider Opinions

Rider ratings of the transit services are given in Table 2, where it can be seen that they have a positive view of their service for each question asked. Two of the survey enhancements suggested for riders from these questions would be to decrease trip length and improve communication with the dispatcher. Decreasing trip length is often in contrast to operating efficiently from a provider's perspective, but these improvements are what riders are requesting. The questions were weighted based on the responses given to make them easier to compare. An "Always" answer was weighted at 5, a "Never" answer was weighted at 1, and "Invalid" and "Blank" were removed. The weighted averages were then ranked from highest to lowest. The ride length question was inversely weighted due to the nature of the question.

Table 2. Rider opinions of public transit

Survey Question	Weighted Average	Rank
Are drivers friendly and helpful?	4.836	1
Are drivers safe and competent?	4.823	2
Are drivers good at waiting for people to board the vehicle or assisting them in boarding the vehicle if needed?	4.814	3
Do you regard transit as affordable?	4.786	4
Is transit convenient?	4.706	6
Are you getting to your appointments on time?	4.683	5
Is the interior of the vehicle clean?	4.682	7
Are the seats on the vehicle comfortable?	4.662	8
Is the temperature of the vehicle comfortable?	4.634	9
Are buses easy to get into and out of?	4.586	10
Are you satisfied with the service you received from calling the transit dispatcher?	4.329	11
Is the ride longer than expected?	4.058	12
Would you recommend riding transit to a family member/friend,/neighbor/associate?	3.789	13

A further set of questions asked riders to order their priorities for improvements to the transit system. These have been weighted in Figure 5 with their top response given 5 points and their lowest response given 1 point. This is the inverse of ordering asked on the questionnaire. This question turned out to be more difficult than was first thought, with some responders leaving various answers blank, while others marked a single ranking through all of the options. Therefore, this is the average of all non-blank responses and will include the above issues in the average.

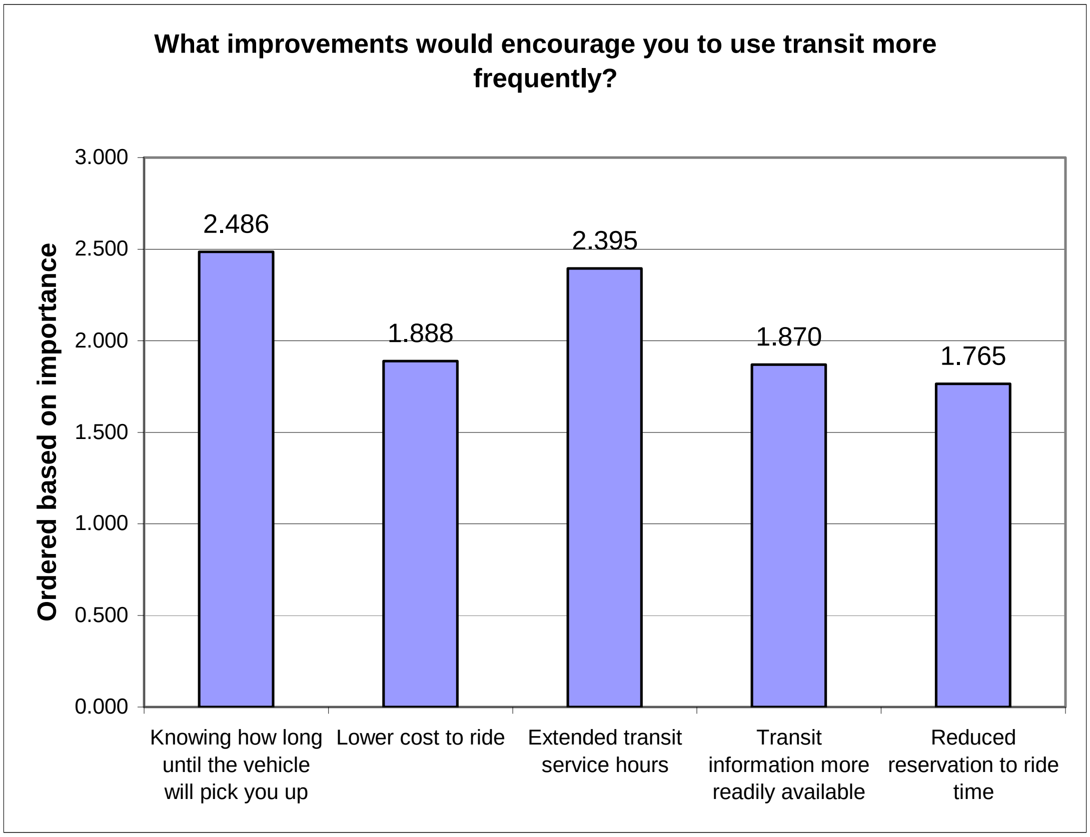


Figure 5. Rider transit improvement ideas

Riders' primary improvements would be to know how long it will be until the vehicle picks them up and to extend the operating hours of their transit service. Costs did not seem to be a significant concern of the riders, possibly because it is their only source of mobility. Shortening the time window that buses can pick up riders would let potential riders do things other than sit and watch for the bus to show up. Extending transit service hours was a typical request from riders in their written comments shown later in this paper, although it is illustrated here and can be compared easily with other possible service enhancements.

Rider Comments

The last three lines of the rider survey were reserved for written suggestions to improve the transit service. However, it turned into more of a general comments section, which proved equally insightful into the thoughts of the respondents. Total comments in this section were 249 out of 445 surveys returned. Select rider comments are included below.

"Drivers need to drive slower. If you're in a wheelchair and the back of the bus, you get bumped up and down a lot. That's hard on backs and necks, etc. Need better suspension, soft rides, for disabled. Thank You!"
"Everything is fine."
"Have no complaints."
"I am 95 years old and use a cane. I have trouble with my balance. I ride the mini bus and it is wonderful, and I am sure the transit buses are also and just what the people here need. Thank you."
"I am very pleased with the service—I have no complaint at all – good drivers, very kind and helpful—they make the trip go fast and also enjoyable as well. It is a wonderful service.
"I am very satisfied with our current service."
"I call, they come."
"I think that the dispatcher should stay out of personal medical problems."
"I would be selfish and out of line to ask for more hours. I feel so fortunate to enjoy the privilege to have this fine service."
"I'm most satisfied with schedule and dispatcher. Drive the BEST. Would like to see more people take advantage, maybe ads to others—seniors in community. Many thanks!!"
"I'm so happy we have buses. Otherwise I could not go to the center to eat."
"It is just fine."
"No improvements, they do a good job."
"None—only if could be available on Saturday and Sunday."
"Start 15 minutes earlier than 8 a.m. so I could be at work before 8 a.m."
"To be available for Sunday for church and Saturday."
"Weekend service."
"Would like the bus to run at least 10 p.m. weekdays."

The majority of the riders fell between being content with the service to being ecstatic with the service provided. Some riders described their personal situation and why they use the service. Often it was due to age or a physical handicap. A small number of riders wanted extended service hours. These requests included both hours of the day and days of the week service extension. Reactions were mixed on the drivers, with responses ranging from thanking them for their kindness to asking them to slow down and be more patient. This question was obviously very provider and driver specific, although overall showed drivers seem to be a non-issue for current riders in most cases. These comments aligned with the survey questions covering opinions of transit. As can be seen from the cross section of comments, users of public transportation in rural Kansas are generally quite pleased with the service being provided. The only two suggestions that arose throughout the comments were hours of operation and customer service. Riders typically wanted weekend, usually Saturday, service over longer daytime hours. A small number of riders had complaints about the dispatcher's service and friendliness, while an even smaller number had complaints about the driver.

Non-Rider Opinions

A large percentage of non-riders had never used public transportation in their area, or were unsure if it even existed in their area. This was aptly demonstrated in the high percentage of "Don't Know" responses each of the questions generated. When questioned about the cost of ownership for personal vehicles, or the increases in gas prices through the summer and fall of 2009, the results were mixed, although less people "Don't Know" and the answers were fairly evenly divided through the remaining five conventional answers.

The last question on the survey asked if non-riders would recommend public transportation to others. While a full 29% still didn't know if they would recommend it, the majority of the others responded either "sometimes" or more positively. Very few responded negatively, even when on a previous question 42% had said it was either "Never" or "Rarely" available when they needed it themselves. The underlying current seemed to be that public transportation was for other people in rural areas, but not oneself.

It can be seen in Figure 6 that non-riders would like greater geographic coverage from their transit service along with a lower cost to ride. Perhaps the most interesting answer is that while 43% of non-riders didn't know if it was hard to get information about transit; having transit information available would be the least likely to encourage them to ride, according to Figure 6. This seemed to show that many of the non-riders had no desire to ride, even if given information about transit.

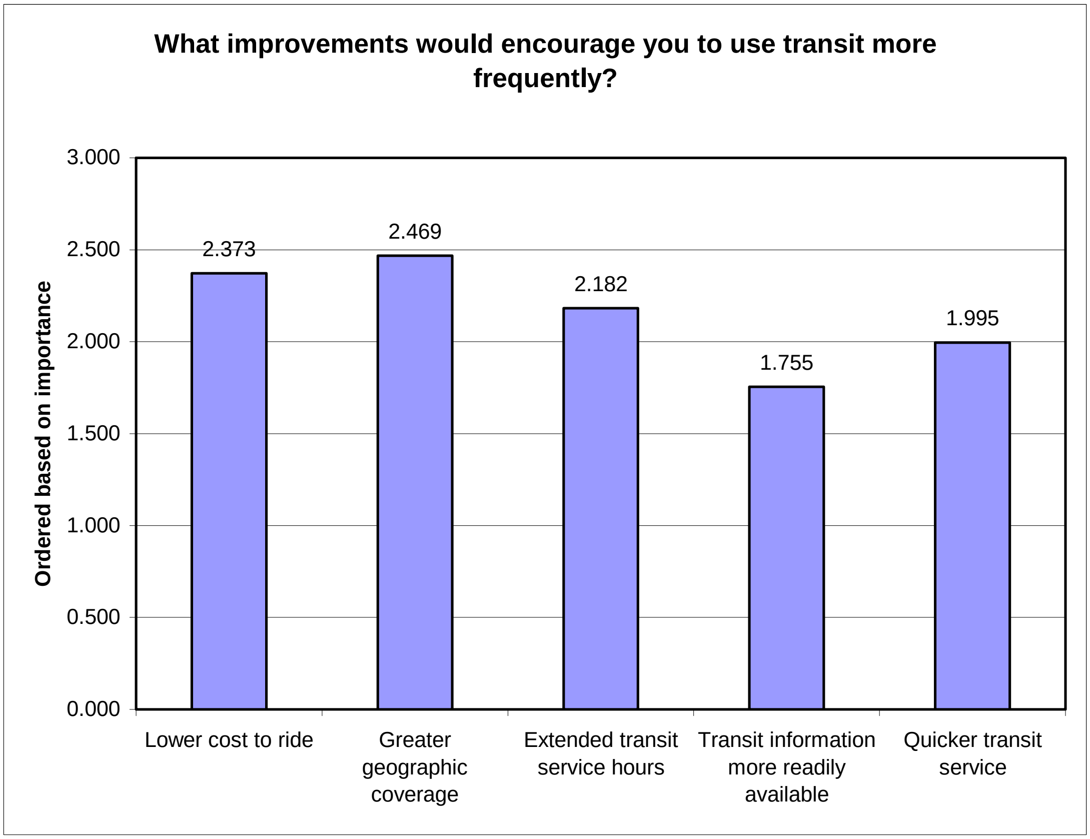


Figure 6. Non-rider transit improvement ideas

Even though a lower cost to ride was the second highest indicator to increasing transit usage, a similar question in Table 3 shows that only 3% of non-riders think transit was "Never" or "Rarely" affordable. These two responses conflict, leading one to assume that the majority of riders would not switch or use public transportation based on a single service improvement. It is doubtful that majority of the non-riders would use public transportation even if it were free because the service would not conform to the freedom of choice a personal vehicle allows people in rural areas.

Table 3. Non-rider opinions of public transit

Survey Question	Weighted Average	Rank
Is public transportation safe to ride in?	4.006	1
Do you regard public transportation as affordable?	3.958	2
Would you recommend using public transportation to a family member, a friend, neighbor or associate?	3.625	3
Is public transportation convenient?	3.441	4
In general, I avoid the use of public transportation if I can help it?	3.272	5
Are you concerned about the cost of owning a personal vehicle?	3.152	6
Does the recent increase in gas prices make you more likely to use public transportation?	2.906	7
Is it hard to get information about public transportation?	2.816	8
Is the bus late or unreliable in your opinion?	2.564	9
Is public transportation available when I need it?	2.136	10

Non-Rider Comments

The last three lines of the survey were reserved for suggestions to improve local public transportation. However, it turned into more of a general comments section, which proved equally insightful into the thoughts of the respondents. Out of 557 surveys returned, 236 non-riders made some sort of comment on in this section. A small cross section of comments is included.

Non-rider comments were much more diverse overall than rider comments. While it was more difficult to group large numbers of comments together, some types of comments did show up multiple times. Some non-riders said public transportation would be great in their area and they would like it to be available. These are interesting comments because they did not say they would use it, only that they wish it were available. Others said they lived in rural areas, so it wasn't practical for public transportation to operate that far out from the city. A few said it was only for seniors or disabled, but most seemed to indicate it was not available at all.

- "Available 7 days/week until 6 p.m. Have at least one day per month (possibly the 1st) when someone would carry in packages for me."
- "Better paved street, better drainage of rain water in street on Pierre."
- "Buses are more for senior citizens. Most younger people won't use it."
- "Expanded hours/days."
- "Fixed routes."
- "I don't use it. So, I don't care."
- "I live in a rural area. I don't see public transportation ever working here."
- "I live in a small town. The city bus customers are mainly the elderly or handicapped."
- "I live in town, but I farm full-time. My farthest piece of land is 45 miles from home. Public transportation is not a viable option for me. I drive a lot of miles every day."
- "I was unaware of Manhattan's public transit system until recently. I am afraid that most people are likewise unaware of its existence. Better advertising would be advised."
- "I would like to see public transportation in our area period."
- "It is not available."

- “I've not thought about it.”
- “Like to actually have one!”
- “None in rural area—limited amount for seniors, I think.”
- “Not interested at all at this time. 20 blocks from work. Ask me again in 20 years!”
- “Nothing unless you can lower the cost of gas.”
- “The closest location we have is 30 miles south of here. I wouldn't ride the bus anyway—I prefer to drive myself—it's convenient—but it's a great service for those that don't have a vehicle.”
- “To establish service to the aging rural population.”
- “We are in such a rural area—I can't see it would be used except by the elderly with no relatives.”

Results from the open-ended question to non-riders showed a wide range of opinions. Some people stated that it didn’t exist in their area, which may or may not be true. Others wanted longer service hours and fixed routes. Some people just wanted to have “it,” which they may not realize are not fixed-route services and are instead demand-response for rural areas.

CONCLUSIONS

Riders of demand-response transit systems in rural Kansas are pleased with the service provided as a whole. The only repeated suggestion or complaint the riders provided was their desire for increased operating hours and days. All areas of questions about using public transportation systems in rural Kansas scored well with riders.

Ridership is significantly skewed toward the elderly, disabled, and those who either choose not to drive or are unable to drive. For most of the riders, public transportation is their only reliable method of mobility and they are transit dependent for mobility. Only 15% of riders would drive themselves if public transit were unavailable. Other methods either take longer or constantly require asking for favors from others to drive them around.

Non-riders are ambivalent toward demand-response transit service. They appreciate the fact that in many cases general public transportation services exist, but they are also generally unwilling to use it themselves. Interest in public transit among non-riders is low, even when it was obvious many non-riders did not know if a transit service existed in their area or thought it did not exist. These are typically choice riders and are unlikely to switch to demand-response transit due to their other mobility options. Many non-riders recognized the fact that the elderly in particular use the service, and a handful put forward that they may even use it themselves in the future as they increased in age. In their current state though, non-riders typically have access to a personal vehicle and find the hassle of calling in advance and then waiting for a public bus in rural areas less convenient than just driving themselves.

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Effectiveness of a HAWK Beacon Signal at Mid-Block Pedestrian Crossings

Ranjit Prasad Godavarthy
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506-5000
ranjitg@ksu.edu

Eugene R. Russell
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506-5000
geno@ksu.edu

ABSTRACT

Pedestrian signals, particularly at signalized, mid-block crossings can cause delay to a driver, which is termed "excessive delay" in this study. In many cases at a mid-block signal, a pedestrian pushes the button and then quickly crosses the street as soon as the walk signal appears, leaving drivers still facing several seconds of solid red ball and by law, must remain stopped, even though no pedestrians remain in the crossing (i.e., "excessive delay"). On a busy street, a queue of vehicles waiting after all pedestrians have crossed can amount to hundreds of hours of excessive delay per year. The **H**igh-intensity **A**ctivated cross**W**al**K** (HAWK) beacon signal, which is now proposed to be called a "pedestrian hybrid signal" by the Federal Highway Administration (FHWA) in the next *Manual on Uniform Traffic Control Devices* (MUTCD) (expected 2009), is proven to be effective in decreasing this excessive delay by its different sequence of signal operation. The city of Lawrence was interested in experimenting with the HAWK beacon signal, and so they installed one at a mid-block crossing. A study was conducted at this site to find out the effectiveness of this HAWK beacon signal in decreasing the delay to drivers by comparing it with a similar signalized mid-block crossing in the city. Video cameras were used to capture video at these sites, and the effectiveness of a HAWK beacon signal to reduce excessive delay was analyzed from the videotapes. The HAWK beacon signal proved to be effective in decreasing the excessive delay to the drivers in this study.

Key words: excessive delay—HAWK beacon—mid-block signals

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Backcalculation of Pavement Moduli Using Bio-Inspired Hybrid Metaheuristics and Cooperative Strategies

Kasthurirangan Gopalakrishnan
Department of Civil, Construction, and Environmental Engineering
Iowa State University
354 Town Engineering
Ames, IA 50011-3232
rangan@iastate.edu

EXTENDED ABSTRACT

Biologically inspired computing or natural computing is a field of research that takes inspiration from nature, biology, physical systems, and social behavior of natural systems for developing computational techniques to solve complex optimization problems. For instance, one of the most well-established nature-inspired heuristic techniques is the genetic algorithm (GA), which is based on the survival-of-the-fittest notion espoused by Darwin's theory of evolution. Similarly, the ant colony optimization (ACO) approach imitates the real-world foraging behavior shown by ants when they search for food, and particle swarm optimization (PSO) is inspired by social behavior of bird flocking or fish schooling. In recent years, such nature-inspired metaheuristics are emerging as successful alternatives to more classical approaches for solving optimization problems that contain uncertainty, stochasticity, and dynamic information in their mathematical formulation.

Some well-known nature-inspired metaheuristics, which are basically high-level strategies that guide the search process to efficiently explore the search space in order to find (near-) optimal solutions, include but are not limited to GA, PSO, ACO, simulated annealing (SA), shuffled complex evolution (SCE), artificial immune systems (AIS), etc. The problem of pavement layer moduli backcalculation, in the context of pavement nondestructive testing (NDT), involves searching for the optimal combination of pavement layer stiffness solutions in an unsmooth, multi-modal, complex search space, which makes it amenable to the application of nature-inspired cooperative global optimization strategies.

Over the years, several techniques have been proposed for backcalculation of pavement layer moduli, such as the least-squares (parameter identification), database search, artificial neural networks (ANNs), neuro-fuzzy systems, and GAs. Applications of two nature-inspired hybrid optimization approaches to pavement backcalculation—which take advantage of the combined efficiency and accuracy achieved by integrating advanced pavement numerical modeling schemes, computational-intelligence–based surrogate mapping techniques, and stochastic nature-inspired metaheuristics with global optimization strategies using a system-of-systems approach, and yet provide a user-friendly pavement evaluation toolbox for the pavement engineer to use on a real-time basis for accurate evaluation of transportation infrastructure systems—are illustrated in this paper

Key words: backcalculation—bio-inspired computing—hybrid system—natural computing—pavement

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INTRODUCTION

The objective (fitness) function or the cost function for the proposed hybrid optimization approach is the difference between measured falling weight deflectometer (FWD) deflections and computed pavement surface deflections. In this paper, the implementation of the hybrid optimization approach is discussed for a three-layered flexible pavement structure, although it can be used for other pavement types with varying number of layers owing to its flexible and integrated modular systems approach. A typical three-layered flexible pavement structure consists of a hot mix asphalt (HMA) surface layer, a granular base layer consisting of unbound aggregates, and the bottommost layer consisting of subgrade soils.

In the proposed hybrid optimization approach (see Figure. 1), a trained neural network (NN) serves as a surrogate forward pavement response model that has learned the mapping between pavement layer elastic moduli and resulting pavement surface deflections for a variety of case scenarios generated using a 2-D axisymmetric pavement finite element program (Raad and Figueroa 1980). The stochastic global optimization (SGO) algorithm (GA, PSO, SCE, etc.), in essence, finds the optimal values of the NN inputs (pavement layer moduli) iteratively, such that the corresponding values of the network outputs (deflections) match the measured pavement surface deflections to minimize the differences between the measured and computer deflections. Although the error-minimization deflection-based objective function can be defined in a number of ways, a simple objective function representing sum of the squared differences between measured and computed deflections, as shown in equation (1), was selected for this study (where n = 6):

$$f = \sum_{i=1}^{n} (D_i - d_i)^2 \qquad (1)$$

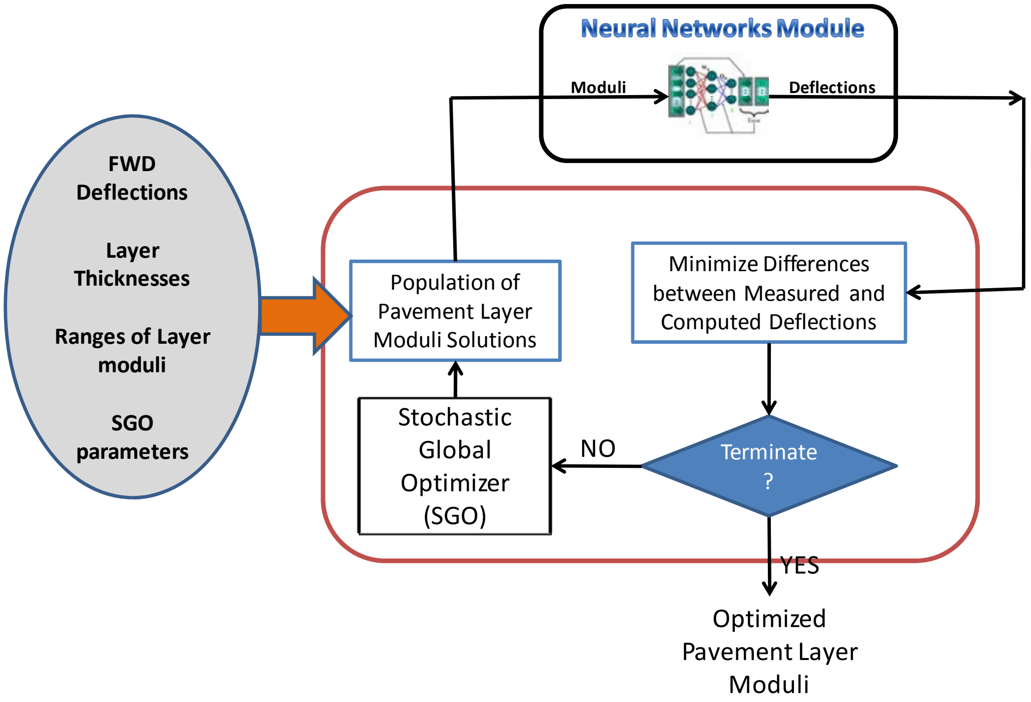
Neural Networks Module
Moduli
Deflections
FWD Deflections
Layer Thicknesses
Ranges of Layer moduli
SGO parameters
Population of Pavement Layer Moduli Solutions
Minimize Differences between Measured and Computed Deflections
Stochastic Global Optimizer (SGO)
NO
Terminate ?
YES
Optimized Pavement Layer Moduli

Figure 1. Implementation of hybrid optimization approach for pavement layer backcalculation

The hybrid optimization framework was implemented in MATLAB (Gopalakrishnan 2009). The input variables include six FWD measured surface deflections at 300 mm radial offsets starting from the center of the FWD loading plate, HMA surface and base layer thicknesses, and the corresponding min-max ranges of pavement layer moduli. In the following sections, the individual SGO algorithms and their corresponding results are discussed for two nature-inspired optimization strategies.

PARTICLE SWARM OPTIMIZATION (PSO)

PSO is a type of artificial intelligence method based on the collective behavior of decentralized, self-organized systems has been proved to be an efficient method for many global optimization problems, and in some cases, it does not suffer the difficulties encountered by other evolutionary computation techniques. The PSO concept introduced by Kennedy and Eberhart (1995) draws its roots from artificial life (A-life), bird flocking, fish schooling, swarming theory, as well as genetic algorithms and evolutionary programming. Although it was originally introduced for optimization of nonlinear continuous functions, many advances in PSO development has enabled it to handle a wide class of complex engineering and science optimization problems.

Similar to GAs, a population of potential solutions to the problem under consideration is used to probe the search space in PSO. However, each individual of the population in PSO has an adaptable velocity (position change), according to which it moves in the search space. Moreover, each individual has a memory, remembering the best position of the search space it has ever visited (Eberhart and Shi 1998). The movement of the individual is thus an aggregated acceleration toward its best previously visited position and toward the best individual of a topological neighborhood. Since the “acceleration” term was mainly used for particle systems in particle physics (Reeves 1983) and the term “swarm” for describing population, this algorithm was named as particle swarm optimization. In essence, PSO employs a swarm of particles or possible solutions that fly through the feasible solution space to explore optimal solutions.

Hypothetical data covering wide ranges of layer thicknesses and FWD deflections commonly encountered in the field were first used to evaluate the prediction accuracy of the developed NN-PSO hybrid backcalculation tool. A total of about 150 datasets were independently selected from the comprehensive synthetic FE solutions database to assess the prediction performance. The performance of NN-PSO optimization approach in backcalculating flexible pavement layer moduli is reported in Figure 2. As shown in the plots, all 150 neuro-swarm–based backcalculation predictions fell on the line of equality for the two pavement layer moduli, thus indicating a proper training and very good performance of the proposed hybrid backcalculation model.

SHUFFLED COMPLEX EVOLUTION (SCE)

The SCE algorithm developed at the University of Arizona is reported to be an efficient global optimization method that can be used to handle nonlinear problems with high-parameter dimensionality (Duan et al. 1992; Duan et al. 1993; Duan et al. 1994; Muttil and Liong 2004). It consists of all the four principles for global optimization: the controlled random search, the implicit clustering, the complex shuffling, and the competitive evolution. The search for the optimal solution begins with a randomly selected complex of points spanning the entire feasible space. The implicit clustering helps to concentrate the search in the most promising of the regions. The use of complex shuffling provides a freer and more extensive exploration of the search space in different directions, thereby reducing the chances of the search getting trapped in local optima. Three of these principles are coupled with the competitive complex evolution (CCE) algorithm, which is a statistical reproduction process employing the complex geometric shape to direct the search in the correct direction. The synthesis of these concepts makes the SCE algorithm not only effective and robust but also flexible and efficient (Nunoo and Mrawira 2004).

The performance of neuro-SCE (NESCE) optimization approach in backcalculating flexible pavement layer moduli is reported in Figure 3. As shown in the graphs, all 150 NESCE-based backcalculation predictions fell on the line of equality for the two pavement layer moduli, thus indicating a proper training and very good performance of the proposed hybrid backcalculation model.

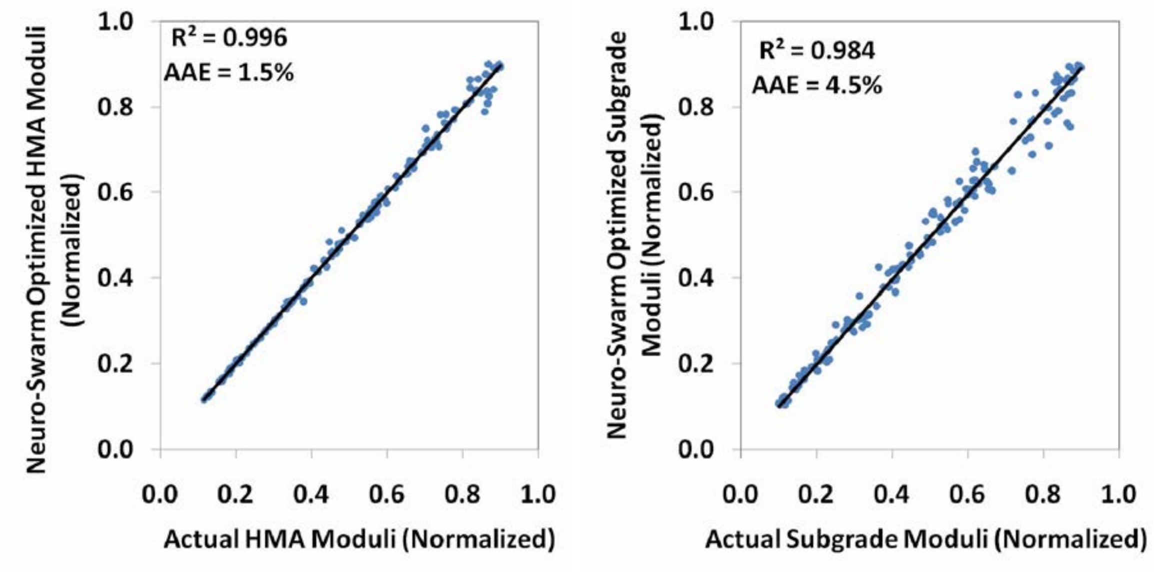

Figure 2. Backcalculation of pavement moduli using hybrid PSO approach

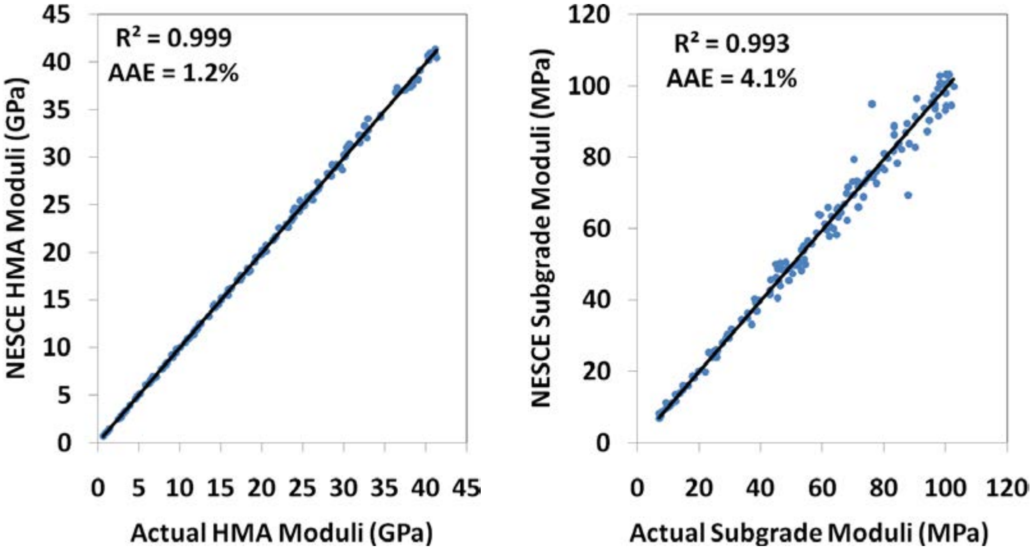

Figure 3. Backcalculation of pavement moduli using hybrid SCE approach

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Food Urbanism: A Sustainable Design Option for Growing Urban Communities in Iowa

Jason Grimm
Department of Landscape Architecture
Iowa State University
146 College of Design
Ames, IA 50011
jason.greenarch@gmail.com

ABSTRACT

This project states that food—its production, movement, marketing, and consumption—is urban infrastructure and can both organize and transform the urban experience through thoughtful, sensitive design of this infrastructure. Continuous urban agriculture can be a rigid and ecological backbone in a community and guide new urban growth sustainably on the urban fringe while connecting neighborhoods, open spaces, and urban markets. This idea is expressed as "Food Urbanism." Research is based on case studies, interviews of producers and city officials in Ames, IA, and studies of urban agriculture (UA) in London, UK. This research demonstrates that urban food systems have a potential of creating environmentally, socially, and economically productive communities in Iowa.

Key words: food urbanism—urban agriculture—urban infrastructure

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Continuous Productive Urban Landscapes: A Sustainable Design Option to Growing Urban Communities in Iowa

Jason Grimm
Department of Landscape Architecture
Iowa State University
146 College of Design
Ames, IA 50011
jason.greenarch@gmail.com

ABSTRACT

As designers and planners of urban landscapes, landscape architects hold a vital tool in the growth of any Iowa community. Both locally and globally food has become a common theme in many discussions. Motivations include the lack of productive urban land, lack of societal knowledge of food growing and preparation, urban/rural conflict at the urban fringe, food insecurity, lack of stable urban markets, and uncontrolled urban growth.

As a senior thesis in landscape architecture, the goal was to research and design based on the theory *Food Urbanism: the study of how food relates the organization of a city and how it can become infrastructure that can transform the urban experience*. Continuous productive landscapes could become a tool and/or mechanism to sustainable growth in urban communities. As infrastructure in a city or town, continuous urban agriculture (UA) has the potential of being a thread that is sewn through a community creating a rigid and ecological backbone to growth that connects neighborhoods, open spaces, and urban markets. Research is based on case studies, interviews of producers, and city officials in Ames, IA, and studies of UA in London, UK. Productive landscapes as tools to sustainable growth have only recently been written about in the United States and Canada (see Figure 1). This research demonstrates that urban food systems have a potential of creating environmentally, socially, and economically productive communities in Iowa.

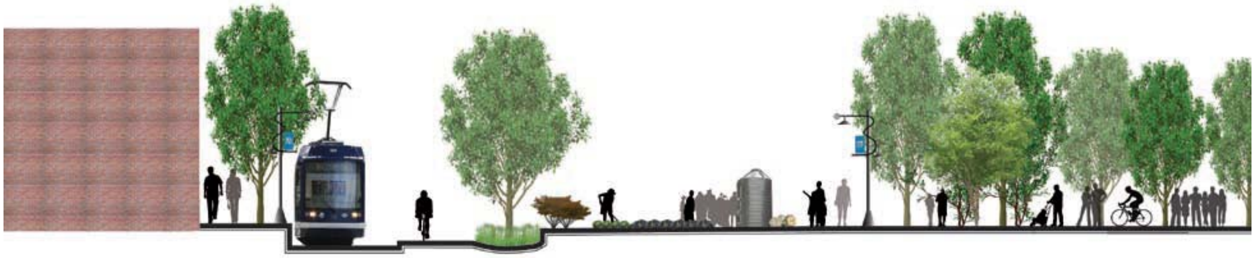

Figure 1. Typical section of continuous productive urban landscape

Key words: landscape architecture—planning—sustainable agriculture—urbanism—urban food systems—urban land inventory

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Multimodal Condition Assessment of Bridge Decks by NDE and Its Validation

Nenad Gucunski
Department of Civil and Environmental Engineering
Rutgers University
623 Bowser Road
Piscataway, NJ 08854
Gucunski@rci.rutgers.edu

Ruediger Feldmann
Center for Advanced Infrastructure and Transportation
Rutgers University
100 Brett Road
Piscataway, NJ 08854
feldmann@rci.rutgers.edu

Francisco Romero
Center for Advanced Infrastructure and Transportation
Rutgers University
100 Brett Road
Piscataway, NJ 08854
faromero@rci.rutgers.edu

Sabine Kruschwitz
Center for Advanced Infrastructure and Transportation
Rutgers University
100 Brett Road
Piscataway, NJ 08854
skruschw@rci.rutgers.edu

Ahmad Abu-Hawash
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Ahmad.Abu-Hawash@dot.iowa.gov

Mark Dunn
Research and Technology Bureau
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Mark.Dunn@dot.iowa.gov

ABSTRACT

Corrosion-induced delamination is a common problem in reinforced concrete bridge decks. The Iowa Department of Transportation initiated a research project on condition assessment of bridge decks using nondestructive evaluation (NDE) technologies with two primary objectives. The first objective was to demonstrate the use of NDE in the condition assessment on a sample of bridges. The second objective was to validate results of the NDE-based condition assessment through a series of "ground truth" and other complementary measurements. Surveying of bridge decks by NDE concentrated on the complementary approach using impact echo (IE) and ground penetrating radar (GPR) techniques. The approach exploits the speed of GPR surveys in identifying deteriorated bridge deck areas and accuracy of IE to detect and characterize delaminations in the deck. As a result of the GPR survey, often conducted at close to highway speeds, a preliminary condition assessment of the deck is made based on the signal attenuation at the rebar level. To demonstrate similarities and differences in the results obtained using different antenna types, both air-coupled (horn) and ground-coupled antennas were used during these surveys. The IE testing was conducted both as point testing using a portable seismic property analyzer (PSPA) and line testing using an automated tester called Stepper. The results of the IE testing, together with other NDE and ground truth data, were used to assist in defining a suitable GPR deterioration threshold separating sound concrete from deteriorated sections within the deck. To both obtain a comprehensive condition assessment of the tested decks and to validate and enhance the interpretation of the GPR and IE evaluation results, a number of complementary NDE and destructive techniques were

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utilized, including ultrasonic pulse echo, half-cell potential, electrical resistivity, coring and deck autopsies.

Key words: bridge deck—condition assessment—nondestructive evaluation (NDE)—ground penetrating radar (GPR)—impact echo—half cell—electrical resistivity—ultrasonics—coring

INTRODUCTION

Bridges can and should be approached as critical nodes in the greater highway network. As such, the development and implementation of means for their rapid rebuilding, nondestructive inspection, and performance monitoring is even more critical than for most other components of the transportation network. This is especially true for bridge decks, for which providing means for rapid, nondestructive, and accurate condition assessment and performance monitoring will make a tremendous difference in the financial resources spent for their renewal and frequency and duration of traffic interruptions. The data collected from nondestructive testing (NDT) of bridge decks should complement other information for understanding of its life-cycle costs, deterioration mechanisms, and the effectiveness of preservation techniques at various stages of the aging process and most importantly, prevent premature and unexpected failure.

The dominant practice by state departments of transportation in evaluation of bridge decks is by visual inspection and the use of simple nondestructive methods like chain drag and hammer sounding. Modern nondestructive evaluation (NDE) of concrete and concrete bridge decks has its origins in geophysics. A number of techniques introduced exploit various physical phenomena (acoustic, seismic, electric, electromagnetic, thermal, etc.) to detect and characterize specific deterioration processes or defects. In general, all the techniques utilize an approach where the objective is to learn about the characteristics of the medium from its response to the applied excitation. One of the biggest challenges of nondestructive evaluation of concrete bridge decks, and concrete in general, is that it is more complex than evaluation of metals. It is so for several reasons, but primarily due to the composite material nature of concrete and of a quality, it is not as easily reproduced as it is for metals. At the same time, while the nondestructive evaluation of metal members dominantly concentrates on detection, characterization, and monitoring of cracks, concrete decks have a whole suite of deterioration processes and defects that require a diverse set of techniques for their detection and monitoring. The complexity of concrete deck evaluation and monitoring and the need for its nondestructive evaluation was best illustrated in a series of SHRP reports in the early nineties (S-323, S325, S-326, S-327, S-330) and the NCHRP Synthesis 333 Report on concrete bridge deck performance (2004).

The Iowa Department of Transportation (DOT) is sponsoring a research study that has a primary objective evaluation of the effectiveness of NDE technologies in detecting and characterizing concrete bridge deck deterioration. For that purpose, decks of nine bridges throughout the state of Iowa (see Figure 1) have been surveyed using a suite of NDE technologies. While the study concentrated on the use of impact echo (IE) and ground penetrating radar (GPR), other NDE technologies were implemented during the surveys, including ultrasonic testing (ultrasonic pulse echo [UPE] and ultrasonic surface waves [USW]), half-cell corrosion potential (HCP), and resistivity measurements (RM). Ground-truth calibrations and validations or measurements were provided through examination of cores taken from the surveyed decks, typically four 4-inch cores per deck.

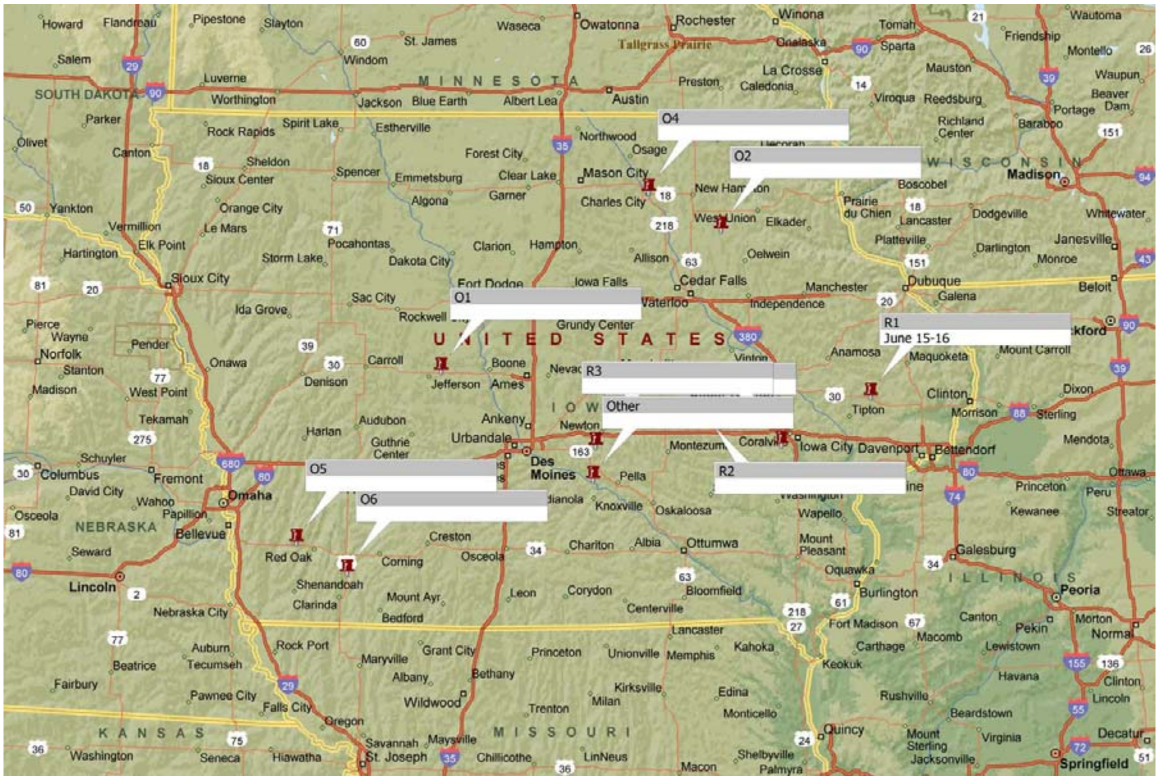

O4
O2
O1
R1
June 15-16
R3
Other
R2
O5
O6
M I N N E S O T A
SOUTH DAKOTA
W I S C O N S I N
U N I T E D S T A T E S
I O W A
NEBRASKA
I L L I N O I S
K A N S A S
M I S S O U R I
Des Moines
Omaha
Sioux City
Mason City
Cedar Falls
Waterloo
Iowa City
Davenport
Dubuque
Ames
Lincoln
Madison
Peoria
Springfield

Figure 1. State of Iowa map and locations of nine tested bridges

BRIDGE DECK DETRIORATION AND CORRESPONDING NDE TECHNIQUES

There are many causes of deterioration in concrete caused by chemical (alkali-silica reaction, carbonation, corrosion, crystallization, leaching, salt and acid action), physical (creep, fatigue, overloading, shrinkage, etc.), and even biological mechanisms (accumulation of organic matter, living organisms). Some mechanisms primarily affect the reinforcement and some of the concrete itself, but all degradation mechanisms lead to a less-resistant structure and, thus, promote other deterioration mechanisms. Understanding these processes is essential in identifying the best techniques for detecting and characterizing bridge deck deterioration. It is also important because different deterioration processes lead to different types of structural defects (delamination, spalling, cracking, rebar size reduction) or material alterations (reduced modulus, changed electrical and chemical properties, etc.). For example, the most common type of deterioration is steel corrosion that leads to concrete deterioration, delamination, contamination, and loss of rebar material. If the corrosion involves large areas, it will cause large cracking and delaminations, and ultimately, spalling of concrete, as illustrated in Figures 2 and 3. On the other hand, overloading will lead to cracking, delamination, and other discontinuities. Those mechanisms will ultimately have different effects on the ability of NDT techniques to detect and quantify them.

For any bridge owner, it will be of interest to be able to assess the condition of a bridge deck at all stages of deterioration. This is illustrated in Figure 3 by a corrosion-induced delamination and spalling. Based on a to-date literature review and the experience of the research team, NDT technologies and procedures used in identification of different stages are illustrated in the figure. The following sections describe the background and implementation of IE, GPR, UPE, USW, HC, and ER in evaluation of decks of the nine bridges.

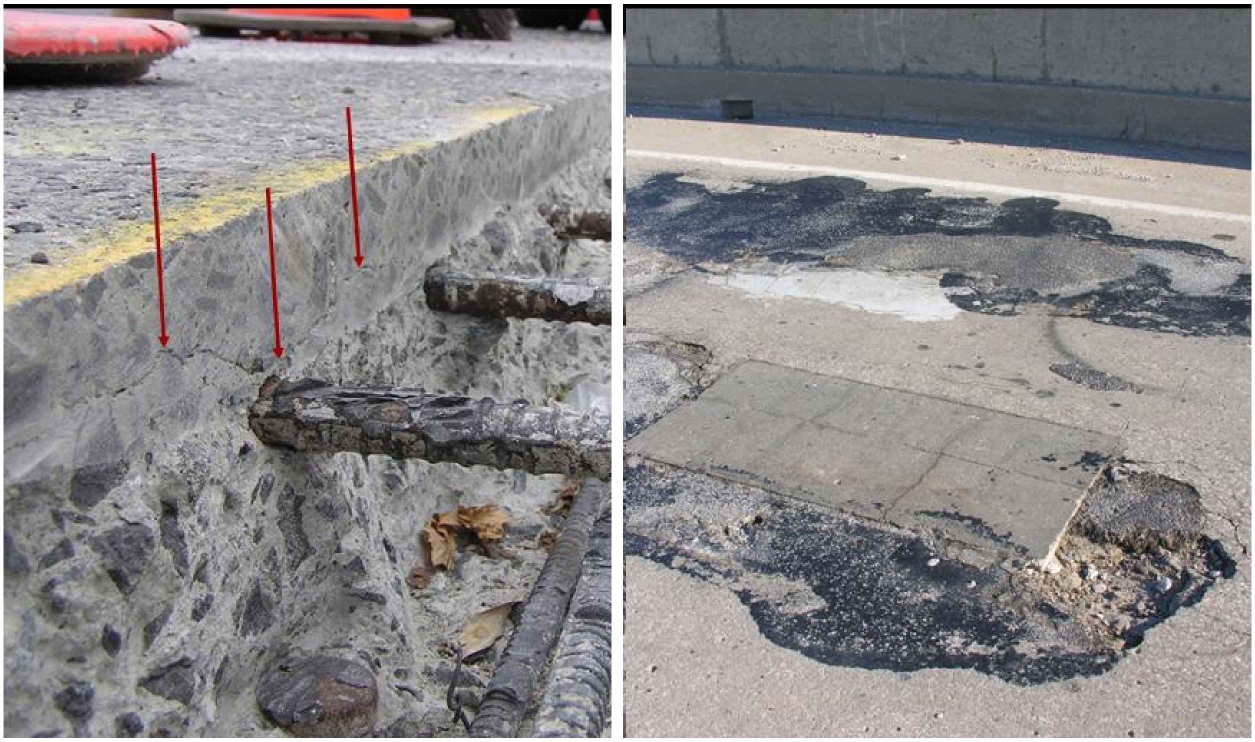


Figure 2. Steel corrosion and delamination (left) and spalling (right)

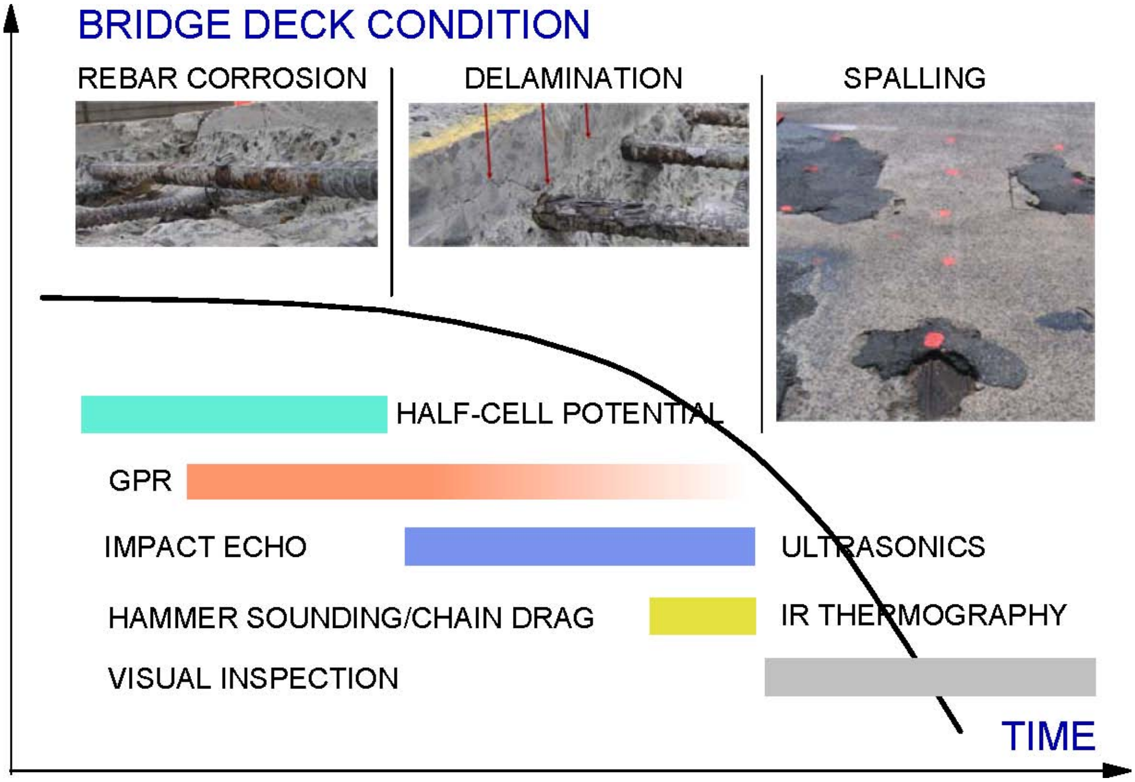

BRIDGE DECK CONDITION
REBAR CORROSION
DELAMINATION
SPALLING
HALF-CELL POTENTIAL
GPR
IMPACT ECHO
ULTRASONICS
HAMMER SOUNDING/CHAIN DRAG
IR THERMOGRAPHY
VISUAL INSPECTION
TIME

Figure 3. Corrosion-induced bridge deck deterioration vs. NDE technologies

Impact Echo

Impact echo has been successfully implemented in the evaluation of bridge decks (Sansalone 1993 and 1997; Gucunski 2000 and 2008; Algernon and Wiggenhauser 2006), it can detect and assess delaminations at various deterioration stages. While IE evaluation is often conducted using an assumption of the concrete compression wave velocity, some integrated seismic/ultrasonic devices provide complementary wave velocity information. That evaluation can be equally described as material quality evaluation, using techniques that draw their principles of velocity (modulus) measurement from pavement analysis (Nazarian and Yuan 1997; Rojas et al. 1999). Application of USW and IE techniques in the assessment of a bridge deck is illustrated in Figure 4. In the first part of the evaluation, the USW test (described later), is conducted using an impact source and two receivers (see Figure 5) to measure the velocity of propagation of compression (P) waves. In the second part of the evaluation, the IE test is conducted using an impact source and a single nearby receiver. Because of a significant contrast in rigidity of concrete and air, the elastic wave is practically entirely reflected off the bottom of the deck back to the deck surface. The frequency of the reflection, called return frequency, can be identified in the response spectrum of the recorded signal. Finally, the depth of the reflector (in this case the deck thickness) can be obtained from the return frequency and the previously determined P-wave velocity. A comprehensive summary of IE testing was provided by Schubert and Koehler (2008).

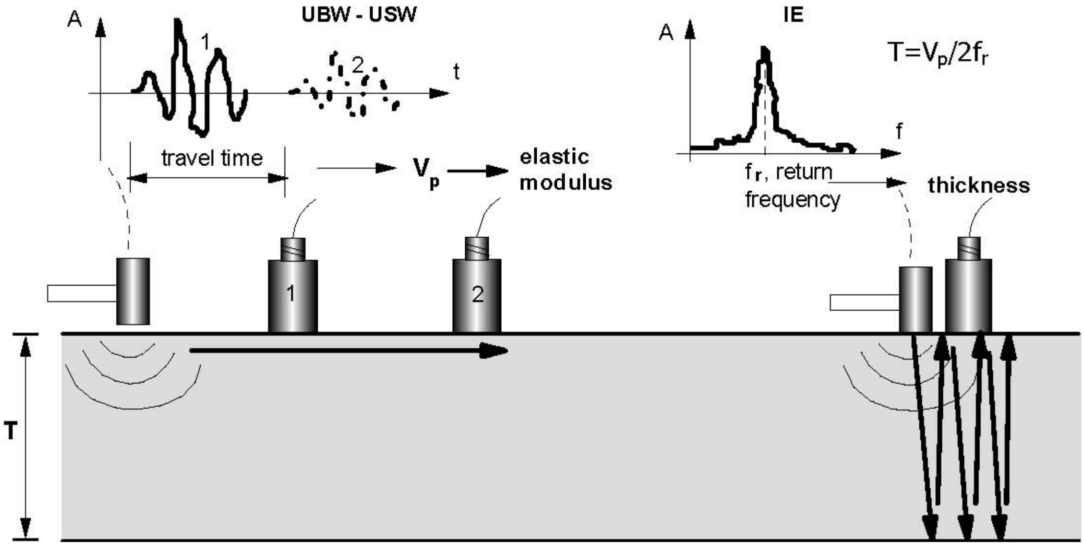


Figure 4. Measurement of bridge deck elastic modulus and thickness by USW and IE methods

While the primary objective of IE testing is to determine dominant reflectors, according to the approximate relationship described in Figure 4, a unique thickness or depth of the reflector can be correlated to every component of the spectrum according to the same relationship. In the case of a delaminated deck, reflections of the P-wave occur at shallower depths, causing a shift in the response spectrum towards higher frequencies. Depending on the extent and continuity of the delamination, the partitioning of energy of elastic waves may vary and different grades can be assigned to that particular section of a deck as a part of the condition assessment process. This is illustrated in Figure 5. In the case of a sound deck (good condition), a distinctive peak in the response spectrum corresponding to the full depth of the deck can be observed. Initial delamination (fair condition) is described as occasional separations between the two deck zones. It can be identified through the presence of two distinct peaks, indicating energy partitioning from two dominant wave propagation patterns. The first peak corresponds to reflections from the bottom of the deck, while the second corresponds to reflections from the delamination. Progressed delamination (poor condition) is characterized by a single peak at a frequency corresponding to a reflector depth that is shallower than the deck thickness, indicating that little or no energy is being propagated towards the bottom of the deck. Finally, in a very severe case of a wide delamination (serious condition), the dominant response of the deck to an impact is characterized by a low-frequency response of flexural mode oscillations of the upper delaminated portion of the deck. This response is almost always in the audible frequency range, unlike response of the deck in the fair and poor conditions that may be in the ultrasonic range. Because it is significantly lower than the return frequency for the deck bottom, it produces an apparent reflector depth that is larger than the deck thickness.

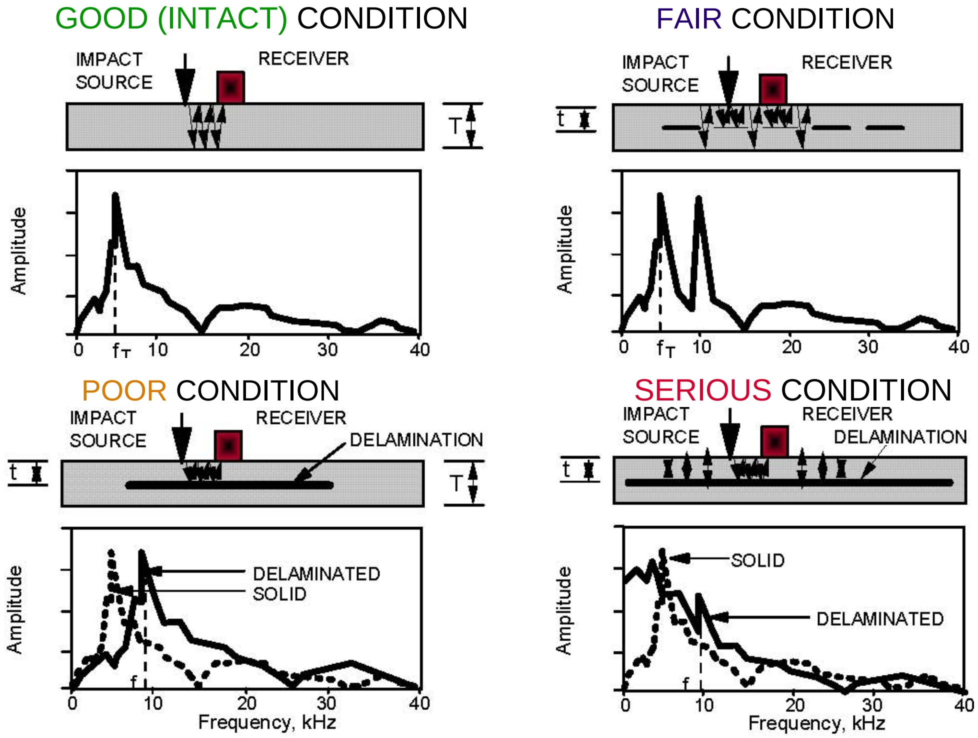


Figure 5. Condition grades for different IE spectra

IE is commonly implemented in deck evaluations by conducting point testing on a grid of a selected spacing. The testing is conducted using impact echo devices, which in some case integrate other ultrasonic seismic methods. One of such devices is the portable seismic property analyzer (PSPA), shown in Figure 6, that has a sole purpose of evaluation of surface pavement layers and bridge decks. The device integrates the previously described ultrasonic techniques (USW and IE). Bridge deck evaluation is typically done on grids of 0.6 x 0.6 m to 0.9 x 0.9 m (Figure 6). Impact echo testing is simple and typically takes less than 30 seconds per test point. On an average, about 50 m^2 of a deck can be tested per hour using a 0.9 m spacing, or about 20 m^2 using a 0.6 m spacing. The testing is relatively insensitive to traffic induced vibrations because those are in a much lower frequency range than the IE test range. Another device that was used in this project is the Stepper, a device for an automated data collection developed at BAM (German Federal Institute for Material Research and Testing) and shown in Figure 7. The Stepper allows continuous data collection at a prescribed spacing between data points at a speed of about 10–15 points per minute for closely spaced points (few inches apart). In addition, the Stepper can carry another or multiple probes and collect data simultaneously; for example, impact echo, ultrasonic echo, or GPR antenna. The Stepper shown in Figure 7 is equipped with impact echo and ultrasonic probes.

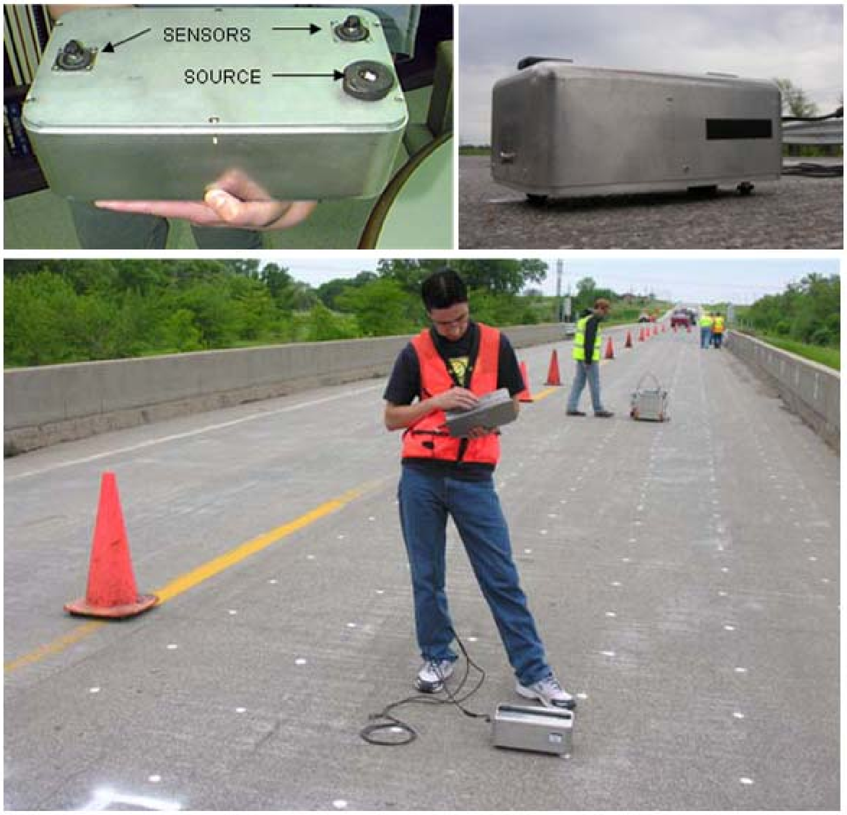


Figure 6. IE and USW sensor box (top) and field implementation (bottom)

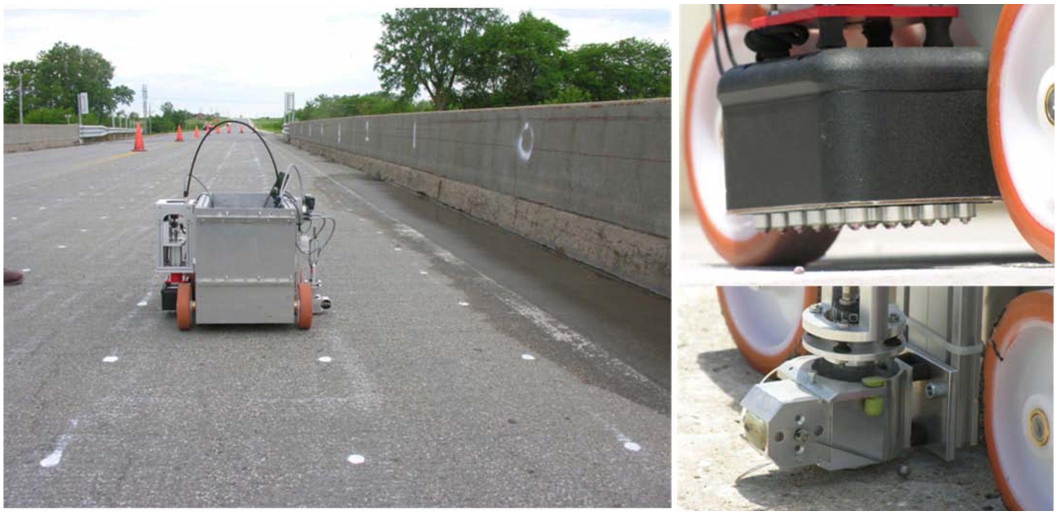


Figure 7. Stepper (left) with an ultrasonic probe (top right) and IE probe (bottom right)

Impact echo/seismic testing results are commonly described in terms of concrete modulus distributions, as illustrated later in the discussion on the use of USW and condition assessment distributions (with respect to the degree of delamination). This is illustrated by a conditions map for a deck of the bridge deck O1 in Figure 8. The condition map is plotted in terms of the four condition grades. The areas marked for repair based on the chain drag survey are superimposed on the graph. The map clearly shows a good agreement of the two approaches but also some limitations of the chain drag in identifying areas that are in the initial to progressed delamination stage (fair and poor).

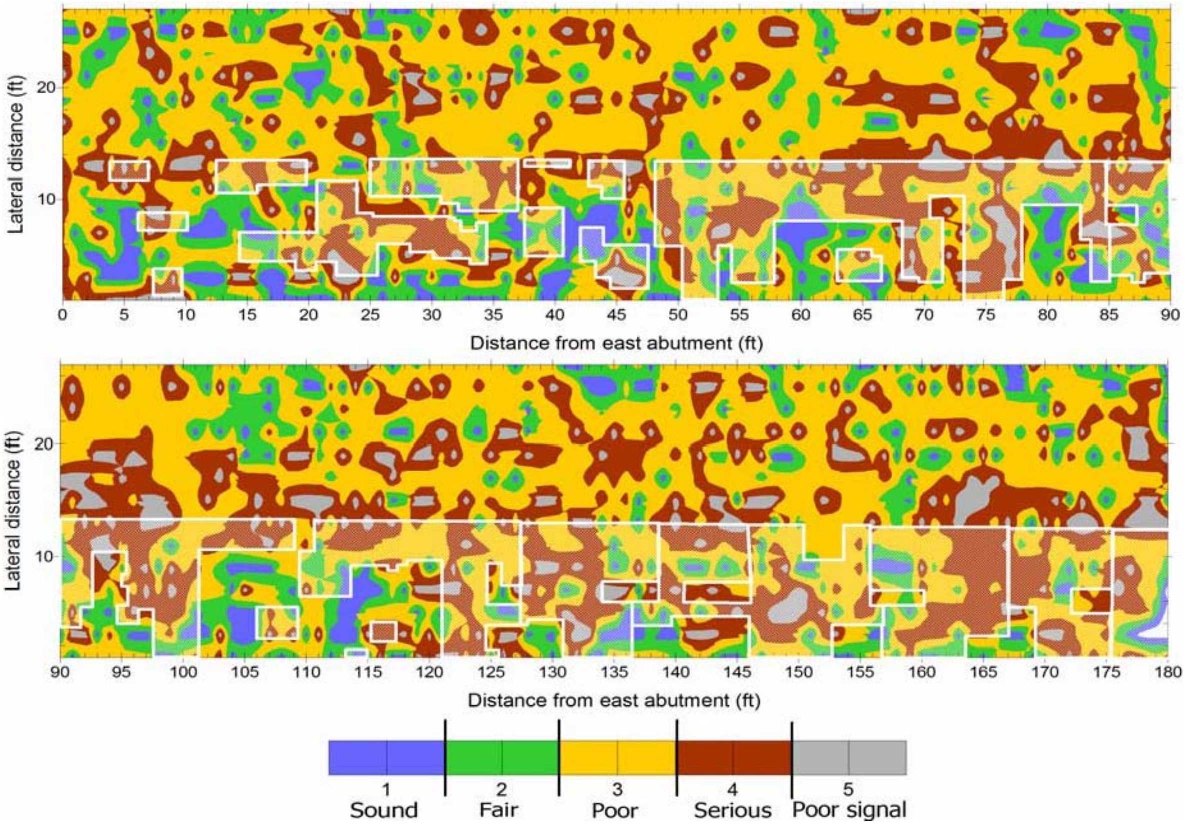


Figure 8. IE condition assessment compared to chain drag (white lines) for bridge deck O1

Ground Penetrating Radar (GPR)

Hundreds of bridge decks have been evaluated using a variety of GPR systems and deployment configurations (Romero et al. 2000; Barnes and Trottier 2000; Maser and Rawson 1992). Typical GPR applications included evaluations of deck thickness, concrete cover and rebar configuration, potential for delamination, concrete deterioration, and estimation of concrete properties. The GPR condition assessment is based on measurement of signal attenuation on the top rebar level, which provides a rational approach in characterization of the severity of deterioration of concrete and potential for bridge deck delamination. Electrical conductivity, as well as material dielectric properties, play primary roles in how a GPR signal will travel through a material. Most directly, electrical conductivity (inverse of resistivity) affects how well GPR signals can penetrate through a material. Metals cannot be penetrated (even dense wire screens or thin foils are impermeable to GPR); most construction materials (concrete, asphalt, or engineered pavement soils) are fair to good host materials for GPR. Similarly, concrete that is moist and high in free chloride ions (or other conductive materials), such as a reinforced deck that has undergone deterioration due to corrosion of the rebar, can also significantly affect a GPR signal. In most cases, ground-truth validation of radar results is restricted to point specific areas. Horn (air-coupled) antennas (see Figure 9) have been used in the past to primarily provide a fast overview of the condition of the deck, while ground-coupled antennas provide more detailed imaging and analysis of the deck condition. The attenuation of the GPR signal at the top rebar level is normally used to represent the condition of the deck. When the antenna is centered directly above the rebar, the highest amplitude return for the area near the rebar is obtained. This amplitude will be highest when the deck is in a good condition and weak when delamination and corrosion are present. Amplitudes for all points are normalized with respect to the best possible condition to obtain the plot of attenuation. Once the final interpretation is completed, a unique deterioration threshold is established using ground truth, such as cores or NDE methods like impact echo (Barnes and Trottier 2000; Gucunski et al. 2005).

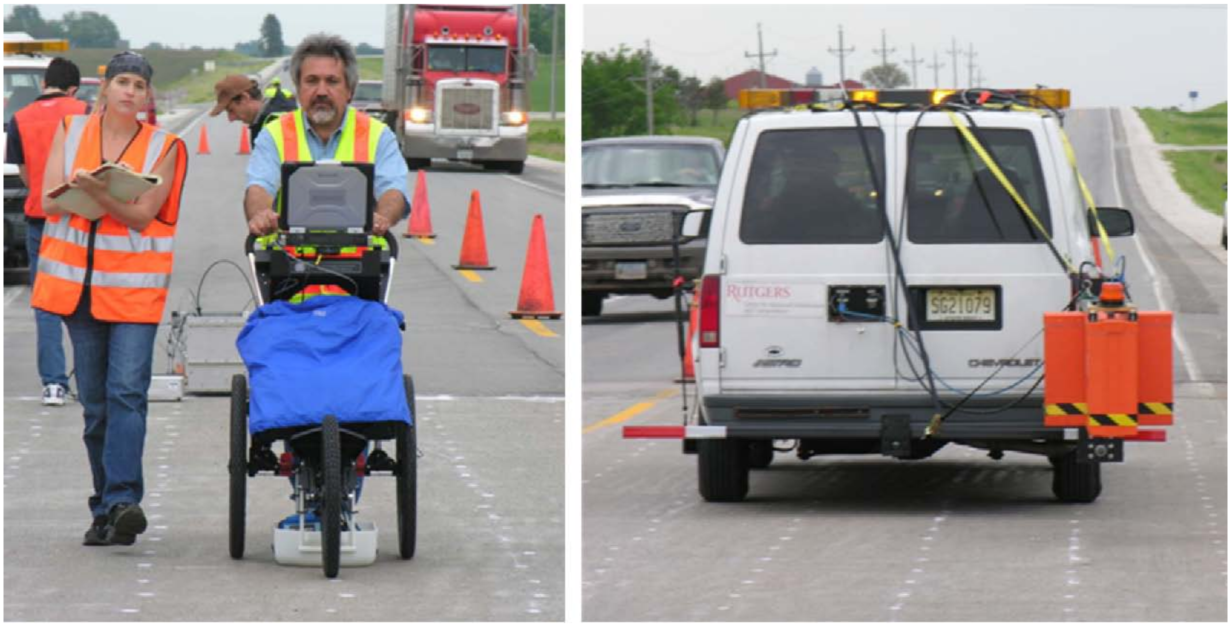
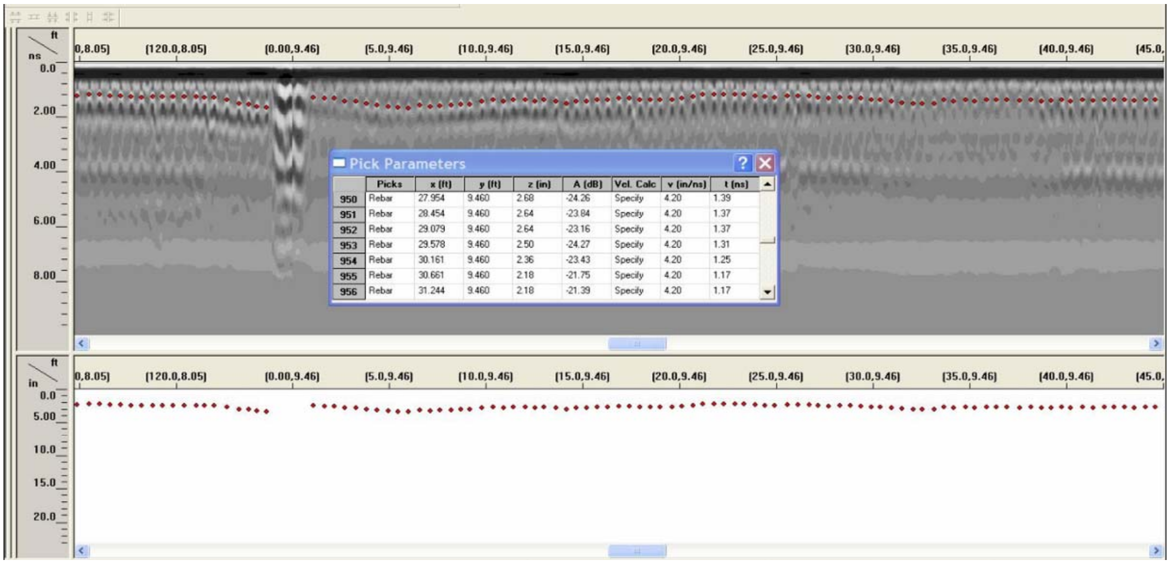


Figure 9. Data collection: a 2.6 GHz ground-coupled antenna (left) 1 GHz dual polarization air-coupled antennae (right)

There are three main processing steps involved in creating a deterioration map of a deck (Parrillo et al. 2006). The first processing step, most commonly implemented in a semi-automated manner, performs time-zero correction, migration, and rebar reflection mapping. The second step is an interactive interpretation where the rebar locations may be reviewed and edited. The result of the process is a table of rebar position (depth) and amplitude of reflection, as shown in Figure 10 for rebar picking in the scan from the 1.5 GHz antenna. Zones of weak reflections or strong attenuation are clearly visible in the image. Since the air-coupled (horn) antenna is suspended approximately 50 cm (20 in.) above the ground surface, the energy transmitted from antenna is less focused as it reaches the surface of the deck and travels to the rebar. Consequently, rebars from the horn antenna appear in the data as a layer rather than the individual hyperbolas.

The last step is the presentation of the attenuation of the signal at the rebar level and, thus, relative deterioration of the deck, in the form of a contour map. Color-coded contour plots are generated using the normalized or corrected amplitude of the reflection at the rebar level as the gradient in the plot. Color levels assigned to the amplitude represent the level of attenuation and, qualitatively, the severity of deterioration. The hot colors (reds) represent the severest levels of deterioration and the cool colors (blues and greens) low levels of deterioration or a good condition. The contour plots of amplitude attenuation at the rebar level for a 1.5 GHz ground-coupled antenna before and after correction for rebar depth are shown in Figure 11.


ft
ns
0.0
2.00
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0,8.05)
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(0.00,9.46)
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(10.0,9.46)
(15.0,9.46)
(20.0,9.46)
(25.0,9.46)
(30.0,9.46)
(35.0,9.46)
(40.0,9.46)
(45.0,
Pick Parameters
in
0.0
5.00
10.0
15.0
20.0

	Picks	x (ft)	y (ft)	z (in)	A (dB)	Vel. Calc	v (in/ns)	t (ns)
950	Rebar	27.954	9.460	2.68	-24.26	Specify	4.20	1.39
951	Rebar	28.454	9.460	2.64	-23.84	Specify	4.20	1.37
952	Rebar	29.079	9.460	2.64	-23.16	Specify	4.20	1.37
953	Rebar	29.578	9.460	2.50	-24.27	Specify	4.20	1.31
954	Rebar	30.161	9.460	2.36	-23.43	Specify	4.20	1.25
955	Rebar	30.661	9.460	2.18	-21.75	Specify	4.20	1.17
956	Rebar	31.244	9.460	2.18	-21.39	Specify	4.20	1.17

Figure 10. Processed data and rebar picking for 1.5 GHz antenna at 24 scans/foot

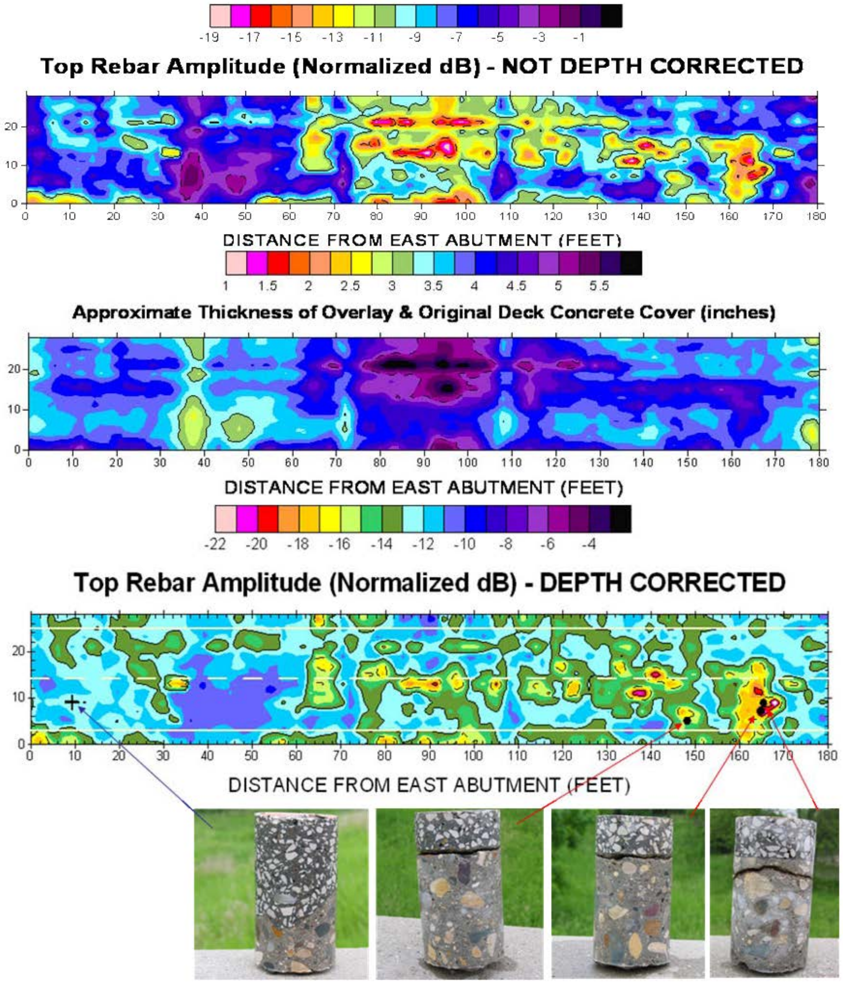

Figure 11. Normalized top rebar amplitude without (top) and with depth correction (bottom) and approximate combined thickness of overlay and concrete cover for bridge O1

Ultrasonic Surface Waves (USW) Method

The USW test is identical to the spectral analysis of surface waves (SASW) test (Nazarian et al. 1983; Stokoe et al. 1994), except that the frequency range of interest is limited to a narrow high-frequency range where the velocity of the surface wave (phase velocity) does not vary significantly with frequency (Nazarian et al. 1993). The SASW method is based on the phenomenon of dispersion of a surface (Rayleigh) wave in layered systems, i.e., the phenomenon that the velocity of propagation is frequency dependent. The objective of the test is to determine that velocity-frequency relationship, termed the dispersion curve, and afterwards, through the process of inversion or backcalculation, to obtain the shear-wave velocity profile. The elastic modulus profile can then be easily obtained using simple relationships between the velocity of propagation and measured or approximated values for mass density and Poisson's ratio of different layers. Variation in the phase velocity would be an indication of variation of material properties (elastic moduli) with depth.

In a case of a single homogeneous layer material evaluation like a bridge deck, the SASW process becomes significantly simpler. The material modulus can be described as being directly measured instead of backcalculated. A PSPA shown in Figure 12 can be used in evaluation of concrete modulus by USW method. Concrete modulus can be obtained directly in the field, as is illustrated in the figure by the computer screen shot and a bridge deck modulus distribution.

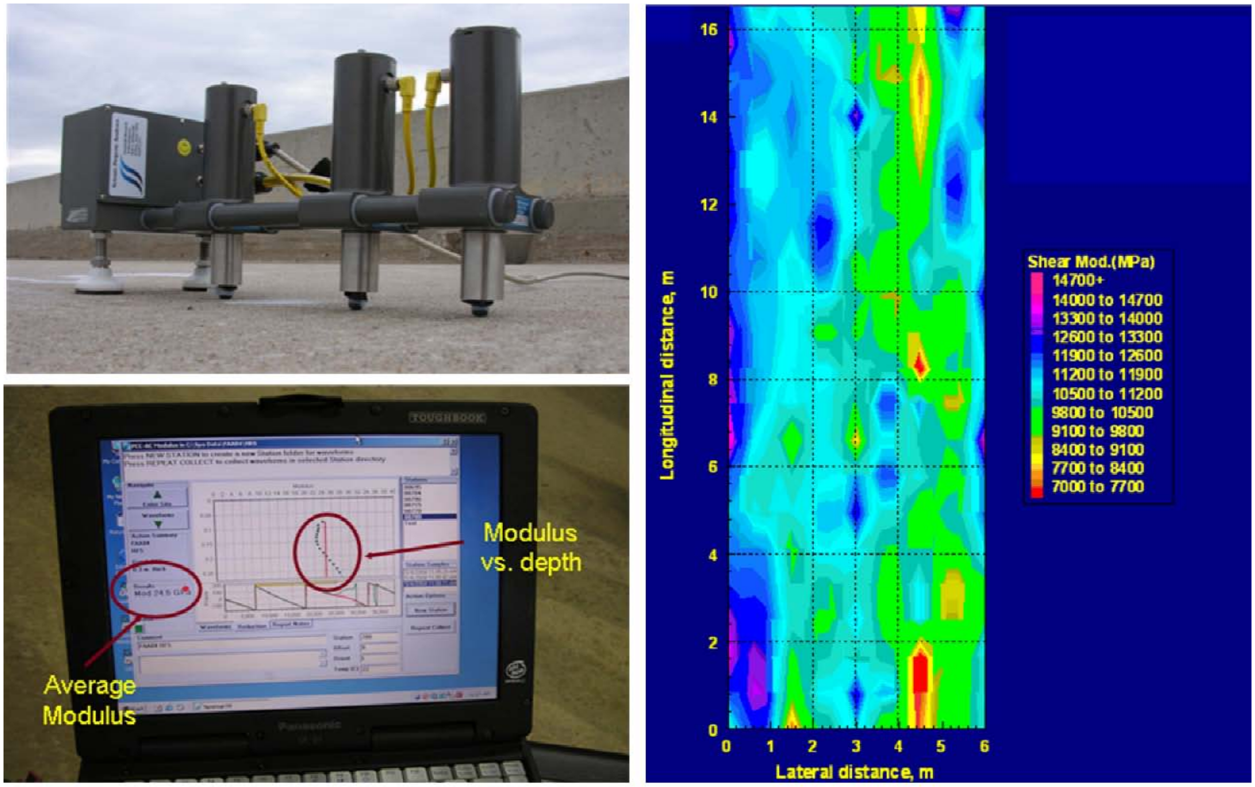

Figure 12. PSPA (top left), concrete modulus evaluation (bottom left), and concrete modulus variation (right)

Half-Cell Corrosion Potential Measurement

The half-cell potential measurement is a simple way to assess the probability of steel corrosion. The corrosion process dissolves Fe^{++} ions from the steel into the concrete pore water. Thus, the potential of the reinforcement gets a negative polarization. Half-cell corrosion potential involves the measurement of the electrical potential between the reinforcement and a reference electrode (usually copper electrode in a copper sulfate solution) coupled to the concrete surface, as shown in Figure 13. By putting the electrode from one point to another or by using a wheel electrode a potential map can be created, as shown in the same figure. Regions with a more negative potential indicate a higher probability of corrosion.

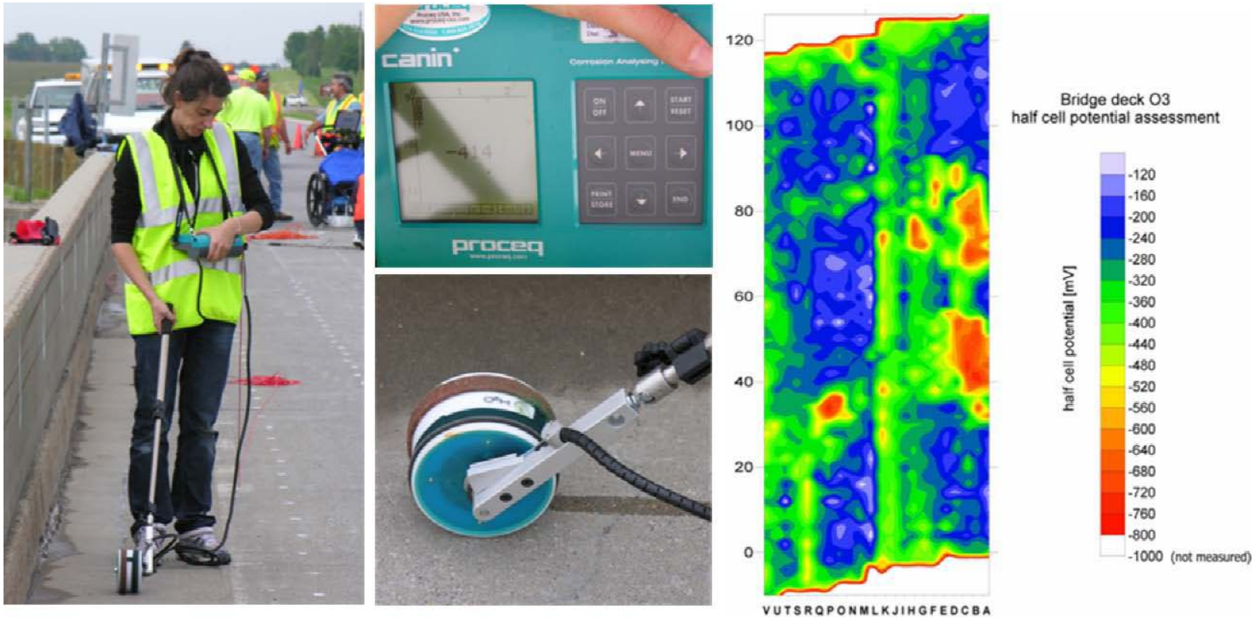

Figure 13. Half-cell corrosion potential measurement (left), display and probe (middle) and potential map

The ASTM C 876 gives general guidelines for evaluating corrosion probability in concrete structures. In general, if the potential reading is higher than -0.2 V, there is 90% probability that there is no corrosion. On the other hand, if it is lower (more negative) than -0.35 V, there is 90% probability of corrosion. It should be mentioned that half-cell potential measurements cannot give quantitative information about the actual corrosion rate of the reinforcement. Furthermore, the measured potential values are not only influenced by corrosion activity, but also by the concrete cover and the concrete resistance. The resistance varies with moisture content, temperature, and ion concentrations. All this means that the potential values given in the guidelines should not be considered as generally valid. A combination of half-cell measurements with resistivity measurements may help interpret the collected data (Elsner 2003). A comparison of half-cell and GPR measurements on the deck O1 is shown in Figure 14. Quite clearly, a good correlation can be established in the zones of the highest deterioration.

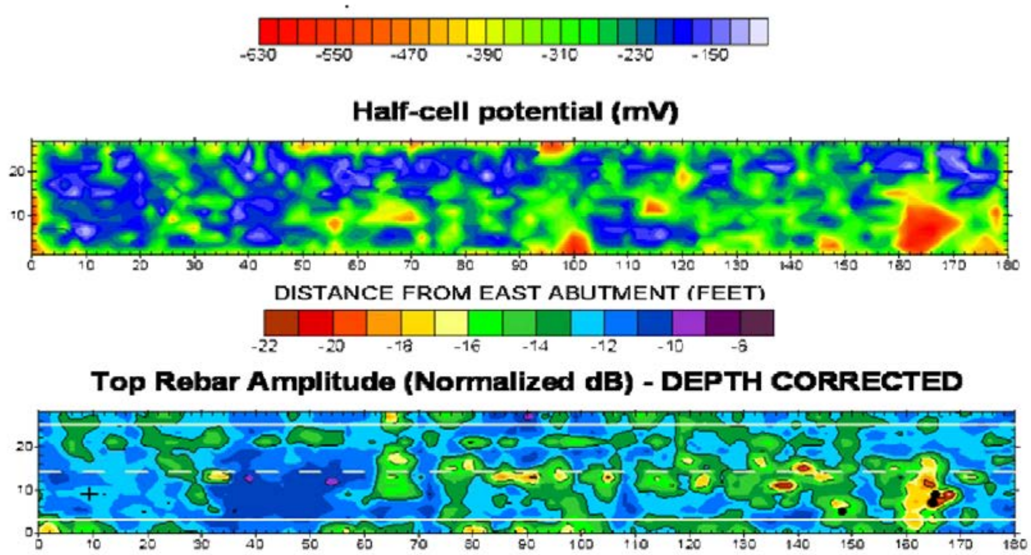

Figure 14. Comparison of half-cell potential and GPR attenuation maps

Electrical Resistivity—Wenner probe

Electrical resistivity of concrete is of particular interest in consideration of the corrosion potential of reinforcement steel. The presence and amount of water in concrete are important parameters in assessing its corrosion state. Electrical conduction in concrete systems occurs mainly due to electrolytic current flow through the open pore system and the formation of electrochemical corrosion cells. In damaged and cracked areas, the increased porosity leads to preferential paths for fluid and ion flow. The higher the electrical resistivity of the concrete, the lower is the corrosion current passing between anodic and cathodic areas of the reinforcement steel. It has been observed that a resistivity of less than 5,000 ohm*cm can support very rapid corrosion of steel (Brown 1980). In contrast, the concrete poses a high resistance to the passage of current if it is dry and unable to support ionic flow. Consequently, corrosion will only occur at a very low rate, if at all. Various researchers have reported that corrosion can be limited by increasing the concrete resistivity (e.g., Tremper 1958; Vassie 1980; Alonso et al. 1988). Whiting and Nagi (2003) have summarized the relationship between electrical resistivity and corrosion rates for steel in concrete. These values, however, should rather be understood as guidelines than exact thresholds. Usually, it is also recommended to carry out resistivity surveys in combination with other corrosion techniques like half-cell potential mapping (Millard 1991; Millard and Gowers 1999). Probably the most common electrode layout in civil engineering applications is the Wenner setup (see Figure 15). It uses four probes that are equally spaced. A current is applied between the outer electrodes and the potential measured across the two inner ones.

When electrical resistivity measurement are made on a concrete bridge deck, recordings should be taken in approximately a 0.6 x 0.6 m grid to ensure adequate data quality. Depending on the properties of the probe, in most cases the concrete surface has to be pre-wetted, for example, with water from a spray bottle. This becomes particularly significant when the electrical contact is established by wet foam pads like shown in Figure 15 (left); in this case, the concrete surface must not be covered with any electrically insulating coating. Results of a resistivity survey of a deck are shown in Figure 16. Red and orange colors indicate low resistivities and, hence, high corrosion rates and a serious condition of the deck. Yellow and green colors mark intermediate states and blue marks sound areas. In this case, the water probably penetrated into the bridge deck at the building joint (dashed line) between the two traffic lanes and the abutments.

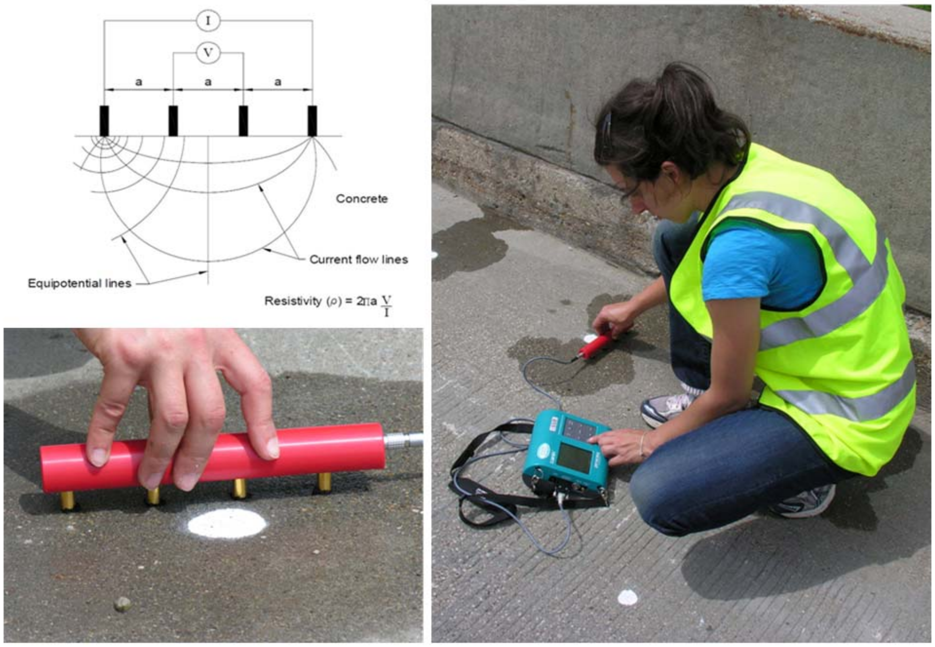

Figure 15. Four point Wenner spread applied in (geo-) electrical testing surveys (top right, from Whiting and Nagi 2003), and Wenner probe testing on a bridge deck (bottom left and right)

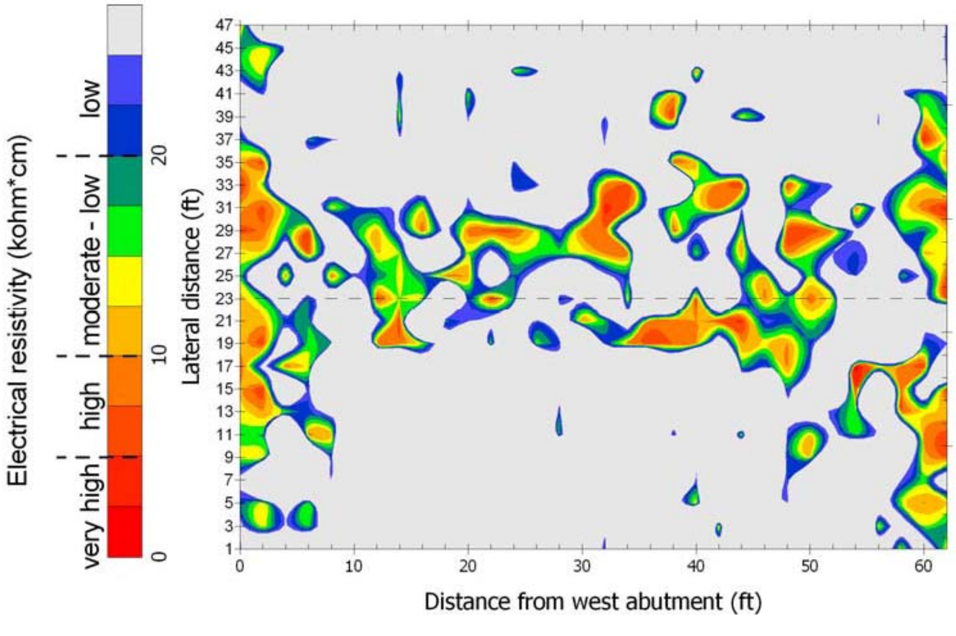

Figure 16. Results of electrical resistivity measurement on a bridge deck

CONCLUSIONS

Condition assessment of bridge decks using NDE technologies has shown its ability to detect and characterize different types and levels of deterioration. For example, half-cell potential and electrical resistivity measurements enable assessment of likelihood and severity of corrosion. On the other hand, GPR provides information about deterioration/alteration of concrete. Finally, impact echo and ultrasonic measurements enable detection and characterization of delamination in the deck. Delamination detection was validated through the comparison with cores taken and selected locations. A comparison between impact echo and chain drag delamination detection confirms the ability of the chain drag to detect progressed delamination but also its limitation in detection of initial delaminations.

ACKNOWLEDGMENTS

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Freight Bottlenecks in the Upper Midwest: Identification, Collaboration, and Alleviation

Qi Gong
Department of Civil Engineering
University of Wisconsin, Madison
1241 Engineering Hall
1415 Engineering Dr
Madison, WI 53706
qgong2@wisc.edu

Jessica Guo
Department of Civil Engineering
University of Wisconsin, Madison
1206 Engineering Hall
1415 Engineering Dr
Madison, WI 53706
jyguo@wisc.edu

Andrew Obernesser
Department of Urban and Regional Planning
University of Wisconsin, Madison
2205 Engineering Hall
1415 Engineering Dr
Madison, WI 53706
obernesser@wisc.edu

ABSTRACT

Freight movements throughout the Mississippi Valley region are complicated by the presence of recurring congestion along the highway, rail, and port freight networks. This congestion has several deleterious effects on freight carriers, commuters, governments, and markets. State departments of transportation (DOTs) in the Mississippi Valley Freight Coalition are looking to identify the points of concern along their networks and make plans to improve congested conditions.

In some areas, congestion is a result of physical conditions in the network that constrict efficient freight mobility. These conditions are defined here as various types of "freight bottlenecks." In an effort to identify the locations of these bottlenecks and their effect on freight flows, this research considers flow throughout the networks designated as freight routes, determines locations wherein physical or operational constraints exist, and calculates the truck delay associated with the existing conditions.

The highway bottleneck locations calculated through geographic information system (GIS) analysis are verified through communication with state transportation engineers and planners in the Mississippi Valley region. Through these discussions, alternative methodologies are considered and site-specific conditions are further detailed by those interviewed. The bottleneck locations found in this study are also cross-checked against state projects planned for alleviating known bottleneck conditions.

This presentation will discuss the methodologies developed to identify freight bottleneck locations, the process of collaborating with state DOTs to verify these locations, and the inventory of alleviation projects that are already planned or recommended by the research team to address these highway freight bottlenecks.

Key words: freight transportation—Mississippi Valley Freight Coalition—traffic bottlenecks

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INTRODUCTION

The freight that passes through the Mississippi Valley Region is high volume and has a substantial impact on the economy of the region. According to the Bureau of Transportation Statistics-sponsored Commodity Flow Survey, trucks carried almost 2.5 billion tons of freight across the highways of the 10 states of the Mississippi Valley region in 2002. Efficient movement of freight through this region is critical to the economic competitiveness of the nation. However, previous studies indicate that the existence of recurring highway bottlenecks in this region has been jeopardizing the reliability and efficiency of freight movement significantly, which calls for an immediate remedy.

In an effort to alleviate the impact of traffic bottleneck to freight movement through the Upper Midwest, this study performs a comprehensive analysis to identify, characterize, and prioritize the regional truck bottlenecks. Identification and characterization of freight bottlenecks in the Upper Midwest will enable member states of the Mississippi Valley Freight Coalition to take appropriate action to relieve existing bottlenecks. Through prioritization of the bottlenecks identified in the study, coalition members can plan to address freight bottleneck solutions in a way that will be optimal to the region’s economic well-being.

STATE OF KNOWLEDGE

The significant increases in travel time, extra cost, and environmental pollution caused by bottlenecks have warranted several studies on bottleneck identification, prioritization, and alleviation. The primary focus of these studies is on the temporal and spatial distribution of bottlenecks on the traffic network. There are generally three approaches to bottleneck analysis that differ by the types of data used for analysis.

The first approach is based on rich traffic data obtained from loop detectors along major corridors, including count of vehicles, occupancy of detector, and speed. One way of using such data is to construct the curves of cumulative vehicle counts and occupancy of detectors at each site and to investigate the bottleneck formation by comparing the curves visually (Cassidy and Bertini 1999; Bertini and Myton 2004). Based on the speed contour map, an alternative method identifies the bottleneck condition through the direct inspection of measured speed (Chen et al. 2004; Ban et al. 2007). Usually, a threshold value is applied to distinguish free-flow condition and bottleneck condition, which is derived from empirical knowledge or selected to best match the identification results and ground truth. As loop detector data are typically collected for specific sites, this approach is usually used for bottleneck analysis at the local scale.

The second bottleneck analysis approach is based on truck global positioning system (GPS) data. For example, using GPS data points obtained from 25 portable GPS devices provided to trucking companies by the Washington State Department of Transportation, McCormack and Hallenbeck (2006) developed a series of benchmarks to examine the roadway segment performance, including speed, mean speed, and speed of various percentiles. Particularly, the data points from four of the GPS devices placed on Boeing trucks traveling in a routine route were used to identify the locations where delays happened as indicated by a slower speed. More recently, a freight performance study sponsored by the American Transportation Research Institute (Short et al. 2009) examines the truck delay on the worst 30 U.S. freight bottlenecks identified from a Federal Highway Administration study. The delay is calculated based on the difference between free-flow speed and average speed measured from GPS data. An assumed study length is employed when calculating the interchange bottleneck delay, which ranges from 2 to 3 miles extending from interchange location.

The third bottleneck analysis approach utilizes the Highway Performance Monitoring System (HPMS) data. This has been perhaps the most widely adopted method for identifying bottlenecks due to the fact that HPMS is available for all states and provides a consistent source of traffic-related information throughout the national highway network. The HPMS datasets consist of two databases: the Universe database, which provides basic physical and traffic information on all sections of all major roads, and the Sample database, which is a statistically selected sample of Universe sections with detailed geometry and operation information reported and a limited number of roadway sections. As one of the earliest studies to identify bottlenecks on a national basis, Cambridge Systematics, Inc. (1999) scans the HPMS database for freeway segments with high ratios of traffic volume to available highway capacity, by which a preliminary list of candidate bottlenecks are developed. The Cambridge Systematics, Inc. (2005) made an initial effort to investigate the national freight bottlenecks by using the HPMS data and Freight Analysis Framework (FAF) data. In its report "An Initial Assessment of Freight Bottlenecks on Highways: White Paper" (hereafter referred to as White Paper), not only are the locations of bottleneck identified, but also a comprehensive typology is developed in this study to characterize freight bottlenecks. The report presents several lists of freight bottlenecks ranked by annual hours of delay for trucks for each type of bottleneck.

STUDY METHODOLOGY

In the present study, the authors are building on the HPMS-based analysis approach and develop a systematic framework to identify, characterize, and prioritize truck bottlenecks in the Mississippi Valley region. The authors favored the HPMS-based approach over other approaches due to data availability. HPMS data are found to be the only consistently and publicly available data source for the regional-level analysis that covers 10 states.

Figure 1 provides an overview of the proposed bottleneck analysis methodology. Due to the common data source used, the method follows the same bottleneck typology defined in the White Paper (2005) and considers four types of truck bottleneck conditions: interchange, lane drop, steep grade, and signalized intersection bottlenecks. However, as opposed to assigning each bottleneck location to exactly one of the four types of bottlenecks, the authors recognize the limitation of the HPMS data and consider the possibility of a highway section being associated with more than one bottleneck condition.

The method also differs from that used in the White Paper (2005) in how highway sections are scanned and identified as potential bottleneck locations. In the White Paper, sections with a high ratio of volume to capacity during peak hour are selected as candidate bottleneck locations. In the analysis, the authors use truck unit delay (total hours of delay for trucks per mile) instead. Truck unit delay is considered as a more suitable measure for the study because it captures the delay for all commercial motor drivers using per mile of a given highway segment. It was considered to more directly capture the congestion impact to freight movement.

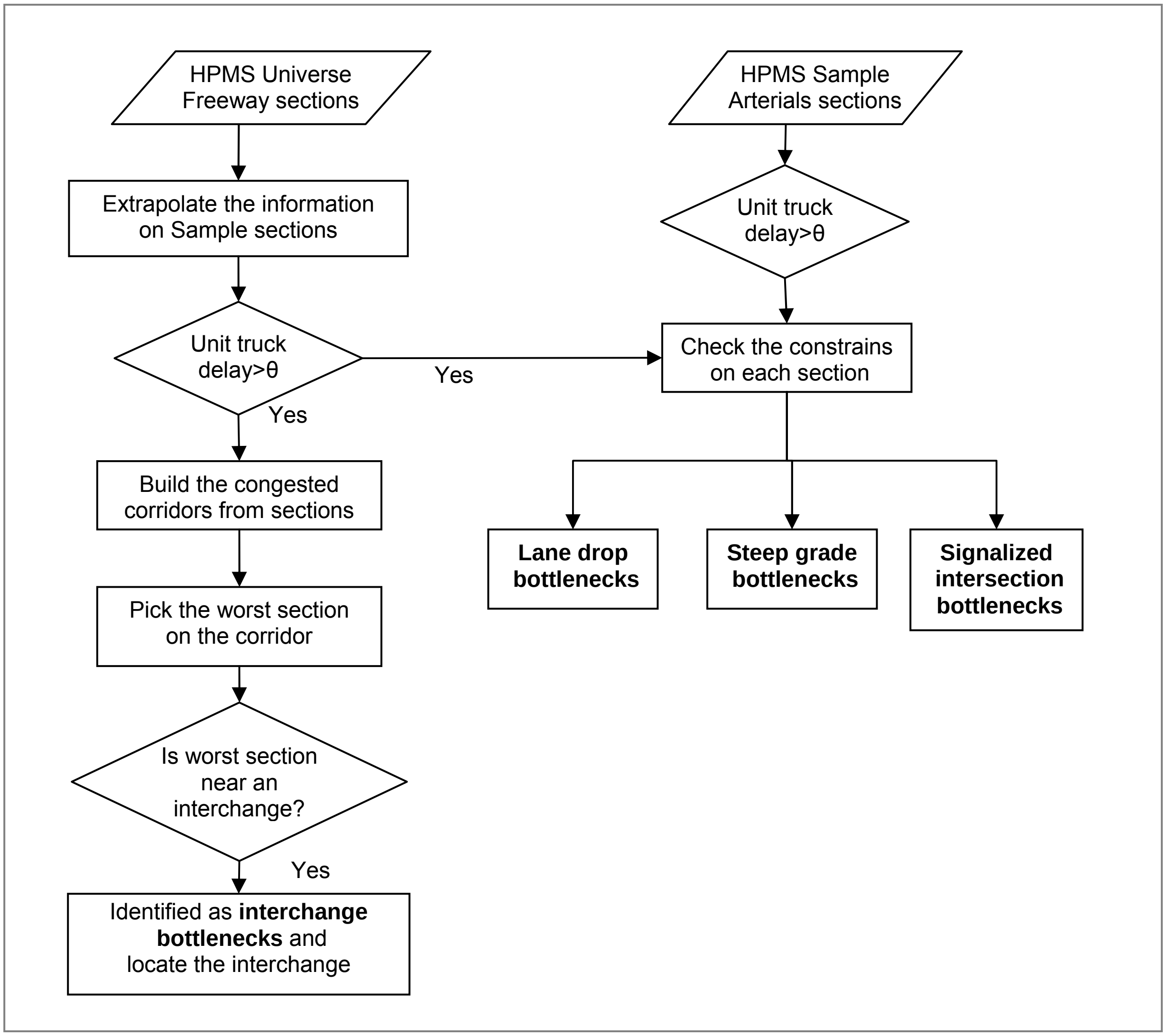
HPMS Universe
Freeway sections
Extrapolate the information
on Sample sections
Unit truck
delay>θ
Yes
Build the congested
corridors from sections
Pick the worst section
on the corridor
Is worst section
near an
interchange?
Yes
Identified as **interchange bottlenecks** and
locate the interchange
Yes
HPMS Sample
Arterials sections
Unit truck
delay>θ
Check the constrains
on each section
Lane drop
bottlenecks
Steep grade
bottlenecks
Signalized
intersection
bottlenecks

Figure 1. Overview of the proposed HPMS-based methodology for identifying and characterizing truck bottlenecks

Because the truck unit delay is determined by the severity of bottleneck and the presence of truck traffic volume at a bottleneck location, three conditions lead to a significant truck unit delay:

- The presence of exceptionally high truck volume
- The presence of exceptionally high hours of delay per vehicle mile
- The combination of the previous two conditions

The last condition could be referred to as a general bottleneck as both passenger cars and heavy trucks passing the location would be stuck in traffic queue and experience increase in travel time and decrease in speed. Many of the state DOTs and metropolitan planning organizations have performed extensive highway congestion studies and identified such bottlenecks. However, the first condition, which describes the case where slight traffic congestion happens, but a high volume of trucks accumulates the unit truck delay, is usually overlooked in general bottleneck study. By taking truck volume into the consideration, this study contributes to develop a compressive inventory of freight bottlenecks in the region.

Our delay estimation is based on equations borrowed from a previous study by Margiotta et al. (1999), who developed a series of equations to estimate hours of delay and speed on each section by using a simplified queuing-based model, QSIM. This model incorporates several advanced features, including the use of queuing analysis, accounting for temporal distribution and daily variation of traffic flow, and so on. On the other hand, there are limitations by using this method to estimate hours of delay for trucks, one of which is the potential overestimate of exposure of truck trips to delay. By multiplying the truck volume with the unit delay estimated from the equations, it is assumed that truck trips follow the similar temporal distribution as passenger car trips. However, most commercial motor carriers make great efforts developing strategies to reschedule and/or reroute picking up and delivering works in order to avoid known recurring bottleneck. This might lead to the underlying difference in temporal distribution patterns between truck trips and passenger car trips, suggesting an overestimate of truck delay.

Because interchange bottlenecks usually cause system congestion, simply examining the sections with significant truck unit delay on freeways would identify many spatially closed bottlenecks, which are actually located on a congested corridor as a result of one interchange bottleneck. In order to account for the system impact of interchange bottlenecks, we also develop a corridor congestion growing method to identify the bottleneck locations on corridors. Starting from the sections with high truck unit delay, the neighboring sections are examined to determine if similar congestion patterns exist. By assuming the continuously congested traffic is caused by one bottleneck, the sections immediately adjacent to each other having similar unit delay are connected to build one congestion corridor. The location where vehicles experience the severest delay on a corridor is selected as the bottleneck location. To qualify the bottleneck as an interchange bottleneck, a further inspection is performed to search for the closest interchanges from either end of the section along the congested corridor, with a maximum searching length threshold.

Realizing the fact that the interchange configuration varies significantly from case to case and the most congested location caused by an interchange might be out of its physical scope, the maximum searching length is determined through a sensitivity analysis. The sensitivity analysis is designed to study how total interchange delay grows by including longer extent of highways from interchange location. The vehicle hours of delay is calculated for each leg connecting to an interchange, and the total interchange delay is obtained by summing up vehicle delays on every leg together. The length included in calculation for each leg increases from 0.5 mile to 3 miles with a 0.5 mile increment. By visually inspecting the change of additional total delay and additional vehicles miles traveled (VMT) caused by including longer extent for all interchanges in the region, a sharp decrease of additional interchange delay with slight drop in additional VMT is observed when the length included grows from 1 mile to 1.5 mile. The difference in trends indicates that congestion around interchange locations tends to be alleviated significantly out of a 1 mile scope. After several rounds of empirical tests, the authors finally used 1 mile as the search length to attribute interchange constraint to a bottleneck.

The identification of the other three types of bottlenecks follows the definitions in White Paper (2005). The location qualified as a freight bottleneck is examined to determine if a change of number of lanes on neighboring sections or if at least one at-grade signal exists by using HPMS data, which features the location as lane-drop bottleneck or a signalized intersection bottleneck, respectively. The steep-grade bottleneck is characterized as a congested section with more than 1 mile of steep grades (i.e., grade greater than 4.5%). By investigating all constraints identifiable from HPMS data on each bottleneck, the potential causes leading to congestion are explored.

RESULTS AND CONCLUSIONS

After applying the proposed bottleneck analysis method to the 2006 HPMS data for the 10 states in the Mississippi Valley region, the authors arrive at a master list of regional freight bottlenecks with all constraints checked for each bottleneck. Table 1 shows the number of locations identified for each type of bottleneck condition. The truck bottlenecks are further prioritized by the truck unit delay associated with the existing conditions (see Figure 2 for the distribution of truck unit delay and its range for each type of bottlenecks). The prioritization result of bottleneck identification further confirms that the interchange constraint accounts for the most significant bottleneck condition, followed by the lane-drop constraint. The steep-grade bottlenecks are only associated with a marginal truck unit delay because such sections are usually located in a rural area in the study region where general traffic demand is not intense and congestion is not as severe as that in urban area. However, the great length of sections with steep grade tends to aggravate this issue and might warrant the concerns when the travel demand increases.

Table 1. Number of truck bottlenecks identified from 2006 HPMS data for the Mississippi Valley region

Bottleneck Type	On Freeways	On Other Principle Arterials	Total
Interchange	246	0	246
Signalized Intersection	2	727	729
Lane Drop	486	209	695
Steep Grade	4	0	4

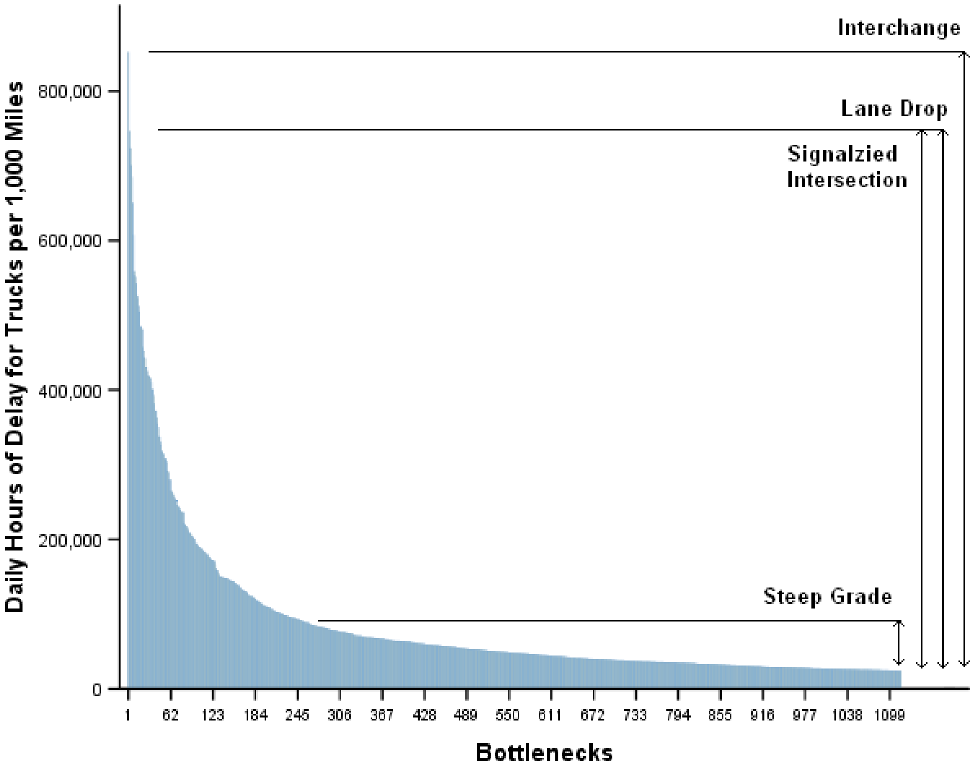

Figure 2. Distribution of truck unit delay across truck bottleneck locations

This ranked list also facilitates the verification of identification results with various sources, including the knowledge from state transportation engineers and planners, user nominations, and previous study results. As driven by the same database in different years, the study results provides a good match with the identification results in White Paper (2005). It also confirms the user-nominated bottleneck locations obtained from a series of surveys with truck drivers and dispatchers in selected states. However, as pointed out by local experts in some states, our methodology fails to identify certain bottleneck locations that reside on highway segments not present in the HPMS sample data. This is a data and methodological limitation that needs to be addressed by supplementing the HPMS-based analysis with local knowledge.

In summary, this study develops a framework to identify, characterize, and prioritize freight bottlenecks on a regional level in the Upper Midwest area. Particularly, the truck unit delay measure is proposed as a truck bottleneck indicator and a congestion corridor growing method is incorporated in the analysis framework to account for the system congestion caused by interchange bottleneck. As the output of this freight bottleneck study, a ranked list of truck bottlenecks serves to stimulate cross-sector dialogue among freight planners and operators and provides a basis to devise the optimal alleviation plan for the greatest benefits for the region.

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Deploying Hybrid Electric School Buses in Iowa

Shauna Hallmark
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
shallmar@iastate.edu

Brittany L. Hallmark
Ankeny High School
306 SW School Street
Ankeny, IA 50023

Dennis Kroeger
Office of Motor Carriers and Highway Safety
Federal Highway Administration
1200 New Jersey Ave. SE
Washington, D.C. 20590

ABSTRACT

There are over 450,000 school buses in the United States transporting 24 million children, resulting in 4 billion miles of travel per year. School children spend an average of one and a half hours per day on a school bus (1,2) that is primarily diesel powered. These buses consume 1.1 billion gallons of diesel fuel annually and emit thousands of tons of pollutants per year (Ewan et al. 2004). One source measured on-board emissions in Connecticut school buses and found that $PM_{2.5}$ emissions were 5–10 times higher on school buses than at fixed site monitoring sites (3). Diesel exhaust affects children with respiratory problems, such as asthma and bronchitis. Additionally, rising fuel costs are a large concern for school districts.

Options to decrease fuel costs and emissions include using different fuels such as biodiesel or natural gas and adding on-emission control devices such as particulate filters and oxidation catalysts. Hybrid electric technology is another option. Hybrids are available in the passenger vehicle market as well as the transit bus market. Until recently, however, there have not been any commercially available hybrid-electric school buses.

Hybrid-electric school buses have the potential to reduce emissions and to reduce the overall life cycle cost when compared to conventional diesel buses. The technology has been demonstrated in passenger vehicles and transit buses. A study found that hybrid transit buses had a 10% higher in-service fuel economy than regular buses. Results of chassis dynamometer tests suggested that fuel economy could be 23% to 64% higher. They also reported results of a chassis dynamometer tests which indicated that CO emissions were 56% to 98% lower than for regular buses, NO_X emissions were 36% to 44% lower, hydrocarbon emissions varied from 43% lower to 88% higher, PM emission were 50% to 99% lower, and CO_2 emissions were 19% to 40% lower. Results varied based on test cycle used. Bus operators also indicated that hybrid buses had better acceleration, better traction in bad weather, and had smoother braking than conventional transit buses (4).

Although hybrid-electric school buses are promising, until recently, the technology was not widely available. The Hybrid Electric School Bus (HESB) Project is a program designed to bring these vehicles to market by creating the demand among school districts required for a manufacturer to invest in the

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development of the technology (5). One phase of the project is to demonstrate that hybrid school buses can provide an economically viable alternative for school districts seeking to reduce emissions from their fleets. However, to penetrate the school bus market, there must be a demonstration of the technology (2).

As part of a national coordinated effort, 14 school districts in the United States, including two school districts in Iowa, have stepped forward to join a national consortium to encourage the demand for hybrid electric school buses. The Center for Transportation Research and Education (CTRE) at Iowa State University (ISU) is conducting the evaluation for the two Iowa school buses. This paper discusses the Iowa program and provides early results.

Key words: diesel fuel—emissions—hybrid electric school bus

Alternative Gas Tax Options

Paul Hanley
Department of Civil and Environmental Engineering and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
paul-hanley@uiowa.edu

ABSTRACT

This paper describes and summarizes the first year of field operations for a National Evaluation of a Mileage-based Road User Charge. The study, which is the second phase of an evaluation of alternative funding mechanisms to potentially replace the current motor fuel tax, was authorized by the 2005 federal transportation reauthorization act (SAFETEA-LU). It is the first such study to access road user charging on a national and multijurisdictional scale.

During the first year of the study, 1,500 volunteer participants were selected from a pool of approximately 40,000 candidates to evaluate a system for automatic collection, reporting, and invoicing of mileage-based road use charges. Participants were recruited and selected from six areas throughout the country to mirror a variety of demographic characteristics of the U.S. population as a whole. The road user charge system is able to collect and apportion mileage charges to multiple jurisdictions, including federal, state, and local. The system utilizes an on-board unit (OBU) installed in the participant's vehicle. The OBU contains a GPS receiver with an associated geographic database to identify the taxing jurisdiction(s) in which the vehicle is travelling. The OBU obtains vehicle miles travelled (VMT) information from the electronic odometer data available on the vehicle's OBD-II bus. Mileage charges for each relevant jurisdiction are computed by the OBU and uploaded to a billing center via a wireless data link,

Throughout the study, participants are administered a series of surveys to assess their attitudes and perceptions regarding various aspects of the system and the overall concept of mileage-based charging.

Key words: fuel tax—road user charge

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Development of a Statewide Horizontal Curve Database for Crash Analysis

Zachary Hans
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
zhans@iastate.edu

Rachael Larkin
Department of Kinesiology
Iowa State University
235 Forker Building
Ames, Iowa 50011
larkin13@iastate.edu

Reg Souleyrette
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
reg@iastate.edu

ABSTRACT

Lane departure crashes represent the single highest category of fatal and major injury crashes in Iowa, including 60% of all fatal crashes (Iowa CHSP). While horizontal curves represent only a small portion of the public roadways in Iowa, 15% of all fatal single-vehicle run-off-road crashes (25% of the major injury crashes) and 11% of all fatal multiple-vehicle cross-centerline/cross-median crashes (18% of the major injury crashes) occur on curves (Iowa CHSP). Therefore, a great opportunity exists to reduce death and serious injuries if the safety of problem curves can be improved. Particularly promising is that the many of the possible improvements at horizontal curves are relatively low cost, such as paved shoulders, rumble strips and stripes, and improved signing and delineation. However, the State of Iowa does not maintain a database of horizontal curves. The objective of this project was to create a comprehensive curve database for use in crash analysis, with emphasis on high-speed rural two-lane roadways. Secondary objectives were to create the database in a systematic manner, without requiring additional field data collection, and extract curve parameters, such as length, radius, and degree of curvature where possible. Ultimately, a database of possible curve locations on all paved high-speed rural two-lane roadways (primary and secondary) in the state was created using GPS data collected as part of Iowa's Pavement Management Program (IPMP). A statewide crash analysis was also performed to identify the top 200 high crash frequency curves and curvilinear sections of roadway based on total crash frequency and total fatal and major injury crash frequency. This analysis was limited to crashes most likely to be curve-related. Site maps, including crash data and aerial images, site descriptions, and summary crash statistics were also prepared as part of the analysis.

Key words: crash—curve database—horizontal curves

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Cost Analysis of Alternative Culvert Installation Practices in Minnesota

Brad Hansen
Department of Bioproducts and Biosystems Engineering
University of Minnesota
1390 Eckles Ave.
St. Paul, MN 55025
hanse038@umn.edu

Chris Lenhart
Department of Bioproducts and Biosystems Engineering
University of Minnesota
1390 Eckles Ave.
St. Paul, MN 55025
Lenh0010@umn.edu

John Nieber
Department of Bioproducts and Biosystems Engineering
University of Minnesota
1390 Eckles Ave.
St. Paul, MN 55025
nieber@umn.edu

ABSTRACT

Various factors associated with conventional culvert design, including shallow water, perched inlets, and high-flow velocities, can cause difficulties for migrating fish and affect their genetic diversity and long-term survival. Conventional culvert design has traditionally been based on hydraulic conveyance, safety, and cost. Recently, some alternative culvert designs have been developed to facilitate salmon migration on the West Coast of the United States. These alternative designs focus on matching the natural dimensions and characteristics of the stream channel through the culvert. The intended purpose of these newer designs is to provide unimpeded passage of aquatic life, reduce maintenance costs, and improve erosion control. Currently, some of these new designs are being implemented in Minnesota, mostly when fish passage is a consideration. There are concerns about the additional costs associated with these alternative designs as well as whether they are really needed at some road crossings. The objectives of this research were to summarize statewide fish-passage concerns related to culvert road crossings on public waters and to perform a cost comparison between the conventional and the alternative culvert designs.

Key words: culvert design—economics—fish passage—stream health

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PROBLEM STATEMENT

The research associated with this report was originated by a problem statement from county engineers concerned about the necessity, function, and additional costs of designing culverts for fish passage. The Department of Bioproducts and Biosytems Engineering at the University of Minnesota received funding from the Minnesota Local Roads Research Board (LRRB) to conduct the research. A technical advisory panel was formed to advise and direct the research problem. Committee members included county engineers, private consultants, Minnesota Department of Natural Resources (DNR) fisheries specialists, and Minnesota Department of Transportation (Mn/DOT) environmental scientists and engineers.

RESEARCH OBJECTIVES

The final three main research objectives were

1. Evaluate fish- and sediment-passage guidance for culverts in the Upper Midwest
2. Determine a statewide picture of fish-passage concerns related to culvert road crossings on public waters
3. Provide cost analysis of alternative culvert design

RESEARCH METHODOLOGY

Evaluate Fish- and Sediment-Passage Guidance for Culverts in the Upper Midwest

This objective was accomplished by a review of the literature related to fish passage concerns in the Upper Midwest. Most of the research done on fish passage through culverts has focused on salmon and trout in the western United States. The amount of literature available for the Upper Midwest was limited. This objective identified the successful fish passage research done on the East and West coasts and determined how well it translated to Minnesota.

Determine a Statewide Picture of Fish-Passage Concerns Related to Culvert Road Crossings on Public Waters

The three steps listed below outline the actions that were taken to address the second objective.

1. Phone conversations with area and regional hydrologists provided information from a regional prospective
2. A review of statewide general and county permits provided background information about current state requirements for culvert installations
3. Online and mailed surveys to county engineers and DNR personnel provided feedback from the practitioners that are involved with fish passage concerns at culverts

Provide Cost Analysis of Alternative Culvert Design

A cost comparison was performed between an alternative culvert installation designed to improve fish passage and stream function and the conventional hydraulic design based on conventional culvert objectives of flood capacity and safety. The alternative design chosen was MESBOAC (Match, Extend, Set, Bury, Offset, Align, Consider). MESBOAC is based on principles of fluvial geomorphology rather than individual fish swimming ability. It was developed in the northern forested region of Minnesota for the U.S. Forest Service but is applicable to the Upper Midwest in general. Each of the seven elements of MESBOAC is described below.

1. Match culvert width to bankfull stream width
2. Extend culvert length through the sideslope toe of the road
3. Set culvert slope the same as stream slope
4. Bury the culvert 4 to 18 inches into the stream bottom. Depth depends on culvert size
5. Offset multiple culverts
6. Align the culvert with the stream channel
7. Consider headcuts and cutoffs

MESBOAC aims to match the culvert width with natural stream dimensions while maintaining sediment balance (sediment in = sediment out). In addition to recessing the culvert invert below the streambed, it also provides for a low-flow channel, which is important for late season migrations (August to November). MESBOAC has the advantage of not requiring analysis of fish-passage flows. It assumes that since the natural flow characteristics are maintained, fish passage will occur. It also tends to minimize maintenance needs over time by reducing scour or aggradation. It has the disadvantage of requiring larger culverts than traditional hydraulic design. It also requires identification of the bankfull elevation, which takes substantial experience or flow-frequency analysis.

To do a cost comparison and determination of how well MESBOAC met traditional culvert design objectives, 15 work plans of recently installed, conventional-design culverts were acquired from around Minnesota. Plans were obtained from counties that represent regional differences in culvert installations, rainfall, and hydrology. Where available, the hydraulic analysis and actual-bid costs for each installation were also obtained. The 15 work plans were re-engineered using the design elements of MESBOAC. Re-engineering involved replacing the conventional in-place culvert design with MESBOAC while maintaining conventional design objectives. The re-engineering was conducted to meet flow capacity, headwater, and stage increase conditions set for the current installations while also meeting the criteria set for MESBOAC. Differences in cost between the conventional culvert design and MESBOAC approach were determined for 11 of the 15 sites.

KEY FINDINGS

Evaluate Fish- and Sediment-Passage Guidance for Culverts in the Upper Midwest

Fish-passage problems at road crossings are widespread across the United States. We researched East and West coast fish-passage issues and techniques for comparison to the Upper Midwest, especially Minnesota. In terms of hydraulics and channel maintenance, the problems creating blockage of fish passages are similar, including excess drop at culvert outlets, high in-pipe velocity and/or turbulence, inadequate water depth in pipe, excess pipe length without fish resting space, and debris or sediment accumulation in-pipe. The major differences are fish species, stream geomorphology, and hydrology. The key upper midwestern fish species—walleye, pike, bass, trout, and panfish—have different life histories and movement patterns than coastal anadromous fish migrating from the ocean, with movement between lakes and rivers taking on greater importance. Upper midwestern rivers are different geomorphologically than most West Coast salmon rivers, as they tend to be lower in gradient. Therefore, geomorphic considerations are important for preventing accumulation of fine sediment and fish blockage at low flow as well as at high flows. Overall, the tools and techniques used in the coastal United States. are applicable to the Upper Midwest. The major differences lie in the prioritization of the issues and the types of fish species targeted for management.

Determine a Statewide Picture of Fish-Passage Concerns Related to Culvert Road Crossings on Public Waters

The importance of fish passage at culverts is addressed by the Minnesota Department of Natural Resources (MNDNR) usually only when the culvert needs to be replaced, which occurs most commonly due to road construction. On public waters, a permit is required from MNDNR before the county or state can proceed with the construction. The culvert design is influenced by the importance of fish passage and is usually worked out between agencies to make sure both fish and adequate flows can pass. The process usually occurs on a case-by-case basis. Local governments have accepted alternative designs, and the additional costs of the benefits to the fishery are obvious. Friction remains between the two agencies when benefits aren't agreed on.

There is not a regional or statewide ranking or prioritization system for fish passage in the state that can be used to identify high-priority road crossings that require more analysis and design. Some aspects of different alternative designs are being implemented in different areas of the state, usually in response to specific local conditions concerning fish passage (e.g., trout streams in southeast and steeper gradient streams in northeast). Some of the different techniques being used are low-flow channels with multiple culverts, V-notch weirs and backwater weirs, culvert alignment, rock baffles inside culverts, and MESBOAC. Most of the design expertise for fish-passage culverts is with the MNDNR. As a result, the data collection and type of design chosen falls on MNDNR personnel. The function of these alternative designs has not yet been evaluated.

MNDNR and area-hydrologists ranked fish-passage concerns as the most important criteria for culvert design, followed by controlling wetland water elevations, flood capacity, and matching existing channel characteristics tying for second rank. Future maintenance costs, controlling water level elevations on agricultural lands, and total installation cost ranked fourth through seventh, respectively.

The previous general permit referenced a flow requirement of "two-year peak flow velocities of no greater than two feet per second" under the fish passage condition. The current general permit wording is "when possible, a single culvert or bridge shall span the bankfull width adequate for natural debris and sediment transport rates to closely resemble those of upstream and downstream conditions". The rest of the wording under the fish passage condition closely matches the seven elements of MESBOAC.

Provide Cost Analysis of Alternative Culvert Design

Work plans for 15 recently installed culverts were acquired from around the state. The site data is summarized in Table 1. All 15 of the installations were box culverts. Thirteen sites were bridge replacements and two were culvert replacements. Eight sites had single-barrel culverts and seven sites multiple barrels.

None of the 15 culvert installations examined had to be modified to match three of the elements of MESBOAC—Extend, Align and Consider headcuts. The four remaining elements—Match bankfull width, Set culvert slope to stream slope, Bury culvert bottom, and Offset multiple culverts—were modified from the original design to match MESBOAC requirements.

Design Analysis

The following parameters all had an effect on design and potential costs and were considered in re-engineering the work plans:

Bury Bankfull-Width Culvert One Foot Below Channel Bed

One of the key elements of a MESBOAC design is to have bed material in the culvert closely match that of the natural channel. All re-engineered designs had a single or double culvert, closely matching bankfull width and recessed one foot below the flowline of the stream bed. The flowline elevation used was taken directly from the work plan. Recessing the culvert allows natural stream sediments to fill in the culvert bottom reducing the flow velocity at that interface. How deep a culvert invert should be recessed can be influenced by culvert size, flood capacity and size of bed material. One foot below the channel bed elevation was chosen for all sites to simplify the cost comparison. Recessing a culvert one-foot below grade can reduce the capacity of the culvert. This results in choosing a larger more expensive culvert for a MESBOAC design versus a conventional design.

Match Recessed Culvert Width to Channel Bankfull Width

Recessed culvert width was selected to match or slightly exceed the channel bankfull width. The widest standard-size culvert is 16 feet. For streams with bankfull width greater than 16 feet, multiple culverts were used to match the bankfull width.

Table 1. Background data on culvert sites

Location	DA (sq. mi.)	Design flood return period (years)	Design flow (cfs)	Number culverts
Aitkin (Snake R. Trib.)	16.4	65	590	1
Cass (Leavitts Lake Channel)	12.7	100	302	1
Cottonwood (So. F. Watonwan)	14.4	100	1100	2
Cottonwood (Unnamed)	5.4	100	1290	2
Fillmore (Donaldson)	9.2	100	3100	3
Fillmore (Duschee)	17.4	30	1260	3
Fillmore (Money Cr.)	18.1	15	1721	2
Jackson (W.F.Little Souix)	68.7	100	2170	3
Kandiyohi (CD27)	11.5	100	555	1
Lincoln (N.B.Yellow Medicine R.)	1	100	419	1
Lincoln (Yellow Medicine R.)	17	100	418	1
Meeker (Unnamed)	19.8	100	530	1
Mille Lacs (Mike Drew Br.)	5.2	10	195	1
Mille Lacs (Tibbets Br.)	9.1	75	915	1
Saint Louis (Stanley Cr.)	1.81	50	454	2

Bankfull Width is a Critical Dimension of MESBOAC

Determining proper bankfull width can take some on-site detective work. This report used a combination of on-site measurements, cross sections from the culvert work plans, and regional curves developed for Minnesota to determine bankfull width (Magner 2008). Table 2 contains the bankfull data for each of the study sites.

Four sites located in Lincoln, Meeker, Kandiyohi, and Cass Counties had bankfull channel widths greater than the existing culvert width. Because the flow gets funneled into a smaller area, the culvert velocity will be greater than channel velocity at bankfull flow. This increase in velocity could be enough to prohibit fish passage. The Cass County site had restrictions that did not allow the installation of a culvert that would match the bankfull width. A simulated roughened channel consisting of eight large boulders placed in the culvert helped reduce the velocities and provide resting locations for fish. None of these sites had offset culverts. Seven sites had culvert widths 3 to 15 feet greater than the channel bankfull width. This situation can lead toward sedimentation or flow depths too shallow for fish passage. The far right column in Table 2 shows the MESBOAC bankfull culvert width chosen for comparison to the bankfull channel width. It does not include the width of additional offset culvert barrels.

Table 2. Bankfull data

Location	**DA (sq. mi.)**	**Number of existing culverts**	**Total existing culvert width all barrels (ft.)**	**Bankfull channel width (ft.)**	**Re-engineered MESBOAC bankfull culvert width (ft.)**
Aitkin (Snake R. Trib.)	16.4	1	16	14	16
Cass (Leavitts Lake Channel)	12.7	1	14	23	14
Cottonwood (So. F. Watonwan)	14.4	2	28	13	14
Cottonwood (Unnamed)	5.4	2	22	8	12
Fillmore (Donaldson)	9.2	3	36	17	16
Fillmore (Duschee)	17.4	3	36	22	24
Fillmore (Money Creek)	18.1	2	24	21	24
Jackson (W.F.Little Souix)	68.7	3	30	28	24
Kandiyohi (CD27)	11.5	1	10	12	12
Lincoln (N.B.Yellow Medicine R.)	1	1	8	4	8
Lincoln (Yellow Medicine R.)	17	1	10	12	12
Meeker (Unnamed)	19.8	1	10	12	12
Mille Lacs (Mike Drew Br.)	5.2	1	14	11	14
Mille Lacs (Tibbets Br.)	9.1	1	10	9	10
Saint Louis (Stanley Cr.)	1.81	2	20	10	10

Offset Culverts Placed Two Feet above Bankfull Culvert Invert

Seven sites had multiple culverts in the original design. Offsetting multiple culverts at higher elevations keeps the main bankfull culvert(s) at the proper width to maintain natural channel characteristics, while allowing a larger design flow to pass without overtopping the road. It also reduces the chances of sediment collecting in the offset barrels. The re-engineered plans set any multiple culverts outside of the bankfull width at an elevation two feet higher than the bankfull culvert invert or one foot above the streamline elevation. To meet needs of the overlying roadbed, the offset culvert(s) were sized so that their tops would have the same elevation as the recessed culvert(s).

Match Design Headwater Elevation

Headwater elevation is the water elevation at the head of the culvert at design flow. Matching the headwater elevation of the original work plans was one of the criteria used to insure the MESBOAC design could adequately pass the design flow of the original work plan The HY8 software, a culvert analysis program produced by the Federal Highway Administration (FWHA) was used to do the analysis (FWHA 2008), In the plan, headwater elevations came directly from the hydrologic and hydraulic analysis done for the culvert replacement. The in-place headwater elevations were generated by HY8 from the work plan culvert data. The two values matched closely enough (+/-0.35 ft.) to give us confidence that our interpretation of the work plan matched those of the original designer. For the MESBOAC design culvert sizing was performed to match the criteria listed above and the headwater elevation of the in-place culvert. The MESBOAC and in-place headwater elevations matched within (+/-0.3 ft.)

Minimize or Reduce Stage Increase

For this report, stage increase is defined as the difference between the headwater and tailwater elevation. Section 13 of the statewide general permit (Mn/DOT 2008) states that new crossings or replacement of existing crossings that have a stage increase of 0.5 feet or less shall have no greater stage increase than 0.5 feet. Replacement structures of existing crossings that have greater than 0.5 feet increase in stage can match the existing structure's stage increase if it does not impact upstream flooding. All of the work plans examined were replacement structures with existing stage increases of 0.5 feet or greater. Culverts were sized to match the existing structures stage increase. The MESBOAC designs increased the stage increase over the existing structure stage increase for three sites but by no greater than 0.16 feet. All the other sites showed a reduction in stage increase.

Slope

Slope also plays a significant role in controlling velocities through culvert structures. Nine culverts had slopes greater than the channel slope. Depending on the tailwater conditions, culvert slopes greater than the channels slopes could produce velocities that would inhibit fish passage. The tailwater conditions of the re-engineered work plans were unknown. Calculations of difference in velocities between the channel and inside the culvert were not calculated.

Cost Comparison

Table 3 outlines the cost differences for 11 of the 15 original work plans. Cass County was more of a lake channel than a stream; the two sites in Lincoln County had incomplete data, and the Stanley Creek site in St. Louis County is already a MESBOAC design. The additional cost of installing a MESBOAC design basically came down to culvert sizing and some additional excavation or fill. As mentioned before, the culvert alignment and culvert extension (beyond toe slope) did not have to be modified. Additional costs for offsetting multiple culverts, changing slope, and burying the bankfull-width culvert is either excavation or additional bed aggregate. At an average price of $3.00/yd for excavation and $15.00/yd for aggregate, the additional costs to the total project are minimal. Additional excavation and aggregate costs ranged from $100 to $1,850 for the eleven sites.

Table 3. Cost comparison

Location	Culvert cost (dollars)		Difference	Difference as percent
	In-place	MESBOAC		
Aitkin (Snake R. Trib.)	32,512.2	35,429	2916.8	9
Cass (Leavitts Lake Channel)	-	-	-	-
Cottonwood (So. F. Watonwan)	7,1795	74,754	2959	4
Cottonwood (Unnamed)	73,043.6	77,423	4379.4	6
Fillmore (Donaldson Cr.)	167,095.6	188,604	21508.4	1.3
Fillmore (Duschee)	121,885.4	123,323.2	1437.8	1
Fillmore (Money Cr.)	83,188	88,942.4	5754.4	7
Jackson (Little Sioux)	81,811.8	77,894	-3917.8	-5
Kandiyohi (CD27)	62,914.6	78,828.4	15913.8	25
Lincoln (Unnamed trib.)	-	-	-	-
Lincoln (Yellow Medicine)	-	-	-	-
Meeker (Unnamed)	29,197	38,920.4	9723.4	33
Mille Lacs (Mike Drew)	39,041.8	42,084.6	3042.8	8
Mille Lacs (Tibbets Brook)	20,178.2	22,370	2191.8	11
St. Louis (Stanley Creek)	-	-	-	-

Culvert sizing came down to matching bankfull width and maintaining adequate capacity. When a bankfull culvert is recessed one foot, an increased culvert diameter of one foot is needed to maintain the same capacity. The costs listed in Table 3 do not reflect the entire cost of a culvert replacement. They include the cost for the culvert(s) itself without consideration for any additional cuts or fill materials. The cost differential ranged from -5% to 33%. This is not an increase of the total project cost. It is a percent increase in the cost of the culvert structure. County engineers we talked to estimate the cost of the culvert structure generally is about 50%–70% of the total project cost. The average percent increase of the MESBOAC design culverts over the in-place culverts was 10%. The highest cost difference was 33% for the Meeker County site. It required a larger culvert both in width and height as the in-place structure was undersized for bankfull width. The Jackson County site showed a reduction in cost for the MESBOAC design as two 14’ by 10’ culverts were used to replace three 10’ by 9’ culverts. The cost reduction was mainly due to requiring only four culvert end sections instead of six.

Other Alternative Culvert Designs

Baffles, roughened channels, and backwater weirs were identified as alternative designs currently being used in Minnesota. These alternative methods are not designed as a replacement for traditional culvert designs but as an addition to an in-place design to facilitate fish passage. Generalized costs were calculated for all three designs. Costs were calculated as a percentage of the cost for the average bankfull-width culvert from the 12 work plans examined in this report. For these cases, the cost of the culvert structure is generally more than half of the total project cost. Tying the cost of alternative designs to the cost of the culvert as a percentage should allow an estimate of alternative design costs as the scale of the project increases or decreases. In the case of backwater weirs, the cost per foot of stream width was used as well as bankfull-culvert cost.

Material costs, transportation, design, labor, and equipment were considered in calculating the cost of the alternative designs. Information about project specific conditions, such as access to the stream bed, time of the year, location of utilities, construction sequence, and permits, were not considered but could greatly influence the total cost of the project. A summary of the costs associated with the three alternative designs is presented in Table 4.

Table 4. Alternative design cost as a percent of bankfullwidth culvert cost

Design	Range ($)	Average Cost ($)	% of Culvert Cost
Baffle	1,000–8,000	4,000	12.5
Roughened Channel	2,400–3,600	3,200	10
Backwater Weir	2,000–7,700	4,850	15.1

Baffles have materials, design, and installation cost but little in terms of transportation costs. Transportation of rock or logs, as well as the expense of the proper equipment to handle it on-site, play a big role in roughened-channel and backwater-weir design costs. Backwater weir designs are more expensive because they require additional excavation or fill and erosion control beyond that of the culvert installation. If done at the time of culvert installation, mobilization costs for all three designs will be cheaper as supervision, labor, and equipment will already be on-site.

Meet Conventional Culvert Design Requirements

The following discussion refers to the use of baffles, roughened channels, and backwater weirs. Some of the conventional design objectives for culverts are

- Safely provide public transportation
- Remain stable and pass worst-case design flood
- Minimize maintenance problems
- Reduce up-stream flooding potential
- Control scour and erosion above and below culvert

There is limited literature as to how well these alternative designs are performing with respect to the above conventional culvert objectives. Also, there is a lack of data on how well alternative designs are actually performing their design function. The following comments are interpretations of the authors.

If designed properly, the alternative designs should have a minimal effect on safety and ability to pass design flows. Erosion and scour should also not be increased, and if designed properly, may be reduced.

Maintenance issues could be increased as all three alternative designs have the potential to catch sediment and debris. Debris accumulating in a culvert due to baffles or roughened channels could significantly reduce the hydraulic capacity of the culvert.

The design life for these alternatives probably is shorter than for the structure itself. The literature provides a number of examples of baffles and rocks being washed out of culverts (Bates et al. 2003). In conversations with county and MNDNR officials, we heard of two cases in which baffles were washed out of culverts in Minnesota. This may suggest the use of these designs in retrofit situations, but for complete replacement, a MESBOAC or stream simulation design may be preferred.

Summary

Overall, the tools and techniques used in the coastal United States are applicable to the Upper Midwest. In terms of hydraulics and channel maintenance, the problems creating blockage of fish passages are similar, including excess drop at culvert outlet, high in-pipe velocity and/or turbulence, inadequate water depth in pipe, excess pipe length without fish resting space, and debris or sediment accumulation in-pipe. The major differences are fish species, stream geomorphology, hydrology, and prioritization of the issues and the types of fish species targeted.

There is no regional or statewide ranking or prioritization system for fish passages in the state that can be used to identify high-priority road crossings that require more analysis and design.

Some aspects of different alternative designs are being implemented in different areas of the state, usually in response to specific local conditions concerning fish passage (e.g., trout streams in southeast and steeper gradient streams in northeast). Some of the different techniques being used are low-flow channels with multiple culverts, V-notch weirs and backwater weirs, culvert alignment, rock baffles inside culverts, and MESBOAC. Most of the design expertise for fish-passage culverts is with the MNDNR. As a result, the data collection and type of design chosen falls on MNDNR personnel. The function of these alternative designs has not yet been evaluated.

The main components of the MESBOAC design of recessing culverts, matching bankfull width and offsetting multiple culverts were fit into the footprint and maintained the design objectives of headwater, stage increase, and flood capacity of the conventional-design culvert. For all cases examined except for one, the MESBOAC design would cost more than the corresponding conventional design. The increase in cost for the culvert structure ranged between -5% and 33%. If the purported additional benefits beyond improved fish passage including reduced erosion and maintenance costs prove to be true, this could offset the additional costs associated with needing a larger culvert for MESBOAC. Matching culvert and channel slope plays an important role in affecting flow velocities; it has little affect on increasing the cost of a project.

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Concrete Overlays Helping to Address the Nation's Preservation/Rehabilitation Needs

Dale Harrington
Snyder and Associates
2727 SW Snyder Blvd.
P.O. Box 1159
Ankeny, IA 50023
dharrington@snyder-associates.com

ABSTRACT

This presentation will discuss how concrete overlays can help address the need to cost-effectively renew the nation's 4 million mile transportation system. The products developed through the research to date, the field application support program, and outcomes expected from the balance of the program of work will be presented.

This presentation is part of the Sustainable Concrete Pavement Technologies session.

Key words: concrete overlays—cost-effective—rehabilitation

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Can Air be Entrained in Roller-Compacted Concrete Mixes?

Chetan V. Hazaree
Department of Civil, Construction, and Environmental Engineering
Iowa State University
136 Town Engineering
Ames, IA 50011-3232
chazaree@iastate.edu

Kasthurirangan Gopalakrishnan
Department of Civil, Construction, and Environmental Engineering
Iowa State University
354 Town Engineering
Ames, IA 50011-3232
rangan@iastate.edu

Halil Ceylan
Department of Civil, Construction, and Environmental Engineering
Iowa State University
394 Town Engineering
Ames, IA 50011-3232
hceylan@iastate.edu

EXTENDED ABSTRACT

Entraining air in dry concretes is a critical problem. Freeze-thaw (F-T) resistance requires concrete to possess a certain configuration of air content for providing adequate durability and performance. Research in the past has not explicitly explored the possibility of entraining air in roller-compacted concrete (RCC) over a wide range of cement contents. In addition, subjective data on the F-T resistance of RCC limit the wide-scale acceptance and application of RCC for pavements.

This paper illustrates the possibility of entraining air in RCC. Eight comparable mixes were cast over a wide range of cement contents to understand the effects of changing the cement content on the air entrainment and microstructure. The applications include pavement bases and wearing courses. Laboratory prepared mixes were used in extracting helpful scanning electron microscope (SEM) specimens.

The paper discusses the features of entrained air, including its size distribution and shapes seen under the SEM. A special software comparable to automated ASTM C 457 was used, and the mixes were analyzed for air entrainment. Results from ASTM C 457 are discussed. It was observed that irrespective of the cement contents, air can be successfully entrained in RCC. The performances of the RCC mixes for the F-T durability are also discussed in the paper.

Key words: air entrainment—image analysis—pavements—roller-compacted concrete (RCC)—scanning electron microscope (SEM)

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INTRODUCTION

Roller-compacted concrete (RCC) technology is a rapidly evolving concrete material and construction methodology. Due to its unique nature, it is widely applied in diverse engineering applications, ranging from hydraulic to pavement structures. RCC pavement (RCCP) technology is finding a growing number of applications in the form of pavement bases, low-maintenance roads, parking areas, cargo loading areas, docks, fast-track intersections, city streets, shoulder reconstruction, etc. In the form of pavement base course, RCC is applied with a 7-day average (cube) compressive strength of 10 MPa (1,500 psi), while in the form of wearing course, it is applied with a 28-day average compressive strength of 40 MPa (5,800 psi).

Conventionally entrained air is characterized as a parameter indicative of the freeze-thaw (F-T) resistance of concrete. Air bubbles can only be formed in concrete mixture if there is a sufficient amount of water, since each bubble has to be surrounded by a film of water (Powers 1964). In the majority of laboratory and field studies conducted on RCC (Whiting 1985; Marchand, Pigeon, Boisvert, Isabelle, and Houdusse 1992), it was observed that a substantial amount of entrapped air is present, while the entrained air was merely formed. In one of the investigations (Delagrave, Marchand, Pigeon, and Boisvert 1997), the air-void characteristics of the RCC mixtures showed a very low (0.1%–0.6%) spherical air bubble content; the spacing factor (84-169 µm) and the specific surface were relatively much lower (117-21.9 mm^{-1}). These observations lead to conclusions that the voids resulting from compaction exist in the concrete matrix in relatively much higher quantities and are relatively larger.

A comprehensive study was thus undertaken with an objective of understanding the differences in the air-entrained and non-air-entrained RCC. The distribution of air-void systems was also studied. The results can be utilized in understanding the exact behavior of compaction voids and spherical entrained air voids in the F-T response of RCC systems.

RESEARCH FOCUS AND MIXTURE PROPORTIONS

The study was focused on two sets (four mixes each) of comparative mixes. Set I (RI) consisted of four mixes with cement contents of 150, 250, 350, and 450 kg/m^3, while Set II (RII) consisted of corresponding mixes with identical cement contents and entrained air in them. Fresh properties, mechanical strength, and F-T durability were measured in addition to scanning electron microscope (SEM) imaging and air-void analysis. Results other than that derived from SEM and air-void analysis are considered to be out of scope of this paper and are submitted for publication elsewhere. Visilog version 4.1.5 programmed by Noesis Vision was used for image and air-void analysis. Locally available materials were used for preparing concrete samples.

METHODS

Scanning electron microscope

A Hitachi S-2460N variable pressure SEM was used in low vacuum mode (40 Pa). Backscattered electrons (BSE) were used to collect images at 20 kV accelerating voltage with 25 mm working distance and approximately 0.5 nA of beam current. BSE generates a specific phase contrast, which allows phases to be identified according to their brightness in the image—those with the greatest atomic number are the brightest, and those with the lower atomic number are darker. This allows the components of the microstructure to be discriminated on the basis of gray level. A minimum of 20 frames were sketched on each specimen, and SEM images were obtained on each frame. These images were used in studying the microstructure and air void structure of concrete mixes. Table 1 shows the steps used for image processing

Table 1. Operations used in image processing

Operation	Function
Gradient	The boundaries between the phases are enhanced and turned into high (light) grey levels, while the bulk of the phase is turned into low (dark) grey levels
Dilation (causes objects to dilate)	Filling of small holes, smoothing of contour
Erosion (Causes objects to shrink)	Disappearance of objects smaller than a mask, eliminates noise, produces smooth shape
Opening (erosion followed by dilation)	Separates objects that are connected in a binary image, smoothes from the inside of the object contour
Closing	Fills in small holes, smoothes from the outside of the object contour
Hole filling	Recovers the missing holes within any void without any change to its size
Labeling	Colors different voids with different colors

RESULTS

Figure 1 shows the results of the cumulative air contents.

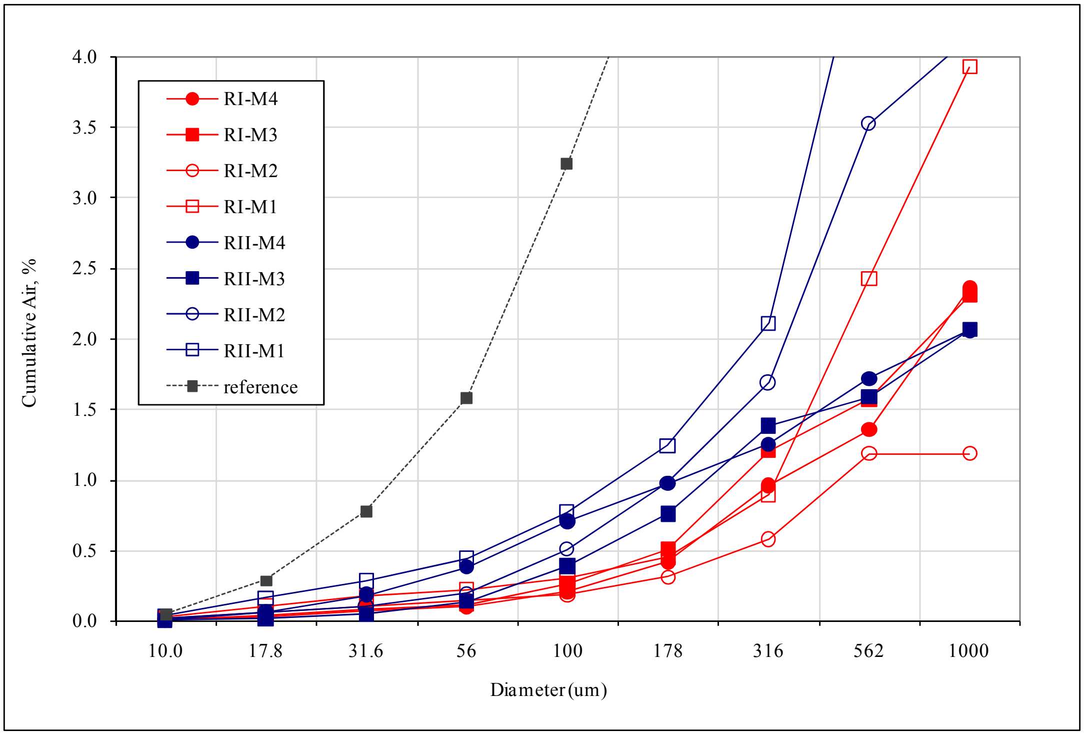

Figure 1. Cumulative air contents

SUMMARY AND CONCLUSIONS

The following conclusions can be drawn based on the limited scope of this study:

- Computer-assisted image analysis and operations like thresholding, segmentation, binary image formation, opening, closing, hole filling, erosion, etc. can be effectively utilized in identifying the air voids in concrete.
- The air-void system can be quantified using equivalent diameter, count, aspect ratio, etc., which help in analyzing the shape and area occupied by the air voids in the frame under consideration. Suitable calibration models are required to extend the results on a volumetric basis for concrete.
- For RCC mixes, compaction voids are forming a substantial portion of the total air content. Although no statistically valid conclusions can be drawn due to the limited nature of this study, it can be said that compaction voids are created irrespective of air entrainment in RCC mixes. Shape factors can be used in analyzing the shape of air voids.
- Further analysis is required in predicting the total air content of RCC mixes, since it is observed that there are air voids beyond 1 mm in diameter, which are difficult to capture using this system. The data obtained from automated image analysis may be utilized in fully predicting the air content of the RCC mixes.
- Air-entraining agent entrains some air and improves the shape factor (tending toward one), indicating that useful air can be entrained in such dry mixes. Further optimization studies are required in order to reach higher entrained-air contents and improve the F-T resistance of RCC mixes.

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The paper is a part of the first authors' MS thesis. The financial support received from the Portland Cement Association (PCA), USA; the Institute for Transportation (InTrans) at Iowa State University; and the Eisenhower fellowship from the National Highway Institute (NHI) during this study are warmly acknowledged. Special thanks to Dr. Warren Straszheim of the Materials Analysis and Research Laboratory (MARL) at Iowa State University for his insight into image analysis and critical discussions on this paper.

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Some Observations on Sorption-Desorption Behaviors of Roller-Compacted Concrete Mixtures

Chetan V. Hazaree
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
136 Town Engineering
Ames, IA 50011-3232
chazaree@iastate.edu

Kasthurirangan Gopalakrishnan
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
354 Town Engineering
Ames, IA 50011-3232
rangan@iastate.edu

Halil Ceylan
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
394 Town Engineering
Ames, IA 50011-3232
hceylan@iastate.edu

ABSTRACT

Water transport in concrete is a crucial phenomenon that to a great extent dictates the durability of concrete. The experimental program described in this paper compares the time rate of change of sorptivity (water uptake) and desorptivity (water loss) and the related coefficients for four distinct mixes with differing cement contents. All the mixes are roller-compacted concrete (RCC) mixes. Studies are being conducted on specially formed and preconditioned samples to simulate three distinct kinds of sorptivity behaviors in real-world circumstances. Different coating methods are also applied. The objective of this study is to appreciate the nature of water uptake and loss by capillary actions.

Over the studied cement content range, it has been observed that the sorptivity and desorptivity behaviors are strongly influenced by the cement content of the mixtures. At the same time, the nature and the direction of water uptake and loss depends on the sample preparation and preconditioning. Sorption and desorption isotherms are obtained to understand the water movements for 500 consecutive hours. This study indicates that the sorption and desorption behaviors could further be generalized by relating them with the desorptivity index, a relatively new index defined in this paper. Data obtained from individual mixes are further utilized for developing mathematical models to describe the water transport behavior. It is anticipated that successful completion of this study would answer several of the questions related to water transport in dry concretes.

Key words: desorption—pavements—roller-compacted concrete (RCC)—sorptivity

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INTRODUCTION

Roller-compacted concrete (RCC) is a stiff, dry concrete and a construction technology that utilizes roller compaction for densifying and finishing the concrete. Relatively lower cement and water contents, higher aggregate/cement ratios, the absence of suitable chemical admixtures, and the inherent nature of laying and compaction operations makes the final material relatively open structured. In addition to this, due to roller-compaction operation, there are irregular-shaped entrapped air voids left in the final structure of RCC mixtures. Drier paste makes it much more difficult to entrain useful air for enhancing the workability and/or freeze-thaw (F-T) resistance. Moreover, larger quantities of air-entraining admixtures are required for entraining even a small amount of air in these mixtures. All these factors combined together offer a final structure of RCC that renders easier pathways to water movement than the conventionally compacted and finished structural concrete.

In general, durable concrete will result if it has low water to cementitious ratio, has achieved adequate thermal and moisture curing, and has achieved a discontinuous capillary structure free of significant micro and macro defects (DeSouza et al. 1998). The mechanisms responsible for the deterioration of building materials are largely mediated and critically determined by the volume and rate of water movement. The conjugate action of mechanical and chemical damages may lead to rapid degradation of bulk mechanical properties of building materials and significantly reduce the service life of constructions. As such, good resistance to capillary absorption (unsaturated flow) of water (and other fluids) is of paramount importance. This is usually characterized by the property called as sorptivity, *S*, which expresses the tendency of a material to absorb and transmit water and other liquids by capillarity (Hall et al. 2002). If the relationship between normalized water uptake (I_A) and the square root of time is well represented by a straight line, the sorptivity, is usually the slope of the least square regression line of I_A against $t^{1/2}$ and is given by

$$S = \frac{(I_A - b)}{t^{1/2}} \quad , \qquad (1)$$

where *b* is a constant intercept on I axis due to several minor edge surface and ambient moisture effects.

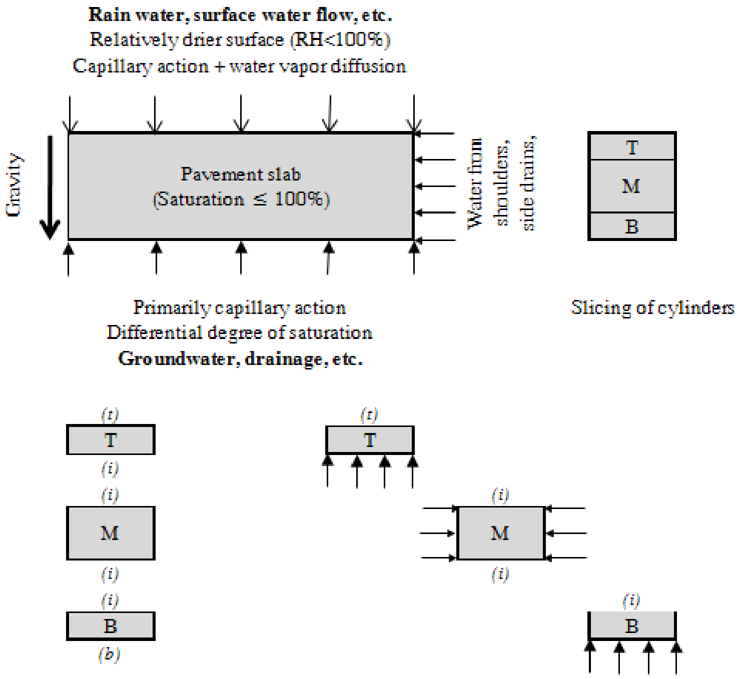
The penetration of water into the concrete via sorption and diffusion may have major implications on its F-T performance. Basheer et al. (1994) have observed good correlation between concrete performance in ASTM C 666 and sorptivities and air permeability. Moreover, sorptivity is seen to play a major role in the F-T scaling of field concrete (Bentz et al. 1999).

RESEARCH FOCUS AND MIXTURE PROPORTIONS

RCC is being applied for a variety of pavement applications, including pavement bases, shoulders, wearing course, in composite pavements, etc. Binder type, binder content, mixture composition, and construction practices vary according to the structural role of RCC layer and the geography of application. A varied range of applications are required for a comprehensive quantification and analyses of various properties. A broad study was thus undertaken for appreciating some of the influences of cement content, surface finish, type of preconditioning, and age of concrete.

METHODS

The sorptivity samples were drawn out of the curing room at the age of 14 days to simulate field conditions of curing. Preconditioning was done by keeping these samples in oven at 50 ± 2ºC for 3 days and then in sealed containers for next 21 days. To measure relative amount of drying of specimens, equivalent specimens were kept in the oven for 24 days at 50 ± 2ºC. This oven drying was used as reference drying. The densities obtained from these specimens were then utilized in calculating the desorption ratios. Specimens T and B (as shown in Figure 1) were coated with a concrete sealer on the curved surfaces, while the top and bottom sections were left uncoated. Coating was done after preconditioning was completed so as to ensure uniform and effective drying and for obtaining close to real-world water uptake. Subsequently, one-dimensional capillary absorption (1-DCA) was measured for all the mixes in accordance with ASTM C 1585. Quadruplicate samples were used for measuring the mass changes due to water absorption and subsequent evolution of water transport profiles.


Rain water, surface water flow, etc.
Relatively drier surface (RH<100%)
Capillary action + water vapor diffusion
Gravity
Pavement slab
(Saturation ≤ 100%)
Water from shoulders, side drains,
T
M
B
Primarily capillary action
Differential degree of saturation
Groundwater, drainage, etc.
Slicing of cylinders
(t)
T
(i)
(i)
M
(i)
(i)
B
(b)
(t)
T
(i)
M
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B

Figure 1. Pavement simulation for water movement samples

RESULTS

Typical results are shown in Figures 2 and 3. Similar results were obtained for other mixtures and analyses were performed.

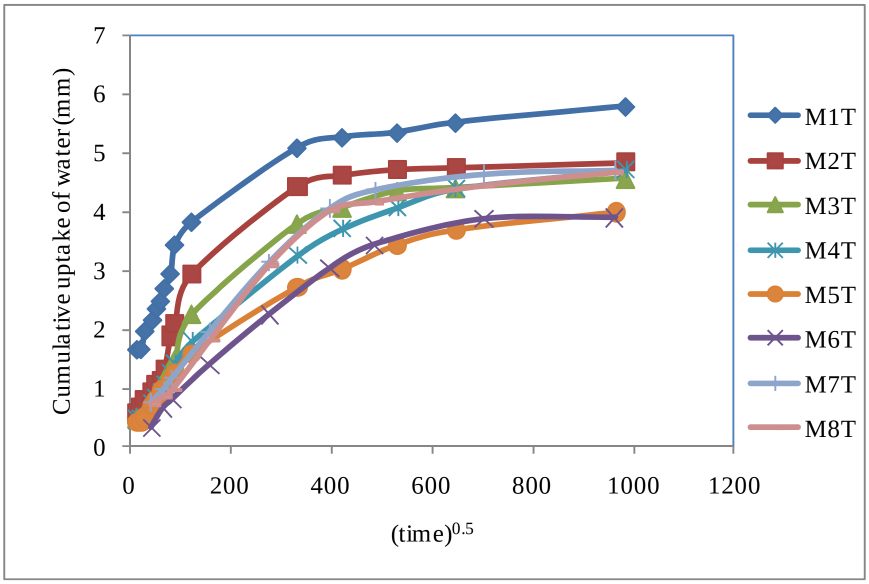

Figure 2. Sorption profiles for different mixtures

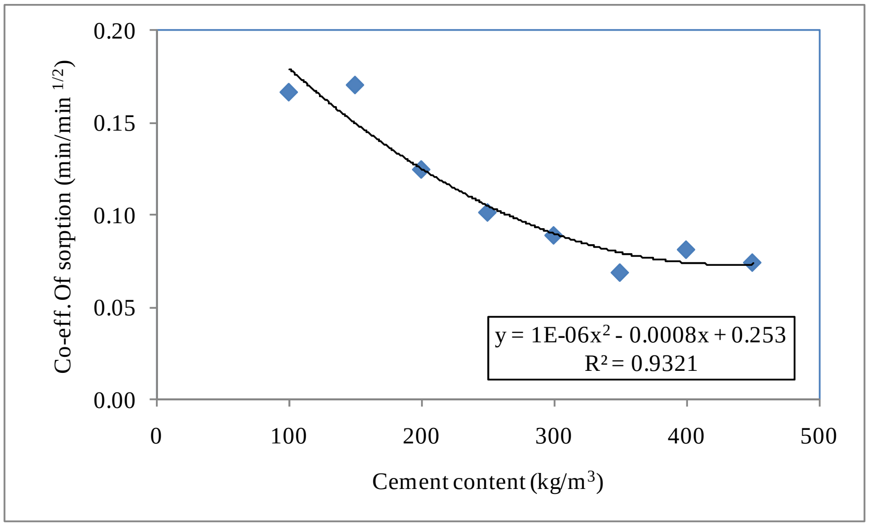

Figure 3. Variation of coefficient of sorption for different mixtures

SUMMARY AND CONCLUSIONS

The following conclusions can be drawn based on the limited scope of this study:

- This comprehensive study leads to the generation of sorptivity database for RCC.
- In general, the coefficient of sorption decreases with the increase in the cement content. The optimum values are obtained for cement contents ranging between 300 ± 50 kg/m^3.
- The desorptivity values also follow similar trends.
- Combined picture obtained from these two sets of values could help in appreciating the durability performance of concrete mixtures.

ACKNOWLEDGMENTS

The paper is a part of the first authors' MS thesis. The financial support received from the Portland Cement Association (PCA), USA; the Institute for Transportation (InTrans) at Iowa State University, and the Eisenhower fellowship from the National Highway Institute (NHI) during this study are warmly acknowledged. Special thanks to Dr. Warren Straszheim of the Materials Analysis and Research Laboratory at Iowa State University for his insight into image analysis and critical discussions on this paper.

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Monitoring of the Saylorville and Red Rock Reservoir Bridges for Wind-Induced Vibrations

Travis Hosteng
Bridge Engineering Center
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010
kickhos@iastate.edu

Bruce Brakke
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Bruce.Brakke@dot.iowa.gov

Doug Wood
Structures Laboratory
Iowa State University
136A Town Engineering
Ames, IA 50011
dwoody@iastate.edu

Brent Phares
Bridge Engineering Center
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010
bphares@iastate.edu

Ahmad Abu-Hawash
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Ahmad.Abu-Hawash@dot.iowa.gov

EXTENDED ABSTRACT

During a high-wind event in January of 2006, several motorists reported vertical movements of the bridge superstructure over the Saylorville Reservoir. On February 2006, an inspection of the bridge was conducted by representatives from the Iowa Department of Transportation (Iowa DOT) and Iowa State University. During that inspection, no signs of excessive bridge movement could be identified. Areas of specific interest during that inspection included cross-bracing, stiffener-to-web welds, and expansion joints. The Iowa DOT subsequently performed a preliminary literature investigation to determine if any similar event might have occurred in the past, and if so, if any technical information was available. The Iowa DOT determined that implementing a monitoring project on the Saylorville Bridge to better assess the potential for a significant wind-induced event to occur was needed. Furthermore, the monitoring project would alert the Iowa DOT of wind events over a preset threshold. After the monitoring system had been in place and operating on the Saylorville Bridge, the Iowa DOT decided to develop a similar monitoring project on the Red Rock Bridge as well.

The Saylorville Bridge is composed of five main sections separated by expansion joints. Each of these five sections is composed of five spans, consisting of two 168 ft end spans and three 216 ft interior spans; the total length of the structure is 4,920 ft. The superstructure is composed of four steel girders spaced at 9 ft 4 in. and a concrete deck. Rather than monitoring the entire structure, the research team selected one span within one of the five sections to install the monitoring system. The team first decided to use an interior main section, in this case, the first interior main section from the southern end of the bridge. Within this main section, it was again deduced that an interior span was desirable, so the first interior span

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from the northern end of this section was selected so as to be closest to the center of the overall bridge as possible. See Figure 1 for the location of the instrumented span.

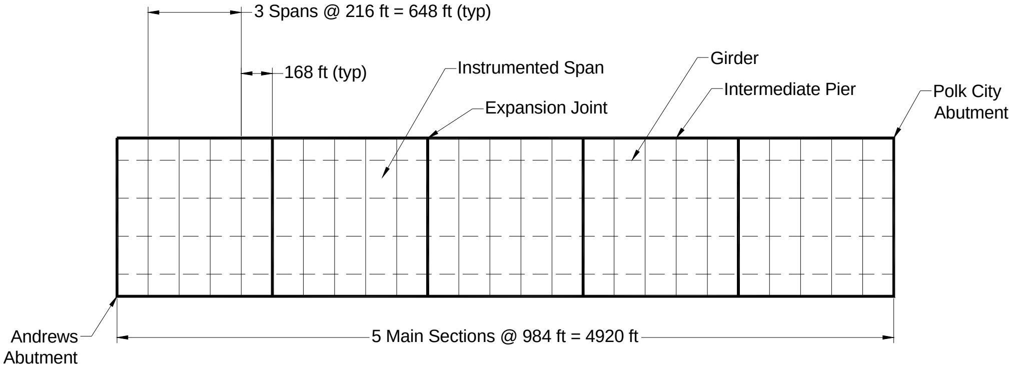

3 Spans @ 216 ft = 648 ft (typ)
168 ft (typ)
Instrumented Span
Expansion Joint
Girder
Intermediate Pier
Polk City Abutment
Andrews Abutment
5 Main Sections @ 984 ft = 4920 ft

Figure 1. Saylorville Bridge plan layout showing instrumented span location

The monitoring system developed for the Saylorville Bridge included strain gages, accelerometers, an anemometer, a data logger, a wireless modem, and a wireless cell phone account. One strain gage was mounted on the bottom flange of each of the four girders of the selected span at mid span. In addition, an accelerometer was installed at mid span and quarter span of one interior girder and mid span of one exterior girder. At the north pier of the selected span, a weatherproof box was mounted on the top of the pier cap to house the datalogger, wireless modem, and battery pack. In addition, a mount was created for the anemometer that attached to the guardrail near the pier. The mount offset the anemometer approximately 5 ft from the guardrail and approximately 20 ft above the roadway (ideal would be 30 ft). Power for the system is obtained from a 10 watt solar panel mounted on the southeastern side of the pier cap. See Figure 2 for photos of the solar panel and weatherproof box.

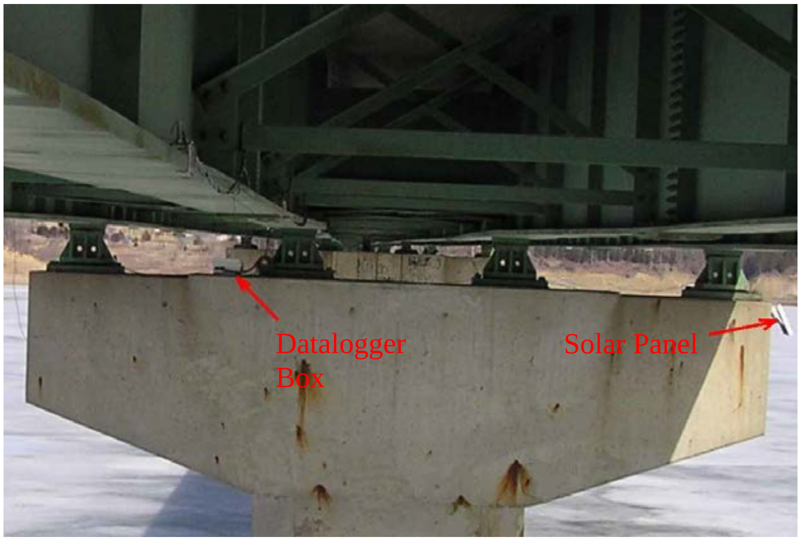
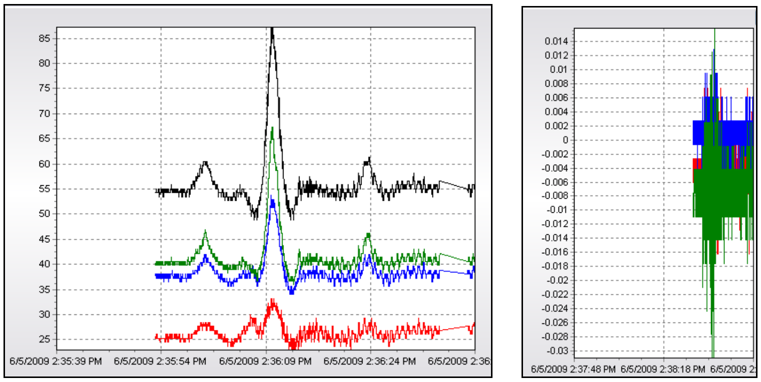

Datalogger Box
Solar Panel

Figure 2. Solar panel and datalogger box

Communication with the system is achieved via an unrestricted static IP address on the Verizon Wireless Network using the cellular modem on site. Using a properly set up computer with internet service, the user can log onto the data logger and view data (strain, accelerations, wind speed, direction, battery power, etc.) in either a graphical or tabular format (see Figure 3 for a typical plot). In addition, preset triggers have been programmed into the system to provide warnings to the appropriate personnel. At the moment, the trigger is set to go off at a wind speed greater than 50 mph from any direction. If the trigger criterion is met, the system sends a text message ‘(Bridge Alarm) Saylorville Bridge System Alert. Trigger Wind Speed is ##.### MPH. Logger time is Date, Time’ to Iowa DOT and Bridge Engineering Center personnel who alert the appropriate personnel to handle the situation. Currently, when a trigger alert is received, Iowa DOT officials respond and close off the bridge to traffic. The trigger interval is set to 20 min, so that as long as triggers are being received, the bridge remains closed. Once 30 min has passed since the last trigger and the officials on site judge that winds have diminished to a safe level, the bridge is reopened.



a. Bottom flange strains **b. Accelerations**

Figure 3. Typical plots of live strain and accelerations from Saylorville Bridge datalogger

Prior to this final trigger setting and response criterion, the trigger level was set at a lower level (25 mph) so that the system would be more frequently triggered and could be adequately tested for functionality. Once the reliability of the system had been proven, final adjustments were made to the current settings. To date, there have been two instances of a bridge closure as a result of wind speeds exceeding the 50 mph trigger threshold. The first instance lasted for several hours and produced peak wind speeds of approximately 68 mph; the second instance was shorter but still produced peak wind speeds of approximately 60 mph.

Currently, a similar system is being set up for installation at the Red Rock Reservoir Bridge. The system will include all the same hardware and software and function the same as the system on the Saylorville Bridge.

Key words: bridge—long-term structural monitoring—structural health monitoring—wind loading—wind-induced vibration

Analysis of Traffic Sign Retroreflectivity in National Wildlife Refuges in the Midwest

Mark Hurt
Bridge Design
Kansas Department of Transportation
State Bridge Office
Eisenhower State Office Building
700 S.W. Harrison St.
Topeka, KS 66603
hurt@ksdot.org

Steven D. Schrock
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
2150 Learned Hall, 1530 W. 15th St.
Lawrence, KS 66045
schrock@ku.edu

Yilei Huang
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
2150 Learned Hall, 1530 W. 15th St.
Lawrence, KS 66045
hyl0510@ku.edu

ABSTRACT

In December 2007, the second revision of the 2003 *Manual on Uniform Traffic Control Devices* (MUTCD) introduced minimum retroreflectivity levels for traffic signs. Public agencies have until January 2012 to have in place sign assessment or management plans, and all regulatory, warning, and ground mounted signs must comply with the new levels by January 2015. In this study, traffic sign data were obtained from 104 national wildlife refuges in 11 states in the Midwest. The type, color, face, size, wording, sheeting type, visibility, retroreflectivity level, MUTCD designation, sign condition, age, and estimated cost of repair/ replacement of all the signs were analyzed systematically. Using estimates of sign life and replacement costs based on data in the current literature, the annual cost to obtain and maintain compliance with the new retroreflectivity requirements was calculated. The results will show the total cost for each national wildlife refuge in the Midwest to maintain the minimum retroreflectivity levels for traffic signs according to the new requirements and the average cost of a unit quantity of signs. Suggestions on methods of implementation of maintenance are given to the public agencies based on the comparison of cost estimation.

Key words: MUTCD—national wildlife refuge—retroreflectivity—traffic sign

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INTRODUCTION

The second revision of the 2003 *Manual on Uniform Traffic Control Devices* (MUTCD) introduced a new standard in Section 2A.09 for minimum values of retroreflectivity for traffic signs. Public agencies are required to maintain the minimum values listed in the new Table 2A-3, Minimum Maintained Retroreflectivity Levels (Figure 1) (1). With the countdown for compliance beginning with the adoption of the Final Rule for Revision 2 in December 2007, agencies are required to have a sign assessment or management method for maintenance of retroreflectivity in place on January 2012; to have regulatory, warning and ground-mounted signs in compliance by January 2015; and to have overhead guide and street signs in compliance by January 2018 (2).

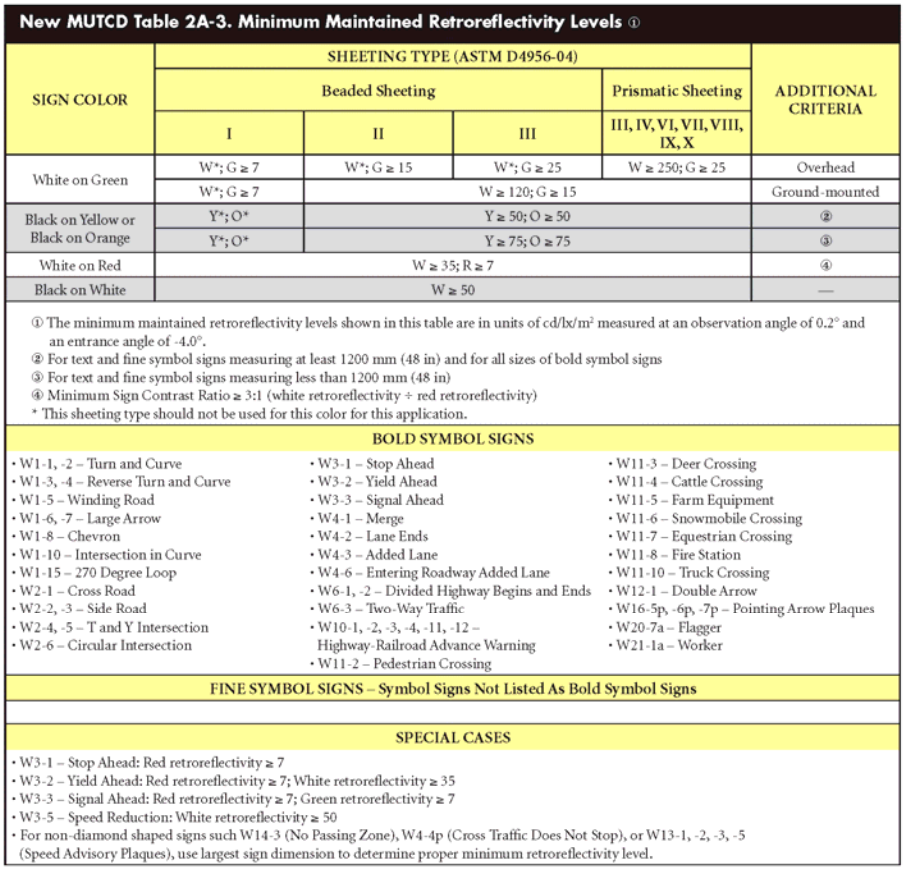
Compliance with these new requirements will require assessing and updating noncompliant signs in existing inventories, a mandate that may pose an unwelcome cost on public agencies facing fiscal challenges in the current economic environment. Additionally, new annual costs may be incurred from the required on-going assessment and maintenance of retroreflectivity values. These impacts have been addressed in some of the current literature with estimates for impacts at the national level (3) (4) and at the state level, particularly in North Carolina (5). In this report, current information on sign deterioration and replacement rate and sign inspection, maintenance, and replacement costs are applied to the recently surveyed sign inventories on parks held by the Bureau of Land Management in seven Midwestern states to obtain the cost impact of implementation of this mandate.

BACKGROUND

Although the establishment of specific required minimum retroreflectivity values for traffic signs is a recent development, the use of retroreflective material for such is not. According to literature from researchers at 3M, the use of “luminous or reflecting elements” for standard traffic signs and markers was first approved by the American Association of State Highway and Transportation Officials (AASHTO) in 1928 (6). The first installation of a traffic sign using retroreflective sheeting in the United State was in 1938 in Minneapolis (7). These first retroreflective sheets had exposed glass beads which failed to reflect in rainfall conditions. This led to development of enclosed lens materials, resulting in production of engineering grade sheeting by 3M in the late 1940s, with commercialization in the early 1950s (6).

Retroreflective backgrounds were shown to improve the visibility of signs at night in studies for signs for the then-new Interstate system (7). The detection distance was four times greater for signs with reflectorized backgrounds than for painted backgrounds. Work in the 1960s tested effectiveness of reflective green backgrounds, with driver error being the least at the brightest signs (8).

Much of the concern about nighttime visibility is centered about the older driver. In 1967, Maryland (9) reported that the light required for good vision doubles every 13 years after the age of 20. Other researchers document reductions in acuity (10) and visual field (11). Graham et al. (12) tested drivers less than 25 years of age and over 65 years of age to determine sign luminance (which is directly correlated to retroreflectivity) required to correctly discern sign legends. At 60 m, older subjects required more than 4.5 times the luminance than did the younger drivers to correctly identify the legends 100% of the time.



New MUTCD Table 2A-3. Minimum Maintained Retroreflectivity Levels ①

SIGN COLOR	SHEETING TYPE (ASTM D4956-04)				ADDITIONAL CRITERIA
	Beaded Sheeting			Prismatic Sheeting	
	I	II	III	III, IV, VI, VII, VIII, IX, X	
White on Green	W*; G ≥ 7	W*; G ≥ 15	W*; G ≥ 25	W ≥ 250; G ≥ 25	Overhead
	W*; G ≥ 7	W ≥ 120; G ≥ 15			Ground-mounted
Black on Yellow or Black on Orange	Y*; O*	Y ≥ 50; O ≥ 50			②
	Y*; O*	Y ≥ 75; O ≥ 75			③
White on Red	W ≥ 35; R ≥ 7				④
Black on White	W ≥ 50				—

① The minimum maintained retroreflectivity levels shown in this table are in units of cd/lx/m² measured at an observation angle of 0.2° and an entrance angle of -4.0°.
② For text and fine symbol signs measuring at least 1200 mm (48 in) and for all sizes of bold symbol signs
③ For text and fine symbol signs measuring less than 1200 mm (48 in)
④ Minimum Sign Contrast Ratio ≥ 3:1 (white retroreflectivity ÷ red retroreflectivity)
* This sheeting type should not be used for this color for this application.

BOLD SYMBOL SIGNS

- W1-1, -2 – Turn and Curve
- W1-3, -4 – Reverse Turn and Curve
- W1-5 – Winding Road
- W1-6, -7 – Large Arrow
- W1-8 – Chevron
- W1-10 – Intersection in Curve
- W1-15 – 270 Degree Loop
- W2-1 – Cross Road
- W2-2, -3 – Side Road
- W2-4, -5 – T and Y Intersection
- W2-6 – Circular Intersection
- W3-1 – Stop Ahead
- W3-2 – Yield Ahead
- W3-3 – Signal Ahead
- W4-1 – Merge
- W4-2 – Lane Ends
- W4-3 – Added Lane
- W4-6 – Entering Roadway Added Lane
- W6-1, -2 – Divided Highway Begins and Ends
- W6-3 – Two-Way Traffic
- W10-1, -2, -3, -4, -11, -12 – Highway-Railroad Advance Warning
- W11-2 – Pedestrian Crossing
- W11-3 – Deer Crossing
- W11-4 – Cattle Crossing
- W11-5 – Farm Equipment
- W11-6 – Snowmobile Crossing
- W11-7 – Equestrian Crossing
- W11-8 – Fire Station
- W11-10 – Truck Crossing
- W12-1 – Double Arrow
- W16-5p, -6p, -7p – Pointing Arrow Plaques
- W20-7a – Flagger
- W21-1a – Worker

FINE SYMBOL SIGNS – Symbol Signs Not Listed As Bold Symbol Signs

SPECIAL CASES

- W3-1 – Stop Ahead: Red retroreflectivity ≥ 7
- W3-2 – Yield Ahead: Red retroreflectivity ≥ 7; White retroreflectivity ≥ 35
- W3-3 – Signal Ahead: Red retroreflectivity ≥ 7; Green retroreflectivity ≥ 7
- W3-5 – Speed Reduction: White retroreflectivity ≥ 50
- For non-diamond shaped signs such W14-3 (No Passing Zone), W4-4p (Cross Traffic Does Not Stop), or W13-1, -2, -3, -5 (Speed Advisory Plaques), use largest sign dimension to determine proper minimum retroreflectivity level.

Figure 1 New Minimum Maintained Retroreflectivity Levels for Traffic Signs (from Ref. 2)

Figure 1. New minimum maintained retroreflectivity levels for traffic signs (FHWA 2009)

In discussion of the background behind the recent development of minimum retroreflectivity standards (13), Carlson et al. ascribe much of the impetus to a petition for such made by the Center for Auto Safety (CAS) in 1984 to the FHWA (14). At that time, retroreflectivity requirements were part of General Service Administration and FHWA specifications, but were only required on projects under direct FHWA supervision (14). Neither states nor local entities were required to include this or any similar specification in their projects. The same consideration for the needs of an aging population that were addressed by the CAS petition led Congress, through the U.S. Department of Transportation and Related Agencies Appropriation Act of 1992, to direct the Secretary of Transportation to develop “a standard for a minimum level of retroreflectivity that must be maintained for pavement markings and signs which apply to all road open to public travel.” In 1993 an initial set of minimum retroreflectivity levels were introduced (15). These were updated in 1998 (16) and in 2003 (17), with the latter forming the basis for the current standards.

CURRENT LITERATURE

With the establishment of minimum retroreflectivity levels and the identification by the FHWA of assessment and management methods (18) that will comply with the new Section 2A.09 of the MUTCD, the current literature focuses on providing data which will allow agencies to implement these methods and assess their costs. Studies of the cost impact due to the implementation of retroreflectivity minimum made on states and nationally are summarized in an April 2007 FHWA publication (4). The publication provides an updated National Impact Assessment of cost at the state and local levels. The assessment assumes approximately 10% replacement for regulatory, warning, and guide signs; 20% replacement for streets signs; and replacement of legends (but not backgrounds) on all overhead guide signs, with a resulting cost of $11.8 million to the states and $25.7 million to the local agencies.

Much of the current literature comes from North Carolina. Harris et al. evaluated 30 sign asset management scenarios using the North Carolina Department of Transportation (NCDOT) sign inventory to determine the cost per sign and the expected percentage of non-compliant (from the new FHWA minimums) signs resulting from each scenario (19). The scenarios varied in maintenance strategy (nighttime visual inspection, measured retroreflectivity values, expected sign life or blanket replacement), retroreflectivity rejection threshold (varied incrementally from the current NCDOT values to the new FHWA values), conversion rate of Type I sheeting to Type III (either the 89% representative of current NCDOT inventory, 100% or 0%) and inspection frequency (either the NCDOT current 2.64 years, 1, 2 or 3 years). The cheapest annual per sign cost was $3.28 for a three year inspection frequency using nighttime visual inspections with NCDOT's current standards for retroreflectivity; however, this resulted in 23.2% of the signs being non-compliant with the new standards. Current NCDOT practice resulted in 20.8% non-compliance for $3.43 per sign per year.

Replacing signs at the end of their expected life resulted in 100% compliance for $5.09 per sign per year. Based on a previous deterioration study (20), lives of 17 years were used for all type III signs and lives of 12, 3, 5, and 11 years for white, yellow, red, and green, respectively, type I signs.

In 2009 a study of existing data on sign deterioration rates was done (21). Eight models (for the four different sign color and for type I and III sheeting) from the existing literature were chosen as best available predictors for useful life. The expected life until the retroreflectivity values reach FHWA minimums is given in Table 1.

Table 1. Age at which sheeting is predicted to degrade to minimum required retroreflectivity

Sign Color	Sign Type	Age at FHWA Minimum (years)
White	I	13 (R_a = 35) 10 (R_a = 50)
	III	56 (R_a = 35) 53 (R_a = 50)
Yellow	I	7
	III	22
Red	I	10
	III	20
Green	I	14
	III	35

Data Collection

Data for this study were obtained from Utah State University, which include a road inventory report and a Microsoft Access database for each of the 65 national wildlife refuges in 11 Midwest states (Illinois, Indiana, Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, North Dakota, Ohio, and South Dakota).

The road inventory reports contain information about support condition, sign condition, visibility, and estimated cost of repair/replacement. The support condition and sign condition determine whether they have to be repaired or replaced. Signs need to be replaced that have incorrect sheeting type, low retroreflectivity measurements, do not have retroreflectivity value required to have, or are in an extremely poor condition or missing. Signs need to be repaired that are in a poor condition or are obscured by a tree, bush, or curve. Supports are to be repaired if in poor condition or replaced if in extremely poor condition. The estimated cost of each sign or support is $100 for repair and $150 for replacement.

The databases contain information about sign type, text, color, background retroreflectivity value, text retroreflectivity value, the ratio of the two values, and retroreflectivity pass/fail. Retroreflectivity pass/fail has “Pass,” “Fail,” and “Not Rated” options, in which “Not Rated” means the sign is not required to have retroreflectivity and the according values are zero and the color is marked as “Other.” Sign type is according to MUTCD Part 2 Signs with a total number of 897.

Data Analysis

Step 1: Determine the Number of All Signs and Non-Compliant Signs in Each State and as a Whole

In the 65 national wildlife refuges, there are 3,861 signs in total ,and the number of signs in each state varies from 33 in Ohio, in which only one refuge has been studied, to 818 in Minnesota, which has 14 studied refuges.

Signs with retroreflectivity have been rated according to the MUTCD standard and are marked as “Pass” or “Fail.” Signs with no retroreflectivity are marked as Not Rated. In the new MUTCD sign retroreflectivity requirements, the following five categories of signs are excluded from the requirements:

- Parking, standing, and stopping signs (R7 and R8 series)
- Walking/Hitchhiking/Crossing signs (R9 series, R10-1 through R10-4b)
- Adopt-A-Highway signs
- All signs with blue or brown backgrounds
- Bikeway signs that are intended for exclusive use by bicyclists or pedestrians

Therefore, signs falling into these five categories are not required to have retroreflectivity. There are also a small portion of signs leaving blank in retroreflectivity pass/fail, which are considered having no retroreflectivity and failing in the requirements.

Table 2 shows that the overall pass rate is around 75% with the peak of 84.12% in Indiana and the bottom of 54.54% in Ohio, mostly due to lack of data. The second lowest pass rate is in Kansas, which has the most signs with retroreflectivity failed in the 11 states. Other states have their pass rates above 70%. What should be noted is that the eligible unrated signs are nearly three times more than those with retroreflectivity, largely because in these national wildlife refuges there are much more guide signs and recreational signs than regulatory signs and warning signs.

Table 2. Sign condition in each state and as a whole

State	Number of Refuges	Number of Signs	Rated		Unrated		Pass (Percent)
			Pass	Fail	Pass	Fail	
Illinois	5	441	77	72	232	60	70.06
Indiana	2	189	40	21	119	9	84.12
Iowa	6	533	105	81	303	44	76.54
Kansas	3	443	35	133	246	29	63.43
Michigan	3	145	30	14	85	16	79.31
Minnesota	14	818	99	102	557	60	80.19
Missouri	5	285	38	55	174	18	74.38
Nebraska	6	281	33	22	199	27	82.56
North Dakota	14	505	104	98	277	26	75.44
Ohio	1	33	0	0	18	15	54.54
South Dakota	6	188	30	20	120	18	79.78
Overall	65	3,861	591	618	2,330	322	75.65

Step 2: Determine the Estimated Cost of Sign Repair and Replacement in Each State and as a Whole

Signs that are required to be retroreflective must be replaced if the sheeting type is incorrect or if retroreflectivity measurements are insufficient before the Section 2A.09 deadlines. Signs without retroreflectivity need to be replaced only if they are in extremely poor condition or are missing. The refuge studies also provided estimated cost for repair and replacement. The estimated cost for a sign replacement is $150. All signs are to be repaired if they are in a poor condition or obscured by tree, bush, building, hill, or curve. The estimated cost to repair a sign in poor condition is $100 and to remove the obstacle is $50. For supports, the estimated cost to replace is $150 and to repair is $100.

In this study, the repair and replacement of signs with no retroreflectivity are included as “Other Reasons,” as seen in Table 3, which include “Poor Condition,” “Replace Condition,” “Missing,” and “Obscured.” Although the total number of signs with retroreflectivity is less than one-half of that with no retroreflectivity, signs with retroreflectivity requiring repair or replacement are nearly twice as much as those without retroreflectivity needing repair or replacement. Supports condition is relatively much better than signs.

Kansas has the largest amount of estimated cost, largely because the number of signs with incorrect sheeting type is greater, which accounts for one-fourth of that in all the 11 states. It also accounts for one-fourth of the number of total signs in Kansas. Looking back into the database in detail, it can be found that of the three national wildlife refuges in Kansas, Flint Hills National Wildlife Refuge has a sign pass rate of only 36.5% and a rated sign pass rate of less than 20%. This may help explain why Kansas has the most signs with retroreflectivity failed and the most estimated cost. The next most estimated cost states are Minnesota, North Dakota, and Iowa, partly because they have the largest amount of signs in the 11 states. Ohio, Michigan, Indiana, and South Dakota, which have the fewest signs, also have the least estimated cost.

In terms of the percentage of sign repair/replace due to retroreflectivity reasons, the average rate is 64.91%, which means the cost to replace signs failing the new retroreflectivity standard accounts for the majority of the total repair/replacement cost. Excluding the state of Ohio, the lowest percentage is in Indiana with a little below 50%, followed by Michigan and Nebraska. Kansas and North Dakota are the only two states that overpass the average rate, again because of the big amount of incorrect sheeting signs.

Table 3. Estimated cost of sign repair/replace in each state

State	Number of Signs	Sign Repair/Replace			Support Repair/ Replace	Cost ($)	Retro Cost as Percent of Total Repair/Replace
		Incorrect Sheeting	Low Retro-Reflectivity	Other Reasons			
Illinois	441	25	45	42	10	16,100	62.50
Indiana	189	14	7	23	1	5,250	47.72
Iowa	533	46	31	46	4	17,350	62.60
Kansas	443	109	24	21	2	22,150	86.36
Michigan	145	8	4	12	2	3,000	50.00
Minnesota	818	65	35	61	15	21,950	62.11
Missouri	285	47	8	37	13	13,800	59.78
Nebraska	281	14	8	21	4	6,050	51.16
North Dakota	505	68	29	45	14	20,050	68.30
Ohio	33	0	0	3	0	250	0
South Dakota	188	15	5	17	6	5,300	54.05
Overall	3,861	411	196	328	71	131,250	64.91

Step 3: Determine the Annual Cost of Signs with Retroreflectivity Based on Expected Sign Life and Discount Rates

In the study of Venkata P. K. Immaneni et al. (21), eight models (one for each combination of the four different sign colors and for Types I and III sheetings) were chosen as best available predictors for useful life. The expected life until the retroreflectivity values reach FHWA minimums is given in Table 1. Table 4 shows the total number of signs in each of the above colors in each state. Sign type is not included in the national wildlife refuge database.

Table 4. Number of signs in each color

State	Number of Signs	Number of Retro-Reflective Signs	Sign Color (Background)			
			Yellow/Orange	White	Green	Red
Illinois	441	149	27	82	0	40
Indiana	189	61	29	20	0	12
Iowa	533	186	70	59	6	51
Kansas	443	168	109	24	0	35
Michigan	145	44	8	28	0	8
Minnesota	818	201	91	47	1	62
Missouri	285	93	47	26	0	20
Nebraska	281	55	9	19	0	27
North Dakota	505	202	87	73	3	39
Ohio	33	0	0	0	0	0
South Dakota	188	50	30	8	0	12
Overall	3,861	1,209	507	386	10	306

Note in the above table that white signs make up approximately 30%of the total inventory of signs. In Table 1, the expected sign life for white sheeting (of either Type I or III) is significantly longer than that for the other three colors. In the same report, its authors counseled against completely trusting the predicted life for white sheeting as that the R^2 values for its models were relatively low and counseled that there was no data on the Type III sheeting over 15 years in age.

In other works (19) (22), NCDOT used a 15-year lifetime for Type III sheeting and an earlier Minnesota Department of Transportation (Mn/DOT) study used 14 years, both regardless of color. In these same works, it was noted that the agencies are proceeding to using only Type III sheeting for sign replacement, on the basis of previous studies of the cost effectiveness of using Type I vs. Type III. The expected longer life of Type III sheeting led to less replacement cost, overcoming the initial higher sheeting costs (sheeting material cost is a relatively small part of sign replacement cost), and resulting in lower lifetime costs.

Conversations with refuge staff and with a former Federal Lands engineer indicate that there is not an ongoing program of sign maintenance in the refuges due to a lack of staff. Signs are replaced only as they are discovered missing or significantly damaged. Given that there are no current dedicated sign maintenance activities on the refuges, it is difficult to estimate costs for implementing sign retroreflectivity inspections. So for this study, the cost of blanket replacement of retroreflective signs using a Type III lifetime of 15 years was calculated.

To calculate equal annual costs, a discount rate must be assumed. According to Office of Management and Budget Circular A-94 Appendix C, revised December 2008, the nominal discount rate to use for federal projects is 4.7% for 20 year analysis and 4.5% for 30 year analysis. The discount rate used is 4.7% for the 15 years.

The annual cost for signs with retroreflectivity can be found by $[150-150\times(1\text{-}4.7\text{ percent})^{15}]$ /15=$5.14 per sign. By multiplying this cost with the total number of signs with retroreflectivity in each state, we can get an annual cost for replacing these signs at the end of their retroreflectivity service life, as shown in Table 5. North Dakota and Minnesota have the most retroreflectivity signs and have the annual cost above $1,000, followed by Iowa, Kansas, and Illinois. The total annual cost is relatively related to the total number of signs in each state.

Table 5. Annual cost for replacing signs at the end of retroreflectivity service life

State	Number of Signs	Number of Retro-Reflective Signs	Annual Cost ($)
Illinois	441	149	765.86
Indiana	189	61	313.54
Iowa	533	186	956.04
Kansas	443	168	863.52
Michigan	145	44	226.16
Minnesota	818	201	1,033.14
Missouri	285	93	478.02
Nebraska	281	55	282.70
North Dakota	505	202	1,038.28
Ohio	33	0	0.00
South Dakota	188	50	257.00
Overall	3,861	1,209	6,214.26

Step 4: Comparison of the Expected Annual Cost of Replacing Signs at the End of Retroreflective Service Life and Replacing Only for Damage as Current Practice

According to research for the NCDOT (23), the typical annual sign replacement rate for damage and vandalism is about 2.5%. The estimated cost of sign replacement for damage and vandalism is shown in Table 6. From the comparison, the annual cost of replacing retroreflective signs at the end of their service lives is almost double that of replacing only for damage or vandalism on the overall basis. Note, though, that given the number of signs currently requiring repair or replacement, performing such maintenance every year would require an increase from the current maintenance activity level.

For individual states, although Michigan has the lowest annual cost on both (Ohio excluded), the cost of replacing for retroreflectivity is two times more than that for damage or vandalism. Half of the 11 states have the ratio between 180% and 220%, and the average for all refuges studied is 189.4%, indicating that the cost to meet MUTCD retroreflectivity requirements will be doubled that compared with replacing for damage or vandalism as the targeted current practice.

Table 6. Comparison of annual cost of replacing damaged signs vs. expected cost to maintain retroreflectivity

State	Number of Signs	Signs Now Requiring Repair/ Replace	Cost ($)	Annual Replacement due to Damage/ Vandalism Cost ($)	Retro-reflectivity Maintenance Annual Cost ($)	Retro Maint/ Damage Replacement Annual Cost Ratio (%)
Illinois	441	112	16,100	402.50	765.86	190.27
Indiana	189	44	5,250	131.25	313.54	238.88
Iowa	533	123	17,350	433.75	956.04	220.41
Kansas	443	154	22,150	553.75	863.52	155.94
Michigan	145	24	3,000	75.00	226.16	301.54
Minnesota	818	161	21,950	548.75	1,033.14	188.27
Missouri	285	92	13,800	345.00	478.02	138.55
Nebraska	281	43	6,050	151.25	282.70	186.90
North Dakota	505	142	20,050	501.25	1,038.28	207.13
Ohio	33	3	250	6.25	0.00	-
South Dakota	188	37	5,300	132.50	257.00	193.96
Overall	3,861	935	131,250	3,281.25	6,214.26	189.38

Step 5: Comparison of the Expected Annual Cost from National Wildlife Refuge Databases and State DOTs' Studies

In the national wildlife refuge databases, the estimated cost is $150 for a sign replacement and $100 for a sign repair, which is the basis of all the above data analysis. However, in the state DOTs' studies, estimated cost is found to be up to $75 for a sign replacement, less than half of the refuge's replacement cost. The refuge's estimated cost may probably come from the FHWA impact report, which assumed sign replacements cost between $100 and $200 per sign. The other reason of the big difference might be the inspection and maintenance fee of the national wildlife refuges. Maintenance staff and activities are relatively much fewer than those in state DOTs, which will lead to a tremendous increase of inspection and maintenance fee. Table 7 shows the difference between the estimated annual costs made from national wildlife refuge databases, from small staffs, and state DOTs' studies, made by large staffs.

Table 7. Comparison of annual cost from refuge data and state DOTs' costs

State	Number of Signs	Refuge Annual Cost	Difference
Illinois	441	765.86	382.93
Indiana	189	313.54	156.77
Iowa	533	956.04	478.02
Kansas	443	863.52	431.76
Michigan	145	226.16	113.08
Minnesota	818	1,033.14	516.57
Missouri	285	478.02	239.01
Nebraska	281	282.70	141.35
North Dakota	505	1,038.28	519.14
Ohio	33	0.00	0.00
South Dakota	188	257.00	128.50

CONCLUSION

Though the costs to maintain new retroreflectivity requirements may seem high in terms of current practice, the absolute cost is relatively low. Using maintenance costs twice that of state DOT's, the cost to bring the entire inventory of signs on the wildlife refuges in 11 states up to full repair and to the new retroreflective requirements is only $131,250; and the approximate annual cost to replace damaged signs is $3,300 and to maintain retroreflectivity standards is $6,200.

The real challenge to implement of the new retroreflectivity standards on the refuges will be the relative increase in maintenance efforts required. A solution may be for the refuges to partner with the DOTs in their respective states to provide sign maintenance. A model might be found in the relationship between the Kansas Department of Wildlife and Parks (KDWP) and the Kansas Department of Transportation (KDOT). KDOT currently inspects bridges in the KDWP system and administers any substantial maintenance projects that are required. This allows KDWP to take advantage of the in-house capacity and expertise of KDOT's Bridge Management Office. Using DOT crews to maintain refuge signs would allow the work to take place without having to increase refuge staff and would allow the work to be done by experienced crews.

The annual per sign cost of maintaining retroreflectivity was calculated to be $5.14 for a 15-year blanket replacement program, which compares to the $5.09 per sign calculated for blanket replacement by NCDOT (19). Using DOT crews might allow for an inspection-based sign maintenance program on the refuges, with its significantly lower costs.

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Review of Crashes at Bridges in Kansas

Mark Hurt
Bridge Design Squad
Kansas Department of Transportation
700 Southwest Harrison St
Topeka, KS 66603
hurt@ksdot.org

Robert Rescot
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
1530 West 15th Street, Suite 2160
Lawrence, KS 66045
rrescot@ku.edu

Steven D. Schrock
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
1530 West 15th Street, Room 2150
Lawrence, KS 66045
schrock@ku.edu

ABSTRACT

In 2005, there were 1,919 crashes on bridges in Kansas, including 26 fatal crashes. While overall these accounted for less than 3% of all traffic crashes, they accounted for almost 7% of the total number of fatal crashes. The presence of bridges unavoidably introduces substantive obstacles into the roadside environment with the potential to increase the severity of crashes. A literature review of research into crashes at bridges is made to determine factors associated with these crashes. Data for crashes on bridges in the State of Kansas for the year 2005 were reviewed and characterized. Crash data and bridge inspection data for bridges on the State Highway System for the years 2005–2007 were analyzed to determine if either the type or condition of specific bridge elements are variables with any correlation to crash rates. Positive correlations with crashes were found with both the presence of parapet rails, an older type of bridge railing, and categorization of bridges as “Structurally Deficient.” Negative correlations with crashes were found with the presences of overlays on bridges with older rail types, which suggest that bridge maintenance actions may have a positive effect on traffic safety.

Key words: bridge—bridge maintenance—bridge rail—crash rate

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INTRODUCTION

A saying common among bridge designers is that, “a bridge is to a road as a diamond is to a ring.” They are artifacts of sophisticated engineering and intensive construction efforts provided to carry drivers over inherent hazards and conflicts, such as streams and cross traffic. Bridge sites are typically among the most complex roadside environments with the presence of railing, guard fence and other appurtenances and common transitions in roadway width and surfacing. As such, bridge sites may present drivers with more features to process and negotiate, while providing fewer opportunities for recovery than adjacent roadways.

In a review of the literature, bridge sites have been shown to be locations for a disproportionately higher number of crashes than the surrounding roadways. Additionally, crashes at bridges have tended to be more severe. This is confirmed by a review of the 2005 crash data in the State of Kansas. In that year, there were 1,919 crashes on bridges in Kansas, including 26 fatal crashes. While overall these accounted for less than 3% of all of that year’s traffic crashes, they accounted for almost 7% of the total number of fatal crashes.

Previous investigations have examined the relationships between geometric conditions (such as grade and roadway width), traffic characteristics (volume, capacity, composition), and crash rates, but there has been little, if any, investigation into relationships between the physical condition of bridges and crash rates. Bridge condition information from periodic bridge inspections is data that are readily available to any jurisdiction. By federal law (Title 23, Code of Federal Regulations, Part 650, Subpart C), all bridges longer than 20 feet on public roads in the United States must be inspected according to the National Bridge Inspection Standards (NBIS) with the results reported to the Federal Highway Administration. The default frequency of inspection is a two-year interval. Bridges with more critical conditions may be inspected more often, and states may request that less complex structures (such as culverts) be inspected at a less frequent (typically four year) interval.

In this study, condition and component information from inspections of bridges on the Kansas State Highway System (State, US, and Interstate routes) for which crashes were reported to have occurred at in the years 2005–2007 is examined to determine if there is a correlation between the condition of any particular bridge element, or the type of any particular element, and crash rate. Any significant correlations would allow entities responsible for maintaining bridge inventories to identify actions (such as overlays) that might confer a safety benefit or bridge inspection findings that might indicate that further investigation is warranted.

LITERATURE REVIEW

A comprehensive review of literature published in the 1970s and 1980s concerning crashes at bridges was compiled by Ogden (1989) at Monash University in Melbourne, Australia. The reader is referred to his work for a more in-depth review of research during that period. Two particular works of this period that are important to understanding crashes at bridges are reviewed below.

First, Ivey et al. (1979) studied crashes at narrow bridges for National Cooperative Highway Research Program Report 203. Observing bidirectional traffic at 25 two-lane bridges, they observed that drivers slow approximately two miles per hour when approaching a bridge. The most significant driver reaction, however, was a tendency to reposition the vehicle laterally, towards the centerline of the bridge. The magnitude of the lateral movement was dependent on the width of the bridge roadway and on the ratio of the bridge to approach roadway width. Movement varied from two feet for bridge roadways of 15 feet to one foot on bridges with roadways of 27 feet or more. Additionally, lateral adjustment tended to be slight if the ratio of the bridge to the approach roadway width was 1.25 or greater. For ratios of 1.0, the adjustment was less than a foot.

Furthermore, Ivey (1979) also developed a “bridge safety index” based on ten factors for each bridge site; the characteristics these factors were based on were

- Bridge width
- Relative bridge width (ratio of bridge to approach roadways)
- Guardrail and bridge rail
- Approach sight distance
- Distance from end of bridge to an adjacent horizontal curve
- Grade continuity
- Shoulder reduction
- Volume to capacity ratio
- Traffic composition
- Distractions and roadside activities

Of these, for Ivey, the three most important were the bridge roadway width, the relative width, and the guardrail/bridge rail factors. In his review of this and subsequent works, Ogden (1989) found that the primary factors found to be significant in relation to crashes at bridges were

- Bridge width
- Relative bridge width
- Traffic volume
- Curvature and grade on the approach to the bridge

These findings agree with earlier works by Williams and Fritts (1955) and Gunnerson (1961). Each study found that an approach narrower than the adjacent bridge would contribute to a higher crash rate even, in the case of the Gunnerson study, if the wider approach was the result of upgrade in the approach roadway at an existing bridge.

Second, Turner (1984) examined crashes on rural, two-lane, bidirectional roads in Texas for a four-year period. In analyses of correlation and regression of variables associated with the roadway and the bridges, he found that the three most important variables were the relative width of the bridge (he defined as bridge width minus the approach roadway width), the average daily traffic (ADT), and the approach roadway width. A probability table for crashes per million vehicles based on relative width and approach roadway width was developed. Regression analysis of the dataset based solely on relative width resulted in a model with an R^2 of 0.62. Further refined resulted in an equation relating crash rates solely to relative width for bridges whose roadways weren’t considered “extremely narrow” (more than 4 feet less than the adjacent roadway) with an R^2 of 0.81.

Reviewing the literature, it appears that the consensus of researchers was that the primary factor involved with the increase in crashes at bridge sites relative to the rest of the roadway was lateral repositioning of traffic in response to narrowing of the roadway on the bridge. Later studies focused on collisions of overheight vehicles with bridges and factors associated with crash severity rather than correlations with increased crash rates.

As documented by Fu (2004), a 1999 fatality precipitated by the collision of an overheight truck and load with a pedestrian bridge in Maryland resulted in a study by the Maryland State Highway Administration into overheight vehicle collisions with bridges across the nation. Though such collisions are common, as characterized in the report by noting that one in every five overpasses in Maryland has had such a collision at some point in the life of the structure, they are mainly a maintenance issue. The impact structures may require repair, but rarely is there loss of life as was incurred in the 1999 incident. Overheight collisions were not reviewed in this study.

As part of a larger study of the in-service performance of roadside hardware on the state route system in Washington State, Shankar (2000) conducted a statistical study of the impact of bridge rail types on vehicular crash severity. Metal rails and concrete baluster rails with curb widths 10 inches or over were found to underperform as compared to thrie-beams and concrete safety-shape rail. In particular, concrete balusters were found to be associated with an increase in the probability of injury for crashes at posted speeds over 40 mph. This finding will be of interest when reviewing the Kansas data later in this report.

CHARACTERISTICS OF CRASHES AT BRIDGES IN KANSAS

In 1976, Agent and Deen (1976) of the Kentucky Department of Transportation reported that 8% of crashes studied on Kentucky's Interstates and Parkways involved bridge, with 14% of the fatal crashes. They reported a less severe rate (3% of all crashes and 4% of the fatal crashes) for bridge involvement on the primary and secondary highway system of Kentucky. To examine whether bridges are also disproportionately likely sites for crashes in Kansas in the early part of the 21st century, statistics on all crashes on all roads (local and state jurisdictions) were obtained from the Transportation Planning Bureau of the Kansas Department of Transportation (KDOT). The total number of crashes in Kansas in the years 2005 and 2006 are reported in Table 1, with a breakout by fatal, injury, and property damage only (PDO) crashes.

Table 1. Crashes on all roads in Kansas for the years 2005 and 2006

Year	Crashes				People	
	Total	Fatal	Injury	PDO	Deaths	Injury
2005	68,675	384	16,185	52,106	428	22,723
2006	65,460	427	15,792	49,241	468	22,320
Total	134,135	811	31,977	101,347	896	45,043

The proportion of fatal crashes to total crashes by year is

- 0.6% in 2005
- 0.7% in 2006

The proportion of injury crashes to total crashes by year is

- 23.6% in 2005
- 24.1% in 2006

The database of these crashes was filtered to determine which reports had a "special features" value of 1, which would indicate a bridge at the crash scene. Similar to Table 1, the total number of crashes on all roads at bridges in Kansas in the years 2005 and 2006 are reported in Table 2, with a similar breakout by fatal, injury, and property damage only (PDO) crashes.

Table 2. Crashes on bridges on all roads in Kansas for the years 2005 and 2006

Year	Crashes				People	
	Total	Fatal	Injury	PDO	Deaths	Injury
2005	1,919	26	586	1,307	26	811
2006	1,599	24	478	1,097	24	685
Total	3,518	50	1,064	2,404	50	1,496

At bridges, the proportion of fatal crashes to total crashes by year is

- 1.4% in 2005
- 1.5% in 2006

At bridges, the proportion of injury crashes to total crashes by year is

- 30.5% in 2005
- 29.9% in 2006

For 2005, 2.8% of all crashes were at bridges, while 6.8% of all fatal and 3.6% of all injury crashes were at bridges. For 2006, 2.4% of all crashes were at bridges, while 5.6% of all fatal and 3.0% of all injury crashes were at bridges. The percentage of crashes occurring at bridges with respect to the total number of crashes on all Kansas roads is similar to, but slightly better than, the 3% on Kentucky's primary and secondary highway system in the mid-1970s; however, the proportion of fatal crashes appears to be worse than even Kentucky's Interstate and parkway numbers. In 2005, 2.8% of all crashes resulted in 6.8% of all fatal crashes occurring at bridges in Kansas; in 1976 on Kentucky's Interstate and parkway system, the corresponding percentages were 8% and 14%.

Data for crashes at bridges for the year 2005 were further reviewed to sort by accident classification and contributing circumstance assigned by the officer writing the crash report. The results are shown in Table 3. Accident classification refers to whether the crash involved a collision with another object and what that object might be. The classification codes in the table are self explanatory. On most crash reports in Kansas, the reporting officer assigns a primary contributing circumstance. These are organized into five classes, involving, respectively, the

- Driver—25 possible circumstances are provided for use with code for the officer, from "under the influence of drugs" to "reckless/careless driving"
- Environment—involving weather or obstructions such as vegetation or animals
- On/at road—the condition of the road surface resulting from the weather or state of repair or disrepair
- Vehicle—particular vehicular systems, i.e., brakes, tires, etc.
- Pedestrian—only nine particular circumstances are coded for use by the officer, but circumstances provided cover both impairment of, and improper operation by, the individual.

The majority of crashes involve collisions with either fixed objects in the roadside environment (59.7% of the bridge crashes) or other moving motor vehicles (25.4%). The majority of crashes have had the driver (60.0%) cited as the primary contributing circumstance. This may be consistent with the presumption that lateral repositioning of drivers is a primary contributor to crashes at bridges.

Table 3. Classification of crashes on bridges on all roads in Kansas for the year 2005

Accident Classification	Total	Primary Contributing Circumstance					Not clearly attributed
		Driver	Environmental	On/At Road Conditions	Vehicle	Pedestrian	
Class 00 (Noncollision accidents--Jackknifed, Broken Vehicles, etc)	36	13	1	3	4	0	15
Class 01 (Overturned)	94	54	11	9	0	0	20
Class 02 (Collision with Pedestrian)	1	1	0	0	0	0	0
Class 03 (Collision with Other Motor Vehicle)	487	383	13	32	10	0	49
Class 04 (Collision with Parked Vehicle)	19	14	3	0	0	0	2
Class 05 (Collision with Train)	0	0	0	0	0	0	0
Class 06 (Collision with Pedicycle)	2	0	0	0	0	1	1
Class 07 (Collision with Animal)	109	3	73	0	0	0	33
Class 08 (Collision with Fixed Object)	1146	680	126	145	28	0	167
Class 09 (Other accidents)	25	4	1	14	0	0	6
Total by Primary Contributing Circumstance	1919	1152	228	203	42	1	293

The 2005 database was further examined to review the fixed-object collision and sort by object struck and contributing circumstance. The results are shown below in Table 4.

There are a number of items for which it is counterintuitive to associate with crashes at bridges, i.e., mailboxes and trees, but it should be remembered that the majority of Kansas bridges are small-span structures and that these data includes rural county roads in Kansas. One item of note is that 748 (65.3%) of the fixed-object collisions involve impacts to bridge rails, crash cushions, barriers, and guard fences. Conversely, more than 1 in 3 of fixed-object collisions at a bridge involves items in the roadside that are not designed to redirect or slow impacting vehicles. When looking at the entire 1,919 crashes at bridges in Kansas in 2005, only 39.0% of them involve the safety appurtenances designed to protect drivers from injury.

Table 4. Fixed object collisions on bridges on all roads in Kansas for the year 2005

Fixed Object Type	Total	Primary Contributing Circumstance					Not clearly attributed
		Driver	Environmental	On/At Road Conditions	Vehicle	Pedestrian	
FOType 01- Bridge Structure	251	145	23	19	9	0	55
FOType 02- Bridge Rail	433	247	56	60	12	0	58
FOType 03- Crash Cushion	2	1	0	0	0	0	1
FOType 04- Divider/Median Barrier	95	63	10	15	4	0	3
FOType 05- Overhead Sign Support	2	1	1	0	0	0	0
FOType 06- Utility Pole	8	6	1	0	0	0	1
FOType 07- Other Pole/Post	11	4	0	2	0	0	5
FOType 08- Building	3	2	0	1	0	0	0
FOType 09- Guardrail	218	138	26	25	2	0	27
FOType 10- Sign Post	23	13	0	5	0	0	5
FOType 11- Culvert	11	6	0	1	0	0	4
FOType 12- Curb	4	4	0	0	0	0	0
FOType 13- Fence	15	8	1	4	0	0	2
FOType 14- Hydrant	0	0	0	0	0	0	0
FOType 15- Barricade	3	3	0	0	0	0	0
FOType 16- Mailbox	3	2	0	1	0	0	0
FOType 17- Ditch	16	8	2	4	0	0	2
FOType 18- Embankment	18	12	0	4	1	0	1
FOType 19- Wall	16	9	3	3	0	0	1
FOType 20- Tree	12	7	3	1	0	0	1
FOType 21- RR Crossing Fixtures	0	0	0	0	0	0	0
FOType 88- Other	2	1	0	0	0	0	1
Total by Primary Contributing Circumstance	1146	680	126	145	28	0	167

Analysis of Crash Data

To examine any correlations that might exist between the physical condition or particular roadway components of bridges and crash rates, crash data for bridges only on state and federal routes were obtained from the KDOT Planning Bureau for the years 2005 to 2007. Note that the crashes in Kansas discussed in the previous section occurred on local, state, and federal routes. All bridges on state and federal routes in Kansas have detailed bridge inspection records meeting NBIS on file and available in a

PONTIS Bridge Management database. Crash data retrieved by KDOT were taken from the Kansas Accident Record System (KARS), which contains officer written crash reports. Recognizing the very specific location of the relevant crashes, the data were vetted by the Geographic Information Systems (GIS) section at KDOT to verify that indeed these crashes occurred at a known bridge location. This query returned a dataset of 3,272 crashes. Using GIS technology, details on the specific bridge that each crash occurred on was obtained, which determined that these crashes occurred on 1,418 unique bridges. Then, the NBIS data were added to the crash record along with ADT volumes. Included with the NBIS data is information on condition ratings for each of the major elements of each bridge; for a span structure, these elements include the deck, superstructure, and substructure. Additionally, in Kansas, a condition rating for the approach roadway alignment is also reported. However, for various reasons, some of the NBIS or ADT data field were incomplete, omitted, or contained invalid entries, and thus, the bridge was removed from consideration. This resulted in 1,193 remaining bridges, which accounted for 2,972 crashes. This then served as the full dataset used for the analysis, as shown in Figure 1.

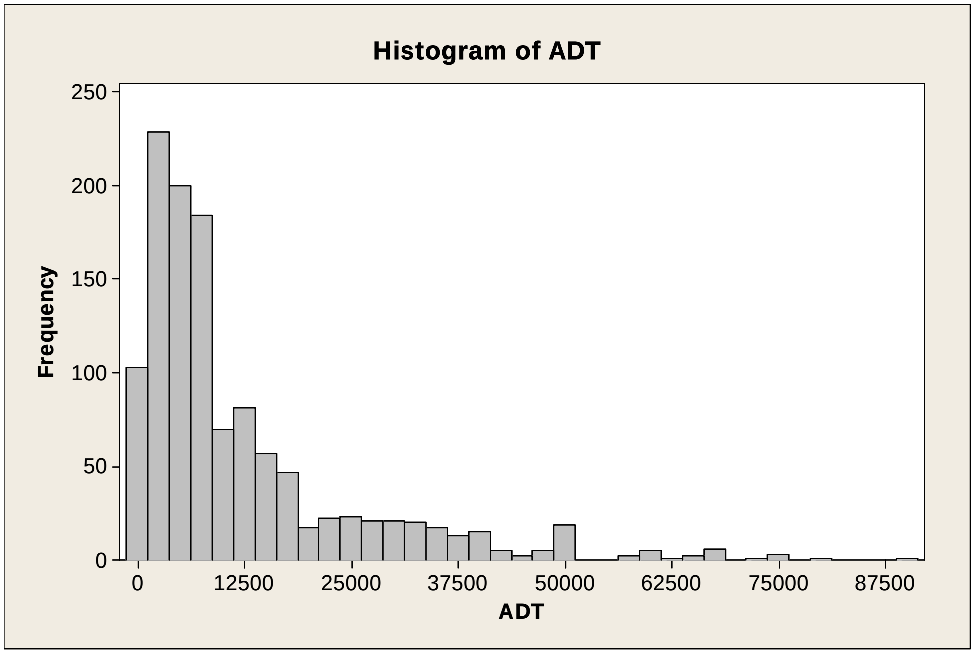

Figure 1. Histogram of ADT for bridges studied

Regression Analysis

In developing a model that accounts for the various factors provided by the dataset, the first task was to create a response variable for the model to predict. This was accomplished for each bridge by taking the number of crashes that had occurred on it during the study period and dividing by a three year estimate of the traffic volume on the bridge. The three year traffic volume was found by multiplying the ADT by 365 days in a year and multiplying again by 3 years. The final step in preparing a response variable was to take a logit transformation of the crash rate because it would result in a more normally distributed value than the straightforward nominal crash rate, as shown in Figure 2.

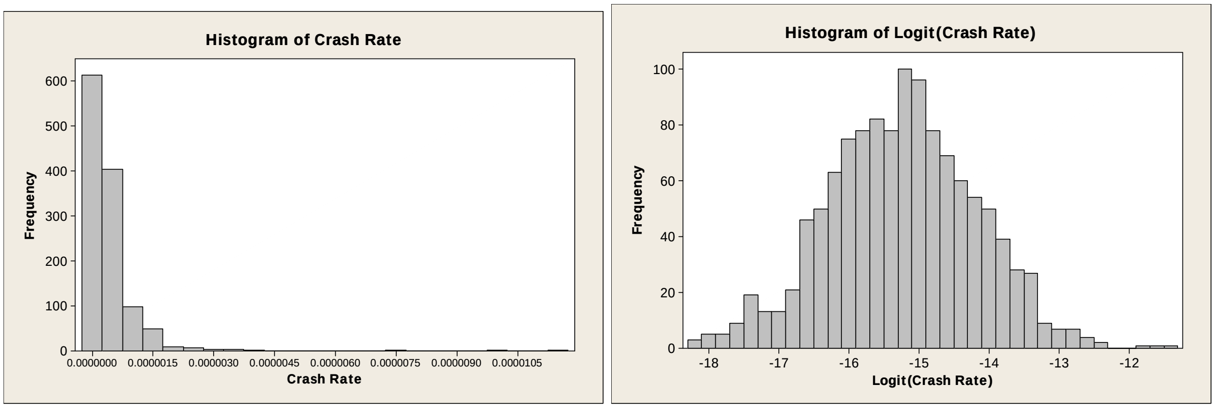

Figure 2. Histograms of crash rate before and after a logit transformation

A multiple linear regression model was then constructed to predict the logit transformed crash rate. Using a level of significance of 0.05, the final model, presented in equation (1), was the result of numerous iterations, utilizing an initial pool of 163 variables, which included 128 possible interaction terms. An extra sum of squares F-Test showed that with 95% confidence, a null hypothesis that the interaction term coefficients were all zero could not be rejected, and thus, were included in the process. The final version of the model as shown in equation (1), has an adjusted R^2 value of 46.7%.

$$\begin{aligned}\text{Logit(Crash Rate)} = & - 13.7 - 0.000036\ \text{ADT} + 0.0133\ \text{Truck\_Percent}\\ & - 0.279\ \text{Functionally\_Obsolete} + 0.609\ \text{Structurally\_Deficient}\\ & - 0.00626\ \text{Sufficiency\_Rating} + 0.00309\ \text{Condition\_Index}\\ & + 0.0883\ \text{Deck\_Rating} - 0.140\ \text{Approach\_Alignment} \qquad (1)\\ & - 0.0142\ \text{Approach\_Roadway\_Near} + 0.734\ \text{Concrete\_Parapet}\\ & - 0.0988\ \text{Silica\_Fume*Concrete NJ 32"}\\ & - 0.710\ \text{Silica\_Fume*Concrete\_Parapet}\\ & - 1.89\ \text{Polymer\_Concrete*Concrete\_Parapet}\end{aligned}$$

The variables are explained in Table 5. All variables are available from the bridge inspection records for bridges on state and federal routes in Kansas.

The variables in the equation that have coefficients with an absolute value greater than 0.1 are (in order of magnitude):

- Polymer Concrete overlay with Concrete Parapet railing (-1.89)
- Concrete Parapet railing (+0.734)
- Silica Fume overlay with Concrete Parapet railing (-0.710
- Structurally Deficient (+0.609)
- Functionally Obsolete (-0.279)
- Approach Alignment rating (-0.140)

Table 5. Variables in the regression model

Variable	Definition
ADT	Current average daily traffic at bridge site.
Truck Percent	Current percentage of truck traffic at bridge site.
Functionally Obsolete	A positive value of one indicates that the bridge is categorized as “Functionally Obsolete” by NBIS. Typically, this is due to a narrow roadway.
Structurally Deficient	A positive value of one indicates that the bridge is categorized as “Structurally Deficient” by NBIS. Typically, this is due to poor superstructure or substructure condition.
Deck Rating	This is an NBI rating of from 9 to 1, with 9 being a new deck and 1 being so poor as to close the bridge. A typical worst case on the Kansas State Highway System is a rating of 4, which requires posting the bridge.
Approach Alignment	This is an NBI rating of from 8 to 1, with 8 being a bridge roadway greater than the approach roadway and including an 8 foot shoulder or greater and 1 being so poor as to close the bridge. Advanced signing is required at a rating of 2.
Approach Roadway Near	This is the width of the approaching roadway, including stabilized (rock or pavement) shoulders, on the upstation side of the bridge. Stationing typically runs from west to east or south to north on routes.
Concrete Parapet	A positive valve indicates the presence of concrete parapet handrail on the bridge. This is an older type of rail illustrated in the photo in Figure 3.
Silica Fume	A positive valve indicates the presence of a silica fume overlay as the wearing surface of the bridge. This is commonly applied as a maintenance action on deteriorated decks or may be applied on new construction if ADT is particularly high.
Concrete NJ 32”	A positive valve indicates the presence of 32-inch high safety-shape barrier.
Polymer Concrete	A positive valve indicates the presence of a polymer concrete overlay as the wearing surface of the bridge. To date, this had been only applied as a maintenance action on deteriorated decks.

Combination terms indicate the presence of both items.

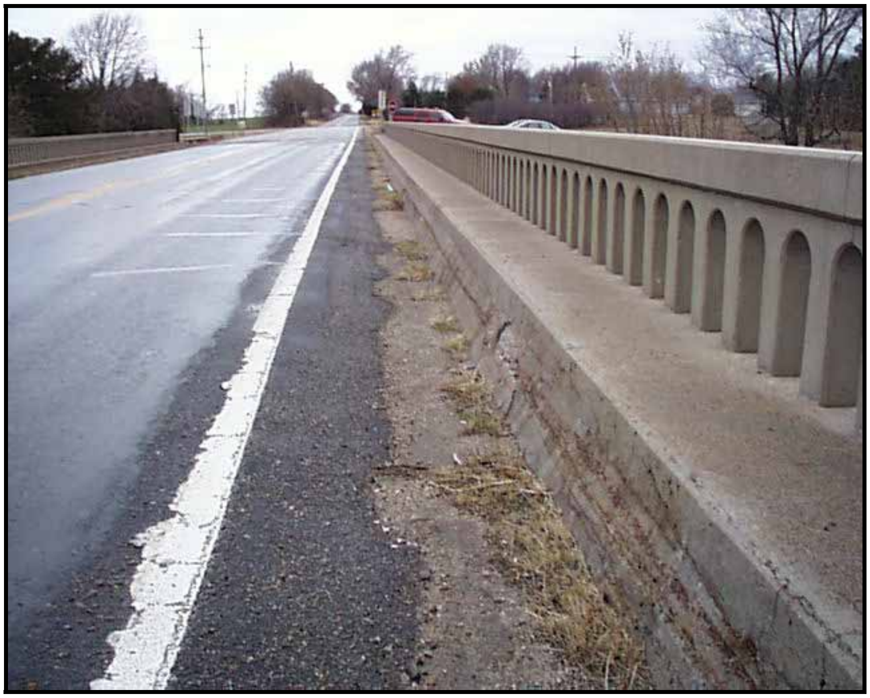

Figure 3. Typical concrete parapet bridge railing in Kansas

The negative coefficient for the approach alignment rating is not surprising; the higher the alignment rating, the better the alignment is. A negative coefficient for “Functionally Obsolete” is surprising, however. The good fortune of two consecutive ten-year highway construction programs may help explain this peculiarity. A “Functionally Obsolete” bridge that is not also “Structurally Deficient” is likely narrow, but not subject to heavy traffic that would cause wear to the physical components of the bridge. After 20 years of improvements to the State Highway System in Kansas, such a bridge is most likely to be found on a rural, well-maintained section of road.

Positive coefficients for “Structurally Deficient” and for the presence of “Concrete Parapet” railing are also not surprising. Both indicate an older bridge, which typically means that the surrounding roadside conditions are artifacts of design from the same vintage. Both also indicate that there have been no major reconstruction projects at the particular site, which would have updated the roadside environment. Concrete parapet is also typically found on narrower bridges. Wider, old bridges were more likely found on routes with higher ADT and were more likely replaced or reconstructed in the past highway programs. The lateral placement of drivers in response to the larger vertical profile of the parapet rail may also have an effect.

The largest negative coefficients were found in combinations of the presence of an overlay with either a New Jersey or a parapet rail. This was unexpected but can be readily explained as the positive outcome of bridge maintenance actions. New or reconstructed bridges on the Kansas State Highway System will have either 32 inch corral rail or 32 inch F-shaped barrier rail. Older bridges will have had the parapet or, in urban areas, the New Jersey barrier rails. Since polymer concrete overlays have yet to be applied on new construction in Kansas, the presence of them on a bridge indicates that a maintenance project has occurred. Typically, bridge maintenance projects also include repair of the deck, approaches, and railing. A silica fume overlay on an older bridge likely indicates a similar maintenance action in an urban area.

CONCLUSIONS

A review of all crashes at bridges in Kansas for the years 2005 and 2006 show that the rate of such crashes might be slightly less than for other states in previous years in the literature; however, the proportion of fatal crashes has not dropped. There still is considerable measure for improvement of bridge sites as roadside locations with respect to crash rates.

Review of all crashes at bridges in Kansas for the year 2005 showed that only 39% of crashes involve the vehicle striking the railing or barrier—the items on the bridge designed for such impact. The severity of over half of the crashes on bridges would not be improved by updating such hardware. Real improvement would result from actions that prevent crashes.

A regression analysis of bridges on the Kansas State Highway System on which crashes occurred in the years 2005–2007 shows that the most strongly positive correlated factors are whether a bridge has the older concrete parapet style of bridge rail and if it is categorized as "Structurally Deficient" according to the NBIS. The most negatively correlated factor is if it is an older bridge (as indicated by railing type) with an overlay, e.g., if the bridge has been subject to a substantial maintenance action by KDOT. It may be that bridge maintenance activities, programmed solely to preserve the value of the bridge inventory in terms of structural adequacy, may also benefit the safety of the traveling public by reducing crash rates.

The most immediate actionable item from this investigation may be to note the presence of either concrete parapet rail or a "Structurally Deficient" classification on a bridge inspection as a impetus to review the accident reports at a given bridge site to see if improvements are needed.

Future Work

The results of this investigation point to multiple possible future avenues of study, but the first item to undertake would be refinement of the regression model to one with a higher R^2 value. After this, it may prove useful to investigate sites with concrete parapet rail on all roads in Kansas and to further investigate the usefulness of bridge maintenance actions with regard to improving traffic safety.

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Study of Iowa PCC Thermal Properties for Mechanistic-Empirical Pavement Design

Jiong Hu
Department of Engineering Technology
Texas State University, San Marcos
1244 RFM, 601 University Drive
San Marcos, TX 78666
jh211@txstate.edu

Zhi Ge
Department of Construction Management and Engineering
North Dakota State University
CME Building, Room 120
Fargo, ND 58105
Zhi.Ge@ndsu.edu

Kejin Wang
Department of Civil, Construction, and Environmental Engineering
Iowa State University
492 Town Engineering
Ames, IA 50011
kejinw@iastate.edu

ABSTRACT

The present research was designed to determine thermal properties, such as coefficient of thermal expansion (CTE) and thermal conductivity, of portland cement concrete (PCC) used in Iowa pavements. These properties are required as input values by the AASHTO *Mechanistic-Empirical Pavement Design Guide* (MEPDG). In this research, a literature survey was conducted to determine the major factors that significantly affect thermal properties of PCC. CTE tests of various lab and field samples were performed. The variations due to the test procedure, equipment used, and consistency of field batch materials were evaluated. The test results showed that the variations in CTE values due to test procedure and batch consistency were relatively small (<5%), while the variation due to the different equipment was relatively large (<15%). Concrete CTE values were significantly affected by different types of coarse aggregate. The CTE values of Iowa concrete made with gravel-limestone, quartzite, dolomite, dolomite-limestone, and limestone were 7.27, 6.86, 6.68, 5.83, and 5.69 microstrain/°F (13.08, 12.35, 12.03, 10.50, and 10.25 microstrain/°C), respectively, which were all higher than the default value of 5.50 microstrain/°F (9.90 microstrain/°C) in the MEPDG program. In addition, the thermal conductivity of a typical Iowa PCC mix and an asphalt cement concrete (ACC) mix (both with limestone as coarse aggregate) were tested. The thermal conductivity was 9.24 Btu•in/h•ft^2•°F (1.33 W/m•K) for PCC and 14.52 Btu•in/h•ft^2•°F (2.09 W/m•K) for ACC, which are different from the default values (15.0 Btu•in/h•ft^2•°F or 2.16 W/m•K for PCC and 8.04 Btu•in/h•ft^2•°F or 1.16 W/m•K for ACC) in the MEPDG program. Results indicated that appropriately documenting concrete thermal properties is essential for updating the typical material input values and providing rational concrete pavement design in future.

Key words: coefficient of thermal expansion (CTE)—MEPDG—thermal conductivity

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PROBLEM STATEMENT

The thermal properties of portland cement concrete (PCC) and asphalt cement concrete (ACC) or hot mix asphalt (HMA), such as thermal conductivity, and coefficient of thermal expansion (CTE), are required as inputs by the new *Mechanistic-Empirical Pavement Design Guide* (MEPDG). Previous research on the MEPDG conducted in Iowa (Coree et al. 2005) has indicated that CTE and thermal conductivity of concrete are either sensitive or extremely sensitive to pavement design results. However, a very small amount of test data is available on the thermal properties of Iowa PCC and ACC materials. In the present research, necessary tests were conducted with Iowa concrete materials to provide engineers with basic thermal input values for the MEPDG in the state of Iowa.

RESEARCH OBJECTIVES

The main objectives of this research were to study the thermal properties of typical Iowa concrete materials and to investigate the effects of Iowa aggregates on those concrete thermal properties. The research was designed to help better implement the MEPDG in the state of Iowa.

LITERATURE SURVEY OF PAVEMENT THERMAL PROPERTIES

A literature review has indicated that the thermal properties of concrete are more complex than those of many other materials because concrete is a composite material and its components have different thermal properties. Table 1 shows that the thermal properties (CTE and thermal conductivity) of concrete and its constituents vary largely. The properties may change even more with the environment in which concrete is exposed since the concrete thermal properties also significantly depend on the moisture content and porosity of the concrete.

Table 1. Thermal properties of concrete and concrete constituents (adopted from Mindess et al. 2003)

	CTE, 10^{-6}/°F (10^{-6}/°C)	**Thermal conductivity, Btu•in/h•ft^2•°F (W/m•k)**
Aggregate		
Granite	4.0-5.0 (7-9)	21.6 (3.1)
Basalt	3.3-4.4 (6-8)	9.6 (1.4)
Limestone	3.3 (6)	21.6 (3.1)
Dolomite	4.0-5.5 (7-10)	25.2 (3.6)
Sandstone	6.1-6.7 (11-12)	27.6 (3.9)
Quartzite	6.1-7.2 (11-13)	30.0 (4.3)
Marble	2.2-4.0 (4-7)	19.2 (2.7)
Cement paste		
w/c=0.4	10-11 (18-20)	9.0 (1.3)
w/c=0.5	10-11 (18-20)	8.4 (1.2)
w/c=0.6	10-11 (18-20)	7.2 (1.0)
Water	---	3.6 (0.5)
Air	---	0.24 (0.03)
Concrete	4.1-7.3 (7.4-13)	10.8-24.0 (1.5-3.5)

Coefficient of Thermal Expansion

The CTE is defined as the change in unit length of a material in response to a degree of temperature change. The stresses on pavement due to drying shrinkage and curling/warping caused by temperature or moisture differences are very sensitive to this parameter. The CTE of concrete is therefore very important for optimizing joint design for jointed plain concrete pavement (JPCP) and designing reinforcement for continuously reinforced concrete pavement (CRCP).

Factors that influence concrete CTE have been studied for many years. These factors include water-to-cement ratio (w/c), cement type, aggregate type, aggregate fraction, temperature, and the humidity condition of the specimen (Emanuel and Hulsey 1977; Kim et al. 2003).

Concrete CTE can be predicted from the CTE of cement paste and aggregate. Neville (1996) reported that the CTE of cement paste generally varies from 6 to 12 microstrain/°F (11 to 20 microstrain/°C), and the CTE of concrete decreases with the increase of aggregate content (see Table 2).

Table 2. Influence of aggregate content on CTE (adopted from Neville 1996)

Cement/sand ratio	CTE at 2 years, 10^{-6}/°F (10^{-6}/°C)
1:0 (paste)	10.3 (18.5)
1:1	7.5 (13.5)
1:3	6.2 (11.2)
1:6	5.6 (10.1)

Table 3 gives some CTE values for concrete made with different types of aggregate and used in dams. The CTE values of concrete containing quartzite and some siliceous aggregates are around 7.2 microstrain/°F (13 microstrain/°C) at normal temperatures; the CTE values of some limestone aggregate concretes can be lower than 3.33 microstrain/°F (6 microstrain/°C) for comparable conditions. As seen in Table 3, there is a wide range of CTE values for concrete, and therefore, it is important to select a proper value for concrete pavement design.

Table 3. CTE of concrete used in dams (Scanlon and McDonald 1994)

Dam	Aggregate type	Concrete CTE, 10^{-6}/°F (10^{-6}/°C)
Hoover	Limestone and granite	5.3 (9.5)
Hungry Horse	Sandstone	6.2 (11.2)
Grand Coulee	Basalt	4.4 (7.9)
Table Rock	Limestone and chert	4.2 (7.6)
Greers Ferry	Quartz	6.7 (12.1)
Dworshak	Granite-gneiss	5.5 (9.9)
Libby	Quartzite and argillite	6.1 (11.0)
Jupia (Brazil)	Quartzite	7.5 (13.6)

Yao and Zheng (2007) showed that for a given amount of water, the CTE of concrete decreased with w/c ratio. However, for a given paste content, CTE increased with w/c ratio. In addition, the CTE of concrete increased significantly at an early age but became a stable value after 28 days due to the effect of cement hydration.

Research showed that the CTE reach its maximum value at a relative humidity from 50%–70% (see Figure 1). Also, the relative humidity at which the CTE is a maximum decreases with age, which is probably owing to a reduction in the potential swelling pressure due to an increase in the amount of “crystalline” material in the hardened paste (Neville 1996). Although Figure 1 refers to neat cement pastes, the trend is similar to that of concrete. Neville (1996) found that for the same concrete, the CTE was 6.11 microstrain/°F (11 microstrain /°C) in winter and 7.22 microstrain /°F (13 microstrain/°C) in summer. Concrete age can also affect CTE test results. Concrete that is aged six months or older may reach 80% of its maximum CTE.

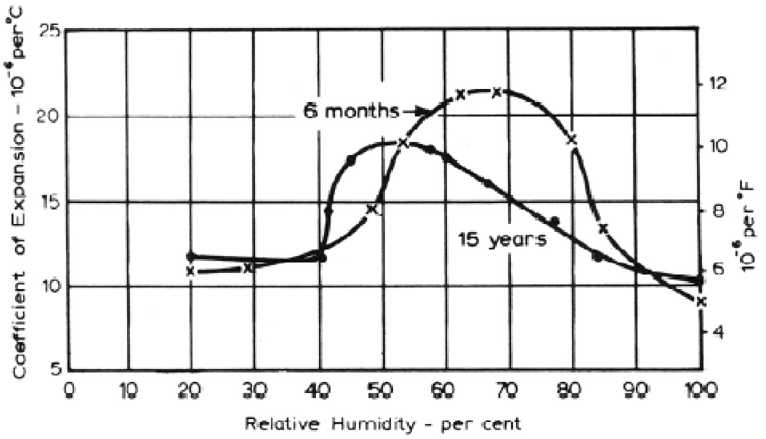


Figure 1. CTE of neat cement paste at different ages (adopted from Neville 1996)

Essentially, cement paste deformation in conventional concrete is restrained by aggregate since aggregate generally has a much higher elastic modulus and a very low thermal expansion. Therefore, the actual CTE of concrete is smaller than the predicted value based on volume fraction of aggregate and paste. Research had been conducted to relate the CTE value to the volume fraction of coarse aggregate and the CTE of coarse aggregate (Ziegeldorf et al. 1978), or weighted average of the CTE of cement paste, fine aggregate, and coarse aggregate (Yang et al.1990). Emanuel and Hulsey (1977) developed an empirical equation for concretes of various mixes, ages, and moisture contents, where the correction factors were used for the consideration of moisture and age and the volume proportion of paste, fine aggregate, and coarse aggregate. The correction factor can be used for estimating concrete CTE under different exposure.

Thermal Conductivity of Concrete

Thermal conductivity represents the ability of a material to transfer heat. It is defined as the ratio of the rate of heat flow to the temperature gradient of a material. The thermal conductivity of PCC or ACC governs the rate at which heat flows into, through, or out of a concrete structure. For normal-weight PCC, thermal conductivity is widely influenced by the mineralogical character of the aggregates, water content, air void content and structure, and the temperature and moisture condition of concrete (Scanlon and McDonald 1994; Kim et al. 2003). The amount of free water in concrete has a major influence on thermal conductivity. Table 1 shows that while water is a relatively poor conductor of heat as compared to aggregate, water’s thermal conductivity is much higher than air; therefore, thermal conductivity significantly decreases with a reduction in moisture content. The mineralogical character of the aggregates largely determines the thermal conductivity of concrete. The effects of moisture and aggregate type on thermal conductivity values are shown in Table 4.

Table 4. Thermal conductivity of concrete with different moisture conditions (adopted from Scanlon and McDonald 1994)

	Moisture conduction	Conductivity, Btu•in/h•ft^2•°F (W/m•K)
Limestone concrete	Moisture	15.0 (2.2)
	50% RH	11.0 (1.7)
	Dry	10.0 (1.4)
Sandstone concrete	Moisture	20.0 (2.9)
	50% RH	15.0 (2.2)
	Dry	10.0 (1.4)
Quartz gravel concrete	Moisture	23.0 (3.3)
	50% RH	19.0 (2.7)
	Dry	16.0 (2.3)
Expanded shale concrete	Moisture	5.9 (0.85)
	50% RH	5.5 (0.79)
	Dry	4.3 (0.62)

EXPERIMENTS AND TEST METHODS

Using the equipment available at the Iowa Department of Transportation (Iowa DOT) and Iowa State University's (ISU) Portland Cement Concrete (PCC) Research Laboratory, this study focused on measuring the CTE of PCC. Samples of a typical Iowa PCC mix and a typical Iowa ACC mix were prepared at ISU but tested for thermal conductivity at Construction Technology Laboratory (CTL) Group in Skokie, Illinois.

Coefficient of Thermal Expansion Test

The CTE of concrete in the present study was determined according to the standard test method AASHTO TP 60-00 (AASHTO 2004). The test method determines the CTE of a cylindrical concrete specimen maintained in a saturated condition by measuring the length change of the specimen over a specified temperature range (50°F–122°F or 10°C–50°C). The test apparatus is shown in Figure 2.

The measured length change is corrected for any length change in the previously determined measuring apparatus. The CTE is then calculated from the corrected length change divided by the temperature change and the specimen length:

$$\boldsymbol{CTE} = \left(\frac{\Delta L_a}{L_0}\right)/\boldsymbol{\Delta T}\ , \tag{1}$$

where ΔL_a = length change of specimen, L_0 = initial measured length of specimen, and ΔT = temperature change.

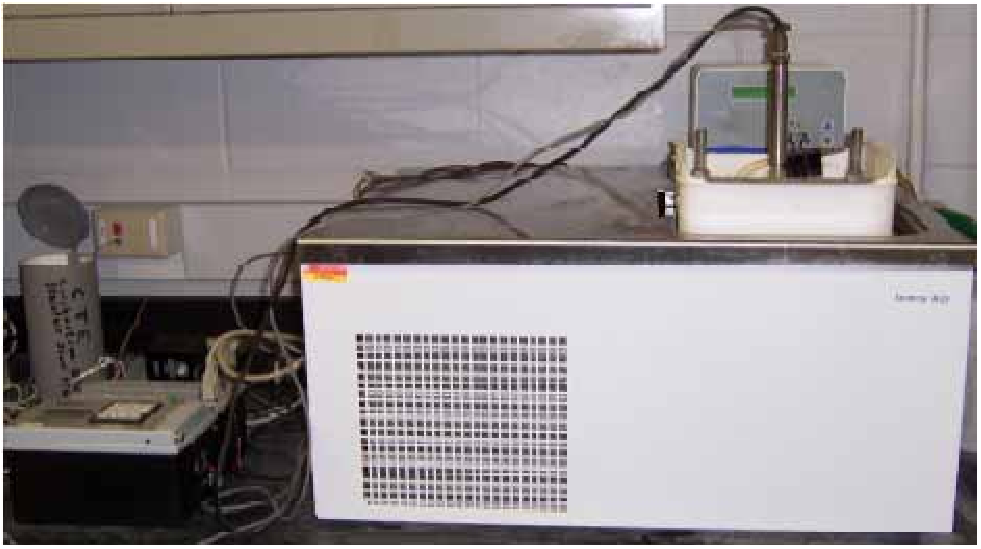


Figure 2. Test setup for CTE

Thermal Conductivity of Concrete

In the MEPDG documentation, the ASTM E 1952 test method is recommended for testing thermal conductivity of PCC and ACC (ASTM 2006). The authors reviewed ASTM E 1952 and found that this test method is specified for homogeneous materials (such as ceramic or glass) having thermal conductivity in the range of 0.7 to 7.2 Btu•in/h•ft^2•°F (0.10 to 1.0 W/m•K) and tested samples of only 10 to 100 milligrams in size. Concrete is not only an inhomogeneous material but also contains large particles. The default thermal conductivity values in the MEPDG are 15.5 and 8.4 Btu•in/h•ft^2•°F (2.16 and 1.16 W/m•K) for PCC and ACC, respectively. The authors also learned that researchers at Arizona State University (ASU) are developing a new test method for concrete thermal conductivity measurement. Considering that this test method is under development and not a standard test method yet, the authors selected a more commonly used standard test method, ASTM C 177, for the thermal conductivity measurements of Iowa PCC and ACC in the present research. The thermal conductivity of concrete was tested at CTL according to ASTM C 177 (ASTM 2004). The test was performed using an apparatus as shown in Figure 3. In the test, two concrete specimens were placed between flat steel plates. The steel plates were heated internally by special electrical resistance heaters. Temperatures were monitored by thermocouples at each surface of the specimens. The heat transferred through the specimens was equal to the power supplied to the heater. Thermal equilibrium was established when temperature and voltage readings were steady. The thermal conductivity was defined as the rate of heat flow through the material per unit thickness per degree of temperature difference across the thickness.

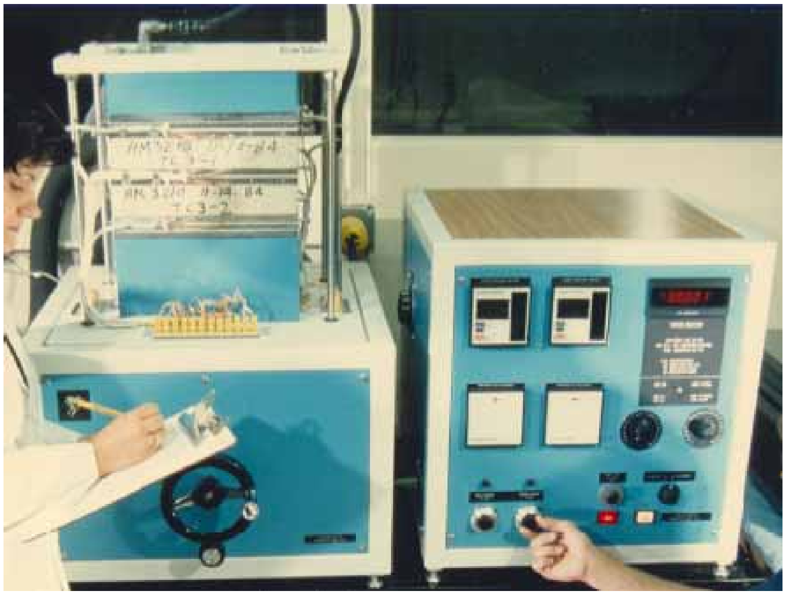


Figure 3. Test setup for thermal conductivity measurement

TEST RESULTS AND DISCUSSION

Variations in the CTE Measurements

Core samples made with three different aggregates (quartzite, limestone, and limestone-dolomite) were collected from the field by the Iowa DOT. Each sample was tested three times to determine variations in the AASHTO CTE test procedure. The test results are shown in Table 5.

Table 5. Variations in CTE from repeated tests

Core #	Aggregate type	CTE, 10^{-6}/°F (10^{-6}/°C)	Avg., 10^{-6}/°F (10^{-6}/°C)	STDEV, 10^{-6}/°F (10^{-6}/°C)	Rel. STDEV, %
3-7196	Quartzite	7.064 (12.714) 6.894 (12.409) 6.926 (12.467)	6.961 (12.530)	0.090 (0.162)	1.3
5-0260	Limestone	6.343 (11.418) 6.265 (11.277) 6.291 (11.324)	6.300 (11.339)	0.040 (0.072)	0.6
54-0004	Limestone +Dolomite	6.592 (11.866) 6.609 (11.897) 6.481 (11.666)	6.561 (11.810)	0.070 (0.125)	1.1

Table 5 shows that the standard deviation of the three tests is less than 0.10 microstrain/°F (0.20 microstrain/°C), or less than 1.5%, which indicates a good repeatability value for the CTE test procedure.

Selected samples were also tested at both the Iowa DOT and ISU to study the variation in CTE due to different equipment. Prior to the CTE tests, both test devices at the Iowa DOT and ISU were calibrated with a standard steel bar. The calibration values (CTE of the standard bar) from the Iowa DOT and ISU were 10.755 and 9.931 microstrain/°F (19.359 and 17.876 microstrain/°C), respectively; the value from the steel bar producer was 10.300 microstrain/°F (18.540 microstrain/°C).

The test results from both ISU and the Iowa DOT are shown in Table 6. The table illustrates that the standard deviations of the average values obtained from the two devices range from 0.027 to 0.784 microstrain/°F (0.048 to 1.412 microstrain/°C), all within 15%. This indicates that the test results from the Iowa DOT and ISU are in good agreement.

Table 6. Variations in CTE resulting from the use of different equipment

Core #	Aggregate	CTE, 10^{-6}/°F (10^{-6}/°C)			Rel. STDEV, %
		IA DOT	ISU PCC	STDEV	
Core 3-7174	Quartzite	6.927 (12.469)	7.072 (12.730)	0.102 (0.185)	1.46
Core 3-7186	Quartzite	7.081 (12.745)	7.118 (12.812)	0.027 (0.048)	0.37
Core 3-7051	Quartzite	6.766 (12.179)	6.844 (12.319)	0.055 (0.099)	0.81
Core 3-7091	Quartzite	6.7981 (2.236)	7.080 (12.743)	0.199 (0.359)	2.87
Core 5-0260	Limestone	5.190 (9.343)*	6.300 (11.339)**	0.784 (1.412)	13.65
Core 54-0004	Limestone +Dolomite	5.984 (10.772)*	6.561 (11.810)**	0.408 (0.734)	6.50

* Average of two testing data
** Average of three testing data

In order to study the variation in CTE tests resulting from batch mixing/production, core samples were taken from two field sites (Field site A and Field site B), both of which used Quartzite as coarse aggregate. Six samples from each site were collected and tested for CTE. The test results are shown in Table 7.

Table 7. Thermal coefficient of Iowa core samples

	Core #	CTE, 10^{-6}/°F (10^{-6}/°C)	AVG, 10^{-6}/°F (10^{-6}/°C)	STDEV, 10^{-6}/°F (10^{-6}/°C)	Rel. STDEV
Field site A	3-7174	7.072 (12.730)	7.092 (12.765)	0.100 (0.179)	1.40
	3-7176	7.179 (12.923)			
	3-7186	7.240 (13.031)			
	3-7177	6.988 (12.579)			
	3-7175	7.000 (12.600)			
	3-7180	7.071 (12.728)			
Field site B	3-7201	7.239 (13.031)	6.952 (12.514)	0.259 (0.466)	3.72
	3-7204	7.025 (12.645)			
	3-7200	7.121 (12.817)			
	3-7199	6.602 (11.884)			
	3-7198	6.663 (11.993)			
	3-7196	7.064 (12.714)			

A statistical analysis was performed to study the distribution of the measured thermal coefficients. For the Field site A and Field site B samples, the results showed a mean CTE of 7.092 and 6.952 microstrain/°F (12.765 and 12.514 microstrain/°C) with a standard deviation of 0.100 and 0.259 microstrain/°F (0.179 and 0.466 microstrain/°C), respectively. This indicates that the degree of variation of CTE test results within the given research was limited.

CTE for PCC with Different Aggregate

The Iowa DOT performed CTE tests for concrete with various aggregate types. These data were collected (see Table 8), and more tests were performed at ISU on additional field samples collected by the Iowa DOT (see Table 5). A total of 28 concrete samples made with commonly used Iowa aggregate were tested.

Table 8 indicates that the order of CTE values for concrete made with different aggregates, from high to low, is quartzite, dolomite, and limestone. Concrete made with limestone as a coarse aggregate has a lower thermal coefficient (5.69 microstrain/°F or10.25 microstrain/°C) compared to concrete made with either dolomite (6.68 microstrain/°F or12.03 microstrain/°C) or quartzite (6.86 microstrain/°F or 12.35 microstrain/°C). These results are consistent with those reported in the literature (see Table 1 and Table 3).

In the MEPDG, the default input value for CTE is 5.5 microstrain/°F (9.9 microstrain/°C), which only matches the value of Iowa concrete made with limestone. This study clearly suggests that the MEPDG should use a different value for concrete made with aggregates other than limestone. In the present study, more CTE data were obtained from the Long-Term Pavement Performance (LTPP) database (LTPP-TST_PC03). However, due to a lack of complete information on the concrete materials, these LTPP data could not be used to study the affect of concrete materials on CTE.

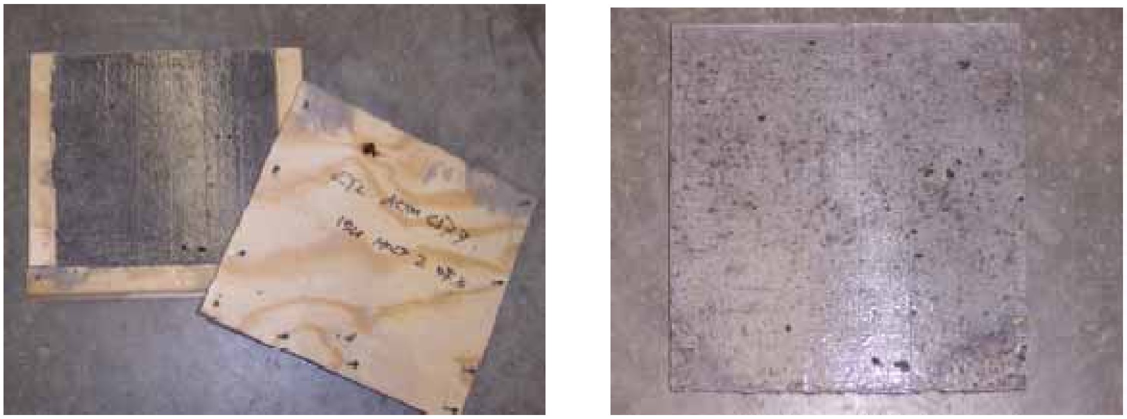
Table 8. Summary of CTE values for Iowa PCC with different types of aggregate

	CTE, 10^{-6}/°F (10^{-6}/°C)	**Test Location**	**Number of data**	**Avg. CTE, 10^{-6}/°F (10^{-6}/°C)**	**STDEV, 10^{-6}/°F (10^{-6}/°C)**
Dolomite	6.772 (12.190) 6.118 (11.012) 13.447 (7.471) 12.939 (7.188)	Iowa DOT	4	6.68 (12.03)	0.589 (1.060)
Limestone	9.844 (5.469) 6.377 (11.479) 5.235 (9.423)	Iowa DOT	3	5.69 (10.25)	0.603 (1.086)
Quartzite	6.766 (12.179) 6.798 (12.236) 6.228 (11.21) 5.858 (10.545) 7.072 (12.730) 7.179 (12.923) 7.240 (13.031) 6.988 (12.579) 7.000 (12.600) 7.071 (12.728) 7.239 (13.031) 7.025 (12.645) 7.121 (12.817) 6.602 (11.884) 6.663 (11.993) 7.064 (12.714)	Iowa DOT and ISU	16	6.86 (12.35)	0.378 (0.680)
Limestone +Gravel	6.522 (11.740) 8.016 (14.429)	Iowa DOT	2	7.27 (13.08)	1.056 (1.901)
Limestone +Dolomite	6.045 (10.882) 5.896 (10.612) 5.561 (10.01)	Iowa DOT	3	5.83 (10.50)	0.248 (0.446)
		Total	28	6.29 (11.32)	0.847 (1.525)

Study of Thermal Conductivity of Concrete

A typical Iowa PCC pavement mix (C-3WR-C20), with limestone as coarse aggregate, was selected for thermal conductivity testing. The fresh concrete had a unit weight of 143.8 pcf (2303 kg/m^3), with a slump of 2.25 in. (57 mm), air content of 5.5%, and a seven-day compressive strength of 4209 psi (29.0 MPa). Three concrete plates, with dimensions of 12 in. × 12 in. × 1.5 in. (305 mm × 305 mm × 38 mm) (see Figure 4), were prepared at ISU and then sent to CTL for testing. The thermal conductivity of the PCC concrete was reported as 9.25 Btu•in/hr•ft^2•°F (1.33W/m•K).

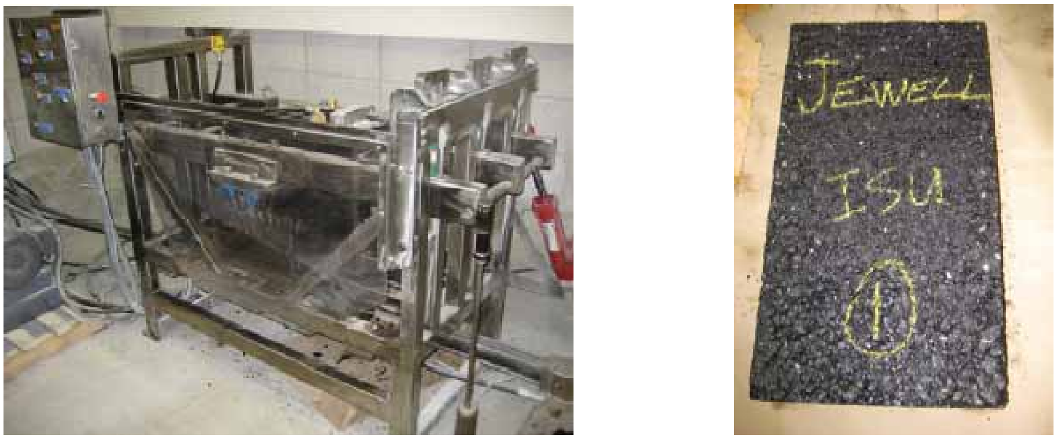
In the MEPDG, the default thermal conductivity value for PCC is 15 Btu•in/hr•ft^2•°F (2.16W/m•K), which is about 50% higher than the typical Iowa pavement mix with limestone as coarse aggregate.



(a) Mold for sample preparing **(b) Thermal conductivity test sample**

Figure 4. Preparation of PCC thermal conductivity samples

A typical Iowa ACC mix with limestone as coarse aggregate was also selected for thermal conductivity testing. The ACC had design air voids of 4%, voids in the mineral aggregate of 14.4%, and voids filled with asphalt of 72.3%. Four ACC concrete plates, with dimensions of 15 in. × 8 in. × 2 in. (381 mm × 203 mm × 51 mm) (see Figure 5), were made using a roller compactor at ISU's asphalt lab and then sent to CTL for testing.



(a) Roller compactor **(b) Thermal conductivity test sample**

Figure 5. Preparation of ACC thermal conductivity samples

In the MEPDG, the default thermal conductivity value for ACC is 8.04 Btu•in/hr•ft^2•°F (1.16W/m•K), which is about 45% lower than the typical Iowa pavement mix with limestone as coarse aggregate.

Concerns were raised regarding the difference in the thermal conductivity values between the tested Iowa PCC and ACC values and the MEPDG default values. Discussions were held among the authors and Iowa DOT members on the effects and sensitivities of the thermal conductivity on pavement performance predicted by MEPDG. With inputs from experts at FHWA, the authors learned that research has shown that as thermal conductivity increases, faulting and cracking of PCC decrease. Cracking is more sensitive to thermal conductivity when compared to faulting. It is therefore very important to have accurate thermal conductivity value for proper use of MEPDG.

However, as mentioned previously, no proper standard thermal conductivity test method is currently available for pavement concrete. The thermal conductivity tests of Iowa concrete presented above were done outside, for one PCC mix and one ACC mix only, and it is difficult to assess the accuracy of the data. Although commonly used for concrete testing, ASTM C 177 is also specified for homogeneous materials. The sample size, 12 in. × 12 in. × 1.5 in. (305 mm × 305 mm × 38 mm), seems too thin to simulate field pavement concrete condition. Thus, the test method may also be unable to provide a "correct or true" thermal conductivity value. It is reported that the test method developed at ASU requires a regular size cylinder sample and has some advantages over the C 177. The investigators have learned that FHWA is interested in getting the necessary equipment to study the thermal conductivity test method proposed by ASU, but no one knows when this will take place.

As a result, the authors suggest using the default thermal conductivity values, rather than the tested values obtained from CTL in MEPDG until this issue is addressed by the MEPDG developers and a new test method is developed and standardized in the future.

CONCLUSIONS AND RECOMMENDATIONS

1. Variations in CTE measurements resulting from test procedures, equipment used, and batch consistency were investigated. The standard deviation due to the AASHTO CTE test procedure ranged from 0.04 to 0.09 microstrain/°F (0.072 to 0.162 microstrain/°C), within 1.5%. The standard deviation due to two different test devices at ISU and the Iowa DOT ranged from 0.027 to 1.412 microstrain/°F (0.048 to 1.412 microstrain/°C), within 15%. The standard deviation due to batch material inconsistency ranged from 0.011 to 0.259 microstrain/°F (0.179 to 0.466 microstrain/°C), within 4%. These variations are generally acceptable in concrete testing.
2. Twenty-eight different CTE samples were collected and tested at the Iowa DOT and ISU. The average CTE values for concrete made with limestone, dolomite, and quartzite were 5.69 microstrain/°F (10.25 microstrain/°C), 6.68 microstrain/°F (12.03 microstrain/°C), and 6.86 microstrain/°F (12.35 microstrain/°C), respectively. In the MEPDG, the default CTE value for PCC is 5.5 microstrain/°F (9.9 microstrain/°C), which is close only to the value of Iowa concrete made with limestone. Therefore, different values should be used in the MEPDG for concrete made with aggregate other than limestone.
3. Typical mixes of Iowa PCC and ACC (both with limestone as coarse aggregate) were selected, and the thermal conductivity values of the concrete mixes were tested at CTL. The thermal conductivity values were reported to be 9.25 Btu•in/hr•ft^2•°F (1.33 W/m•K) for PCC and 14.5 Btu•in/hr•ft^2•(2.09 W/m•K) °F for ACC. Both values were significantly different than the default inputs in the MEPDG, 15 Btu•in/hr•ft^2•°F (2.16 W/m•K) for PCC and 8.04 Btu•in/hr•ft^2•°F (1.16 W/m•K) for ACC. More study shall be conducted to specify the test method and establish proper default values of concrete thermal conductivity for MEPDG.
4. A literature review has shown that the factors that affect the thermal properties of concrete include concrete materials (especially aggregate), mix proportion, moisture condition, and age. Some of these factors have been considered in the concrete CTE prediction equation. However, due to the lack of a complete set of CTE data for Iowa concrete (with CTE values and information on material and mix proportion), the calibration of this prediction equation could not be performed. Proper documentation of all material, design, and construction information is important for further study.
5. In the present research, only a small number of samples were tested and analyzed. A systematic study of the effect of mix design and aggregate type on thermal properties, especially on CTE, and thermal conductivity is essential. State DOTs shall continue to routinely run the CTE test on project cores to build a database to further refine the MEPDG input.

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Highway Capacity Improvements and Land Value Responses: Some Estimates of the Economic Impacts of Upgrading Roads

Michael Iacono
Department of Civil Engineering
University of Minnesota
275 Civil Engineering Building
500 Pillsbury Drive SE
Minneapolis, MN 55455
iaco0009@umn.edu

David Levinson
Department of Civil Engineering
University of Minnesota
275 Civil Engineering Building
500 Pillsbury Drive SE
Minneapolis, MN 55455
dlevinson@umn.edu

ABSTRACT

Improvements to transportation networks, especially those in growing areas, tend to have impacts on local land markets. In principle, an improvement to a link in the network will confer economic benefits to adjacent and nearby properties. Depending on the type of improvement (construction of a new link, capacity addition to an existing link, or upgrading an existing link), the benefit could represent a reduction in the time cost of travel or other variable costs (fuel consumption or mileage-related vehicle depreciation). Urban economic theory would suggest that these benefits are capitalized into local property values, yielding a localized spillover benefit. This paper will explore the nature and magnitude of benefits accruing to nearby properties that arise from major highway construction or reconstruction projects, more precisely those that add capacity to an existing facility. Using a sample of property sales data for Minnesota (MN) counties from 2000 through 2007, we will explore the impacts of upgrading roads on nearby properties of varying type (residential, commercial) by fitting empirical models that predict the price of a given property as a function of structural, location and other relevant characteristics. We find that residential properties benefit from being near an access point on an improved highway, but are negatively affected by being near the facility itself. Our analysis of the ROC 52 reconstruction project in Rochester, MN, also reveals some evidence of a localized benefit for owners of commercial and industrial property near the improved highway in the years following construction.

Key words: economic impact—highways—Minnesota—transportation

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INTRODUCTION

Improvements to transportation networks, especially those in growing areas, tend to have impacts on local land markets. In principle, an improvement to a link in the network will confer economic benefits to adjacent and nearby properties. Depending on the type of improvement (construction of a new link, capacity addition to an existing link, or upgrading an existing link), the benefit could represent a reduction in the time cost of travel or other variable costs, such as fuel consumption or mileage-related vehicle depreciation. It could also represent an improvement to the level of access that a given transportation network provides. Urban economic theory would suggest that these benefits are capitalized into local property values, yielding a localized spillover benefit. This paper will explore the nature and magnitude of benefits accruing to nearby properties that arise from major highway reconstruction projects, more precisely those that add capacity to an existing facility.

Specifically, this paper will take as a case study the reconstruction of U.S. Highway 52 in Rochester, Minnesota, during the period from early 2003 through late 2005. Using a sample of property sales data from Olmsted County, Minnesota, covering the years 2000 through 2007, we estimate the impact of the reconstruction of U.S. Highway 52 (the "ROC 52" project) on nearby residential and commercial properties. The remainder of the paper proceeds as follows. The second section provides a conceptual framework for the interaction of transportation network improvements and land value, tied together through the concept of accessibility. The third section provides a brief introduction to Rochester and Olmsted County, the area under study in this paper. The fourth section introduces the data set and the empirical model to be applied to the property sales data from Olmsted County to analyze the effect of the ROC 52 project. The fifth section reports the results of the empirical analysis of residential and commercial property sales. The sixth and final section summarizes the findings of the research and suggests how they might be used to inform policy.

ACCESSIBILITY, LOCATION, AND URBAN GROWTH

Observed patterns of land use in cities largely reflect the interaction of transportation networks and land markets. The mediating factor that represents this interaction is the concept of *accessibility*. Accessibility can be loosely defined as the ease of reaching desired destinations. What exactly is meant by "desired destinations" can vary, but the term generally encompasses a set of activities that households engage in on a fairly frequent basis. The most important of these activities is employment, which has been consistently identified as one of the most important (and hence, most studied) influences on the location decisions of households. Other types of activities that households might value access to include shopping destinations, entertainment venues, or educational institutions (especially higher education institutions, which are more limited in supply). Locations with higher accessibility tend to command higher prices for land, while locations with less accessibility tend to be cheaper. In cases where land is very expensive, developers substitute additional capital for scarce land, resulting in higher development densities.

The notion of accessibility also extends to the location decisions of firms. Firms, depending upon the type of industry, may value access to other types of things that lead them to cluster in certain locations. Retailers may wish to locate near their customers and near other retailers or suppliers. This leads retailers to cluster together in certain locations, like shopping malls, which are often located in high-accessibility locations (e.g., near access points of major highways). Many office and professional services activities require access to workers, which leads firms specializing in these activities to choose more central locations with higher accessibility to their respective labor markets. The premiums these firms pay for high-accessibility locations reflect the increased productivity that those locations facilitate. Even more footloose industries, like light manufacturing and warehousing, respond to the locational incentives provided by existing transportation networks and locate in places with good highways and, where required, freight rail access.

Accessibility is fundamentally a dynamic concept in that transportation networks are being continually modified over time, and that firms and households respond to these changes to transportation networks and the accessibility they provide by eventually changing their location. These location decisions and the patterns of accessibility they represent eventually become capitalized into land markets, giving rise again to a different set of location incentives. Thus, we can say that land use and transportation systems and their associated patterns of accessibility are characterized by *feedback loops*, which affect all of the different actors in these systems. A stylized representation of these feedback loops, attributable to Levinson (1997), is presented in Figure 1. Note that in Figure 1, the direction of the feedback loops between different elements of the transportation and land use system are represented by the arrows connecting them and that the (+/-) signs indicate whether the feedback effects are positive or negative.

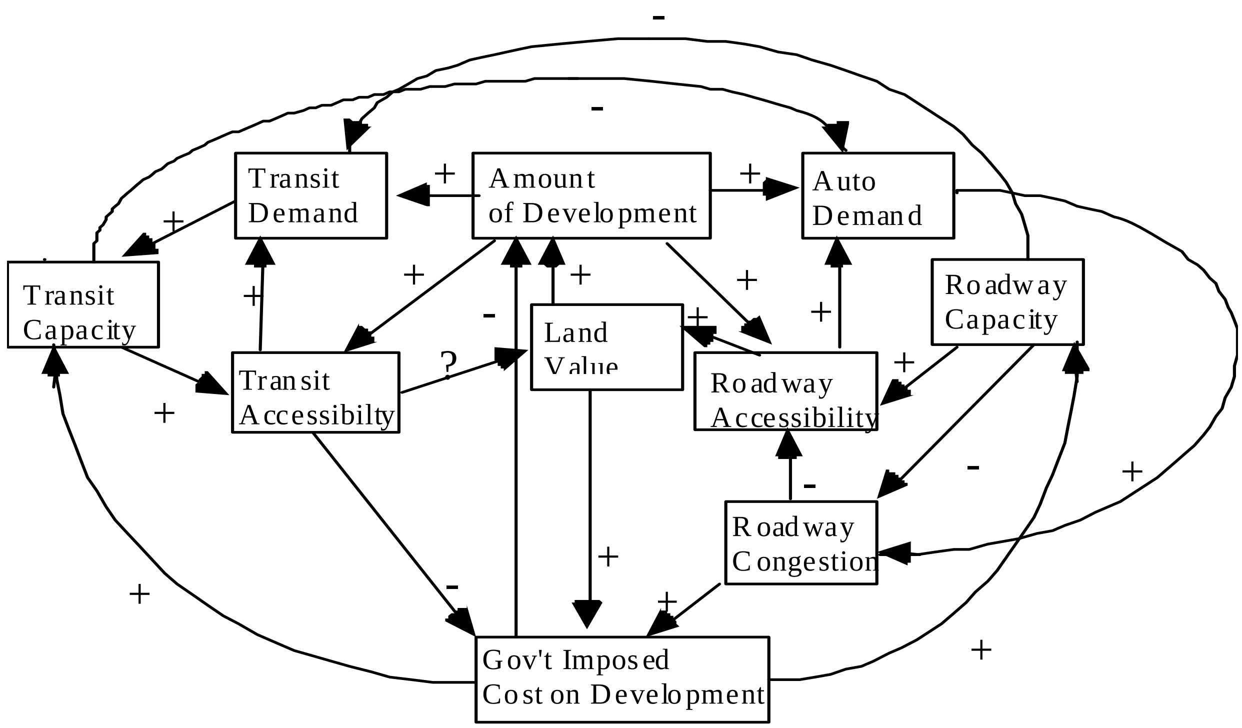


Figure 1. Feedbacks in systems of transportation and land use

The important points to note in Figure 1 are that increases in the capacity of each mode in response to rising demand lead to increases in land value and that allowing congestion to worsen leads to the opposite effect. That is because travel time acts as a disincentive to consumers to choose destinations that are further away, since consumers must expend resources to access those destinations. Increases in travel time or other travel costs reduces the number of destinations that can be feasibly accessed, given the budgets households are restricted to in terms of money or time. The feedback effects continue when the increases in land value caused by increases in accessibility in a given location lead to a larger amount of development, which again begets higher land values. In the long run, these positive and negative feedback effects tend to balance each other, with land prices playing a mediating role.

STUDY AREA

The Minnesota county we will use as a case study to estimate the effects of highway improvements on nearby property values is Olmsted County. Olmsted County is located in southeastern Minnesota, about 75 miles southeast of St. Paul via U.S. Highway 52. As of 2000, the county had a population of just under 125,000 with most of these residents living in the county's largest city, Rochester. Rochester's year 2000 population was reported as 85,806 by the U.S. Census Bureau and has more recently been estimated to be close to 100,000. As an outstate city that has experienced considerable population growth in recent years, Rochester and its surrounding county present a useful study area for examining the link between highway improvements and changes in property values.

The other major consideration in choosing Rochester and Olmsted County as a study area is that it presents an opportunity to evaluate the effects of a major, multi-year highway construction project. The reconstruction of an 11 mile section of U.S. Highway 52 in Rochester took place between 2003 and 2005. Known as the "ROC 52" project, this construction project rebuilt and expanded Highway 52 from four to six lanes between U.S. Highway 63 south of Rochester to 85th Ave. NW on the north end. While the project primarily involved reconstruction of an existing facility, patterns of access were altered as a result of the construction, and a new interchange was added along the rebuilt section. The total cost of the project was around $240 million, making it one of the largest highway construction projects in Minnesota history.

METHODOLOGICAL APPROACH AND DATA

Methodology

The method we use to estimate the effects of road network improvements is the method of *hedonic regression*. Hedonic regression models, as applied to housing markets, seek to estimate the price of housing (or other types of real property) by decomposing it into the bundle of services it provides (attributes), then estimating the implicit values that consumers place on each attribute. The method works best when it is possible to identify a larger number of attributes, especially those relating to the characteristics of structures (houses, commercial buildings, etc.). The base estimating equation (shown in equation [1]) is a standard, partial equilibrium approximation of the hedonic price function using the following form (McMillen and McDonald 2004):

$$\ln P_{it} = \alpha_t + \delta_t U_i + \boldsymbol{\beta}'\mathbf{X_i} + e_{it}\,, \tag{1}$$

where lnP_{it} represents the natural logarithm of the price of property i at its sale at time t, α_t is an indicator variable for houses that sold during time period t, U_i is a dummy variable indicating that property i is within a given distance of an upgraded road segment, $\boldsymbol{\beta}$ is a vector of coefficients to be estimated, $\mathbf{X_i}$ is a matrix of characteristics of property i, and e_{it} is a disturbance term for property i at time t. The way we choose to identify the influence of the reconstructed highway is to construct buffer zones around upgraded segments of U.S. Highway 52, then identify properties within these buffer zones with the indicator variable, U_i. We also attempt to separate out the effect of proximity to an *access point* on the highway in addition to proximity to the roadway itself.

Separate models are estimated for the residential and commercial properties available in our data set. In the case of residential property sales, where a large sample is available, the full model will be estimated with interactions between location and time period of sale. For the smaller sample of commercial-industrial properties, a more limited model that ignores the nuisance effects of proximity to the highway right-of-way is applied. As the data sets represent relatively heterogeneous, cross-sectional samples of property sales, ordinary least squares (OLS) with heteroskedastic-consistent standard errors will be used to obtain the model parameters.

Data

The Minnesota Department of Revenue (DoR) maintains data on all property transactions within the state. These data are reported by the counties and assembled into a larger, statewide database. For the present study, sales data have been collected from Olmsted County for the years 2000 through 2007. Attributes of each property listed in the data set include the property sale price, city and county of sale, indicators for the type of water features on each parcel (lakes, rivers, swamps, etc.), total and tillable acreage and an assessment of its value, as well as several other attributes.

Residential Sales

The property sales data are available for the period from October 1999 to September 2007, with a total of more than 38,000 property transactions recorded during this period. Of the 38,000 records, about 26,000 are residential, providing a potentially large sample for estimation. Parcel shapefiles were obtained from Olmsted County in order to map the geographic location of the parcels. Along with the necessary parcel data, additional building characteristics were collected from the county's property records division, providing information on important attributes such as square footage, number of bedrooms and bathrooms, and heating/cooling systems. The property sales files were first joined to the parcel data, then to the building characteristics. The process of joining the sales data to the parcel files resulted in the loss of a large number of records, including all of the 1999 records and most of the 2000 records. About 15,100 residential sales records were successfully joined. The second step, joining the building characteristics, resulted in the loss of about 150 additional records. Finally, some cleaning was done to the data, in order to try to identify sales that represented errors or non-arms-length transactions. In all, about 60 additional records were removed from the sample. The final sample that was used for estimation contained 14,900 observations.

Figure 2 displays the location of the residential property sales in Olmsted County. It is apparent from the map that most of the sales in the county during this period are clustered around the city of Rochester. The larger number of sales causes the location of some observations to be obscured. To provide more detail, Figure 3 centers the map view on the city of Rochester and identifies the reconstructed section of Highway 52, along with a set of buffer rings around the reconstructed highway at one-fourth mile intervals.

Our data set is divided into three periods, organized around the period coinciding with the major construction work on the ROC 52 project. A pre-construction period is comprised of sales occurring prior to April 2003. Sales from between April 2003 and September 2005 are identified as construction period observations, and any sales following this period are considered post-construction observations. We then created variables that designate the location of the property relative to the upgraded section of Highway 52 and also identify the period of sale. Thus, we can identify whether the effect of the location of property relative to the highway changes over time during the three periods of study.

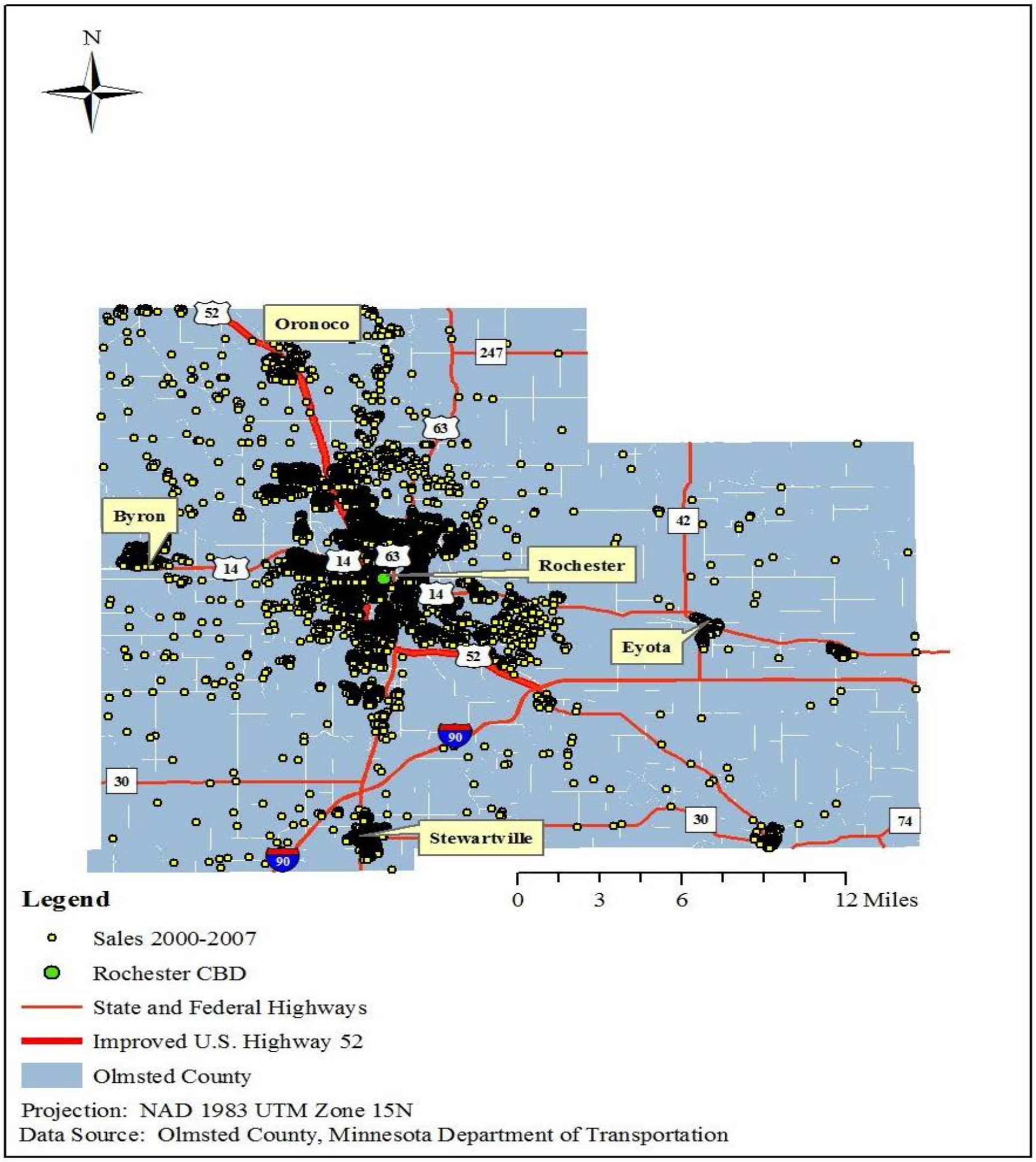
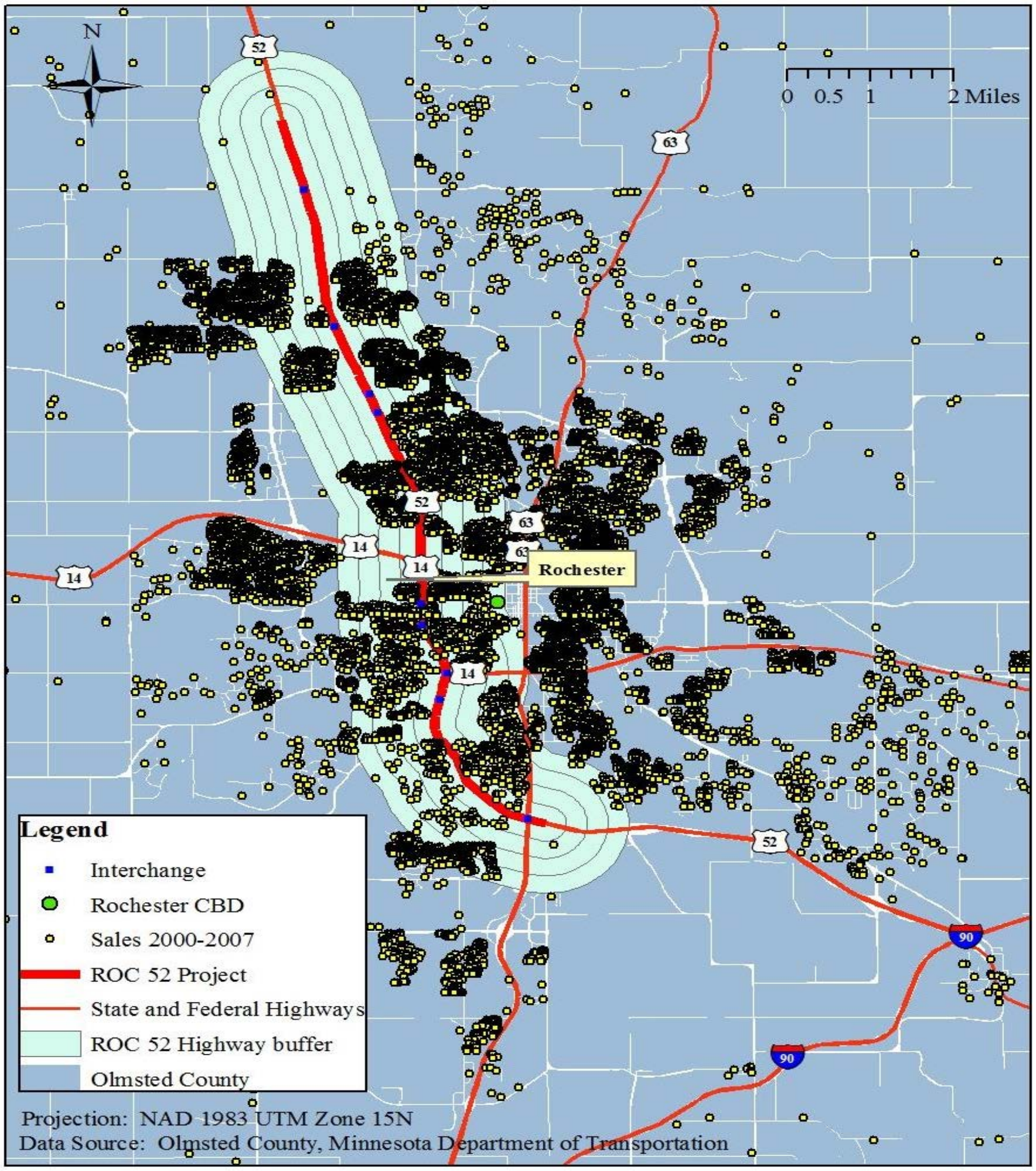

N
52
Oronoco
247
63
Byron
14
63
Rochester
14
42
Eyota
52
90
30
30
74
Stewartville
90
0
3
6
12 Miles
Legend
Sales 2000-2007
Rochester CBD
State and Federal Highways
Improved U.S. Highway 52
Olmsted County
Projection: NAD 1983 UTM Zone 15N
Data Source: Olmsted County, Minnesota Department of Transportation

Figure 2. Location of residential property sales in Olmsted County, 2000–2007

We also considered the possibility that proximity to the highway may generate both positive and negative externalities. Other hedonic price modeling applications in the field of transportation, primarily those concerned with the effect of proximity to rail transit stations, have attempted to separate the positive effects of access to the improved network (e.g., stations) from the nuisance effects that the network infrastructure itself generates (e.g., noise, pollution) (Chen et al. 1998, Goetz et al. 2009, Hess and Almeida 2007). To operationalize this concept, we kept the variables representing sales within various distance bands of the improved highway to serve as proxies for the nuisance effects of the highway. We also created new variables that measure network distance to the nearest access point (interchange) on the improved section of Highway 52, essentially a measure of local accessibility to the upgraded highway. This variable is also split into temporal intervals, coinciding with the pre-, post-, and under construction periods of the ROC 52 project, to determine if the value of highway access changes over time. Thus, the marginal effect of the highway improvement is the net effect of the positive and negative externalities (access versus nuisance effects). Table 1 provides a list of the variables used in the analysis of residential property sales. In addition to those listed in the table, we also included dummy variables for the month and year of sale. The month of sale variables use January as the reference category. The year-specific indicators are defined for 2001 through 2007, leaving the period from October 2000 to the beginning of 2001 as the point of reference. Also of note, a variable is defined representing distance to the central business distance district (CBD) of Rochester. This variable is a proxy measure for regional employment accessibility, as more disaggregate measures were not available. The CBD distance measure is seen as an acceptable proxy, as most of Rochester's major employers, including the Mayo Clinic, are located there. A set of descriptive statistics for the residential property sales data is provided in Table 2.


N
0 0.5 1 2 Miles
52
63
14
90
Rochester
Legend
Interchange
Rochester CBD
Sales 2000-2007
ROC 52 Project
State and Federal Highways
ROC 52 Highway buffer
Olmsted County

Projection: NAD 1983 UTM Zone 15N
Data Source: Olmsted County, Minnesota Department of Transportation

Figure 3. Location of ROC 52 project and residential property sales in Rochester, 2000–2007

Table 1. List of variables included in Olmsted County residential sales model

Variable Name	Description
ln SalePrice	Natural logarithm of sale price
Bedrooms	Number of bedrooms
Bathrooms	Number of bathrooms
BedBath	Bedrooms * Bathrooms
Age	Age of house
AgeSq	Age of house squared
FinishedSqFt	Square ft of house
AirCond	Dummy variable representing houses with air conditioning
River	Dummy variable representing house with river frontage
Condo	Dummy variable denoting housing unit as a condominium
TillAcre	Tillable acres of land
NTAcre	Non-tillable acres of land
CBDdist	Distance from Rochester CBD
Byron	Dummy variable for houses in city of Byron
2001	Dummy variable representing sale in year 2001
2002	Dummy variable representing sale in year 2002
2003	Dummy variable representing sale in year 2003
2004	Dummy variable representing sale in year 2004
2005	Dummy variable representing sale in year 2005
2006	Dummy variable representing sale in year 2006
2007	Dummy variable representing sale in year 2007
Feb	Dummy variable representing sale in month of February
March	Dummy variable representing sale in month of March
April	Dummy variable representing sale in month of April
May	Dummy variable representing sale in month of May
June	Dummy variable representing sale in month of June
July	Dummy variable representing sale in month of July
August	Dummy variable representing sale in month of August
September	Dummy variable representing sale in month of September
October	Dummy variable representing sale in month of October
November	Dummy variable representing sale in month of November
December	Dummy variable representing sale in month of December
1/4Mile	Dummy variable for location within 1/4 mile of upgraded highway
1/2Mile	Dummy variable for location within 1/2 mile of upgraded highway
3/4Mile	Dummy variable for location within 3/4 mile of upgraded highway
Mile	Dummy variable for location within 1 mile of upgraded highway
1/4Mile01	1/4Mile * 2001
1/4Mile02	1/4Mile * 2002

Table 2. List of variables included in Olmsted County residential sales model (continued)

1/4Mile03	1/4Mile * 2003
1/4Mile04	1/4Mile * 2004
1/4Mile05	1/4Mile * 2005
1/4Mile06	1/4Mile * 2006
1/4Mile07	1/4Mile * 2007
1/2Mile01	1/2Mile * 2001
1/2Mile02	1/2Mile * 2002
1/2Mile03	1/2Mile * 2003
1/2Mile04	1/2Mile * 2004
1/2Mile05	1/2Mile * 2005
1/2Mile06	1/2Mile * 2006
1/2Mile07	1/2Mile * 2007
3/4Mile01	3/4Mile * 2001
3/4Mile02	3/4Mile * 2002
3/4Mile03	3/4Mile * 2003
3/4Mile04	3/4Mile * 2004
3/4Mile05	3/4Mile * 2005
3/4Mile06	3/4Mile * 2006
3/4Mile07	3/4Mile * 2007
Mile01	Mile * 2001
Mile02	Mile * 2002
Mile03	Mile * 2003
Mile04	Mile * 2004
Mile05	Mile * 2005
Mile06	Mile * 2006
Mile07	Mile * 2007

Table 3. Descriptive statistics for Olmsted County residential property sales data

Variable	Mean	S.D.	Median	Min	Max
ln SalePrice	12.027	0.468	11.967	9.210	16.244
Bedrooms	1.855	1.643	2	0	11
Bathrooms	1.486	1.231	2	0	9
BedBath	4.217	4.850	3	0	50
Age	31	28	22	1	149
AgeSq	1761	2841	484	1	22201
FinishedSqFt	1630	575	1472	70	12432
AirCond	0.807	0.395	1	0	1
River	0.001	0.028	0	0	1
Condo	0.007	0.085	0	0	1
TillAcre	0.040	1.083	0	0	71
NTAcre	0.503	2.504	0	0	234
CBDdist	4.168	3.505	3.070	0.142	20.087
Byron	0.040	0.196	0	0	1
2001	0.099	0.299	0	0	1
2002	0.029	0.167	0	0	1
2003	0.148	0.355	0	0	1
2004	0.161	0.368	0	0	1
2005	0.208	0.406	0	0	1
2006	0.193	0.395	0	0	1
2007	0.140	0.347	0	0	1
Feb	0.054	0.226	0	0	1
March	0.074	0.262	0	0	1
April	0.084	0.278	0	0	1
May	0.112	0.315	0	0	1
June	0.146	0.353	0	0	1
July	0.100	0.299	0	0	1
August	0.103	0.304	0	0	1
September	0.081	0.273	0	0	1
October	0.078	0.268	0	0	1
November	0.067	0.251	0	0	1
December	0.058	0.235	0	0	1
1/4Mile	0.064	0.245	0	0	1
1/2Mile	0.109	0.312	0	0	1
3/4Mile	0.109	0.311	0	0	1
Mile	0.096	0.295	0	0	1
1/4Mile01	0.007	0.082	0	0	1
1/4Mile02	0.001	0.037	0	0	1

Table 4. Descriptive statistics for Olmsted County residential property sales data (continued)

1/4Mile03	0.010	0.101	0	0	1
1/4Mile04	0.011	0.103	0	0	1
1/4Mile05	0.012	0.111	0	0	1
1/4Mile06	0.012	0.110	0	0	1
1/4Mile07	0.009	0.092	0	0	1
1/2Mile01	0.010	0.100	0	0	1
1/2Mile02	0.004	0.060	0	0	1
1/2Mile03	0.017	0.130	0	0	1
1/2Mile04	0.019	0.137	0	0	1
1/2Mile05	0.020	0.141	0	0	1
1/2Mile06	0.020	0.141	0	0	1
1/2Mile07	0.016	0.125	0	0	1
3/4Mile01	0.012	0.107	0	0	1
3/4Mile02	0.003	0.057	0	0	1
3/4Mile03	0.015	0.123	0	0	1
3/4Mile04	0.019	0.136	0	0	1
3/4Mile05	0.022	0.147	0	0	1
3/4Mile06	0.021	0.144	0	0	1
3/4Mile07	0.014	0.118	0	0	1
Mile01	0.011	0.103	0	0	1
Mile02	0.003	0.055	0	0	1
Mile03	0.013	0.115	0	0	1
Mile04	0.015	0.123	0	0	1
Mile05	0.022	0.146	0	0	1
Mile06	0.018	0.132	0	0	1
Mile07	0.012	0.110	0	0	1

Commercial-Industrial Property Sales

Between 2000 and 2007, over 1,200 commercial and industrial property sales were recorded in Olmsted County—enough to permit a small-scale analysis of the impact of the ROC 52 project. As with the residential property data, the commercial-industrial sales data needed to be first mapped and then joined to data on building characteristics. The process of matching the sales data to the county's parcel records resulted in a loss of about half of the transactions, leaving 647 observations. Joining these data to a set of building attributes resulted in a loss of an additional 145 records. Finally, the data were cleaned to weed out non-arms-length transactions, leaving a total of 471 observations for the analysis. The location of these properties, along with the highway network, is mapped in Figure 4.

The set of attributes of the commercial-industrial properties that could be used to predict property values were somewhat limited, though important features such as building size and age were included. More general location variables were developed, measuring distance from the CBD as well as distance from the nearest highway. Parcel acreage was measured, and was divided into urban and rural acreage. Year-specific dummy variables were again added to attempt to measure any secular trends in prices during the period of observation. Most of the variables used to model commercial-industrial property prices are in fact a subset of the variables used in the analysis of residential property sales.

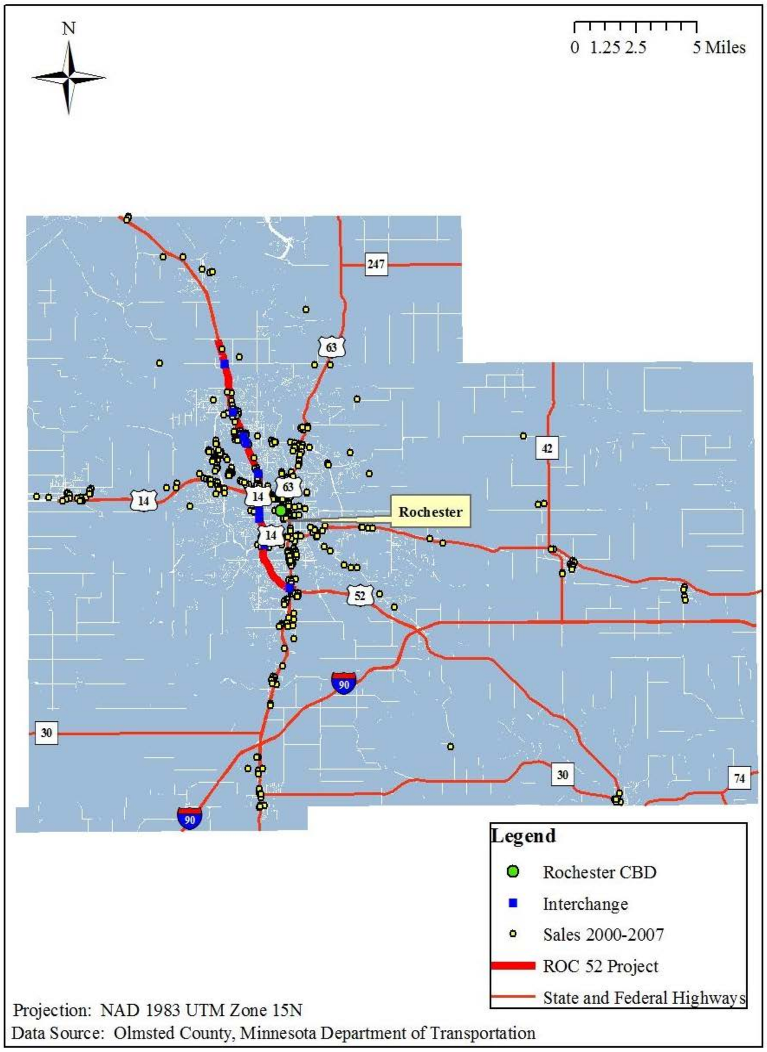

N
0 1.25 2.5 5 Miles
247
63
42
14
63
Rochester
14
14
52
90
30
30
74
90
Legend
Rochester CBD
Interchange
Sales 2000-2007
ROC 52 Project
State and Federal Highways
Projection: NAD 1983 UTM Zone 15N
Data Source: Olmsted County, Minnesota Department of Transportation

Figure 4. Location of commercial-industrial property sales in Olmsted County, 2000–2007

The effects of the upgrade of Highway 52 were measured by defining a variable similar to that used in the residential property analysis, in which network distance to the nearest access point on the improved section of highway is measured during specific time periods. The reasons for doing so were basically twofold. First, there was little reason to believe that externalities from highway traffic would have the same effect on commercial and industrial properties as on residential properties. Second, the smaller sample size for the commercial-industrial properties made difficult the method of identifying distance bands around the improved highway, since the number of observations in each location during each specific period were not consistently large enough to permit valid statistical inference. Instead, a continuous approximation is used to represent the relationship between proximity to the improved highway and property values. Since another variable is included in the model accounting for the distance to the nearest highway for all properties in the sample, the distance variable that is specific to the ROC 52 project should be seen as capturing the presence of any premium that is associated solely with the effect of this project.

RESULTS

Residential Properties

Results of the fitted model for the residential property sales data are presented in Table 3. The fitted model explains more than two-thirds of the variation in residential property prices using a limited set of structural attributes, some variables representing location and amenities, and the transportation attributes of interest. The coefficient on the bedroom variable is negative indicating that, controlling for the square footage of a residential unit, an additional bedroom has no value, though it should be noted that the estimated coefficient is small and statistically insignificant. The bathroom variable is significant, with an additional bathroom adding about 2.8% to the value of a house.

Table 5. Hedonic price model for residential property sales in Olmsted County, 2000–2007

Variable	Coefficient	S.D.	t-value	Sig.
Bedrooms	-0.029	0.006	-4.96	***
Bathrooms	0.014	0.005	2.71	***
BedBath	0.013	0.002	5.44	***
Age	-0.006	0.0004	-13.11	***
AgeSq	0.0000168	0.00000385	4.36	***
FinishedSqFt	0.0005	0.00002	20.06	***
AirCond	0.063	0.008	7.57	***
River	0.326	0.100	3.25	***
Condo	-0.150	0.042	-3.62	***
TillAcre	0.007	0.009	0.71	
NTAcre	0.011	0.019	0.57	
CBDdist	-0.009	0.002	-4.36	***
Byron	0.023	0.016	1.43	
2001	-0.010	0.020	-0.48	
2002	0.049	0.025	1.94	*
2003	0.010	0.019	5.23	***
2004	0.132	0.020	6.62	***
2005	0.163	0.019	8.64	***
2006	0.179	0.019	9.59	***
2007	0.171	0.020	8.72	***
Feb	0.034	0.015	2.20	**
March	0.015	0.015	0.98	
April	0.041	0.014	2.92	***
May	0.053	0.014	3.92	***
June	0.061	0.013	4.58	***
July	0.057	0.014	4.01	***
August	0.056	0.014	3.96	***
September	0.034	0.015	2.32	**

Table 6. Hedonic price model for residential property sales in Olmsted County, 2000–2007 (continued)

October	0.050	0.015	3.44	***
November	0.038	0.016	2.33	**
December	0.056	0.015	3.72	***
1/4Mile01	0.018	0.026	0.68	
1/4Mile02	-0.052	0.090	-0.58	
1/4Mile03	-0.056	0.020	-2.86	***
1/4Mile04	0.012	0.019	0.63	
1/4Mile05	-0.010	0.020	-0.51	
1/4Mile06	-0.013	0.016	-0.81	
1/4Mile07	0.052	0.031	1.69	*
1/2Mile01	0.028	0.021	1.37	
1/2Mile02	0.061	0.033	1.83	*
1/2Mile03	-0.001	0.017	-0.04	
1/2Mile04	-0.010	0.016	-0.61	
1/2Mile05	-0.008	0.014	-0.55	
1/2Mile06	0.016	0.015	1.04	
1/2Mile07	0.025	0.017	1.48	
3/4Mile01	0.034	0.020	1.75	*
3/4Mile02	0.052	0.038	1.36	
3/4Mile03	0.005	0.016	0.33	
3/4Mile04	-0.0004	0.019	-0.02	
3/4Mile05	0.003	0.014	0.21	
3/4Mile06	0.010	0.015	0.66	
3/4Mile07	0.045	0.018	2.51	**
Mile01	0.003	0.027	0.11	
Mile02	0.017	0.033	0.52	
Mile03	0.016	0.017	0.93	
Mile04	-0.017	0.019	-0.89	
Mile05	-0.024	0.012	-2.07	**
Mile06	0.004	0.018	0.20	
Mile07	-0.034	0.028	-1.24	
Constant	11.176	0.043	259.61	***
N = 14,900				
Adjusted R^2	0.682			

Notes:

Dependent variable is the natural logarithm of SALEPRICE

* = variable is statistically significant at $p < 0.1$ level

** = variable is statistically significant at $p < 0.05$ level

*** = variable is statistically significant at $p < 0.01$ level

Both the age and age squared variables are significant, indicating that the desirability of a house (as indicated by its selling price) declines with age, though the rate of decline decreases as age increases. The square footage variable, which is used here largely as a statistical control, has a coefficient of 0.0005. This may be interpreted to mean that a 100 square foot increase in the floor space of a house is associated with a 5% increase in its value. The presence of air conditioning is also estimated to add about 6% to the value of a house. Properties identified as condominiums sell for about 15% less than comparable detached units.

The coefficients on the land acreage variables have the expected sign, but appear not to be significant. River frontage does appear to have a significant effect, with homes with river frontage selling for about 30% more than homes without. Location relative to the Rochester CBD also has a significant effect, with each additional mile from the CBD being associated with a 1% decline in the price of a house.

Variables representing month and year of sale are also significant. The month dummies (which are suppressed from Table 3) are all statistically significant with the exception of March. The coefficients exhibit a pattern of increases during the warmer months of the year, with a peak during summer. The year dummies for 2001 through 2007 trace out the upward trend in home prices in Olmsted County throughout the first half of the decade. Prices in 2006 were, on average, nearly 21% higher than in 2000, controlling for all of the variables entered into the current model.

The effects of the upgrade of Highway 52 are reflected in the coefficients of the variables representing time and location, as well as the set of variables measuring access distance to the improved highway during the pre-construction, construction, and post-construction periods. Figure 5 plots the effects of proximity to the improved highway over time, as measured by the dummy variables denoting distance from the highway during specific time periods.

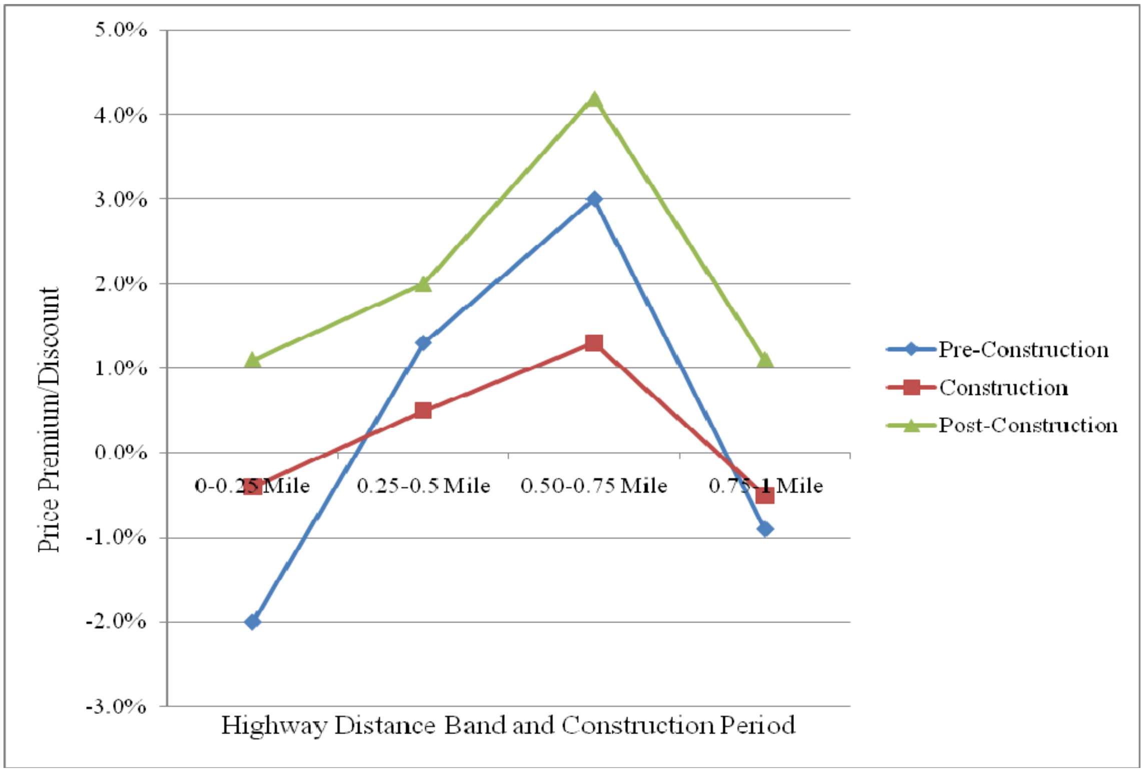


Figure 5. Price effects of proximity of residential properties to upgraded U.S. Highway 52

The set of points representing various distances from the improved highway during each time period trace out a rough price gradient for highway proximity. As the figure indicates, houses closest to the highway sold for slightly less than those not near the highway during the pre-construction and construction periods. During the post-construction period, they sold for slightly more (around 1%). Houses three-quarters of a mile from the improved highway appear to obtain a slight premium during *all* periods, with the largest premium occurring during the post-construction period. In order to attempt to sort out the effects of access to the improved highway, the separate variables representing distance to the nearest highway access point are included. The coefficients on these variables were expected to be negative, indicating that some premium would be placed on having access to the improved highway nearby. As Table 3 indicates, the coefficient representing access distance during the pre-construction period is negative, though very small and not statistically different from zero at the $p > 0.1$ level. The coefficients representing access during the period of major construction and post-construction are both slightly positive, though also statistically insignificant.

Overall, we were unable to detect any premium associated with being located near an access point to the improved highway. Conversely, the dummy variables used to represent proximity to the highway itself do show a slight positive effect at certain distances (0.5 to 0.75 miles). These findings seem to suggest that, at least for residential properties, nuisance effects of being near a highway interact with the effect of the access that the highway provides in subtle ways. This result should, however, be qualified by noting that in each case the magnitude of the effect of the improved highway (whether positive or negative) was quite small, and that only a handful of the variables representing the effects of the highway improvement showed statistically significant (non-zero) effects.

Commercial-Industrial Properties

The model fitted to the Olmsted County commercial-industrial data is shown in Table 4. The coefficient on the square footage variable indicates that each additional 1,000 prime square ft of space add about 1.5% to the price of a commercial-industrial property. Building age is also significant, with each additional year of age associated with a 1% decline in price. The value of commercial-industrial land is indicated by the coefficient estimates for the two acreage variables. An additional acre of urban land adds about 17% to the value of a property, while an acre of rural land (identified as being outside an incorporated town) adds about 2%. Distance from the Rochester CBD appears to be a significant factor in explaining commercial property values, as it is for residential properties. Here, we find that each additional mile from the CBD is associated with a roughly 5% decline in value. Of note, this price gradient appears to be much steeper than the one estimated for residential properties (about 1% for each mile from the CBD).

The variable representing distance to the nearest highway appears to have a rather large influence on property values. On average, property values fall by more than 36% for each additional mile from the nearest highway. This finding appears to underscore the importance of highway access for commercial and industrial properties, a finding that is also readily apparent from the location of these properties in Figure 4. Beyond this effect, the variables representing proximity to access points on the reconstructed section of Highway 52 also appear to be significant. The variable representing highway access during the pre-construction period indicates that for every mile of distance from the nearest access point on the rebuilt Highway 52, property values fall by about 2.5%. This is *in addition to* the more general effect of proximity to highways for all properties in Olmsted County. The variables representing access distance during the construction and post-construction periods have the same sign but a smaller coefficient, indicating that the distance gradient for access to the improved highway may have flattened out over time, with the effect of the improved highway possibly being present at further distances from access points following completion of the ROC 52 project. On one hand, this may be evidence of a real, accessibility-related improvement due to the reconstruction project. On the other hand, the estimated standard errors for each of the three coefficients on the access variables are large enough that we may not rule out the possibility that there is no real difference between the true values of the three coefficient estimates, and that the differences observed in our model are due to chance variation. Nonetheless, our evidence suggests that the effect of the ROC 52 access distance variable is non-zero, meaning that the project resulted in at least some increment in property values for commercial and industrial properties.

CONCLUSIONS

In this paper we have examined the effects of a major highway reconstruction and expansion project on residential and commercial-industrial property values in Olmsted County, Minnesota. Using a set of property sales data from periods before, during, and after the major construction took place, we found tentative evidence that, following an initial decline in prices during construction, residential properties within one mile of the improved Highway 52 saw a small increase (less than 2%) in sale price during the post-construction period. Our examination of commercial-industrial property sales from the same period (2000–2007) revealed no unique, statistically significant effect on prices that could be attributed to the completion of the ROC 52 project. However, our analysis did indicate that, in general, highway access is highly valued among commercial and industrial property owners.

Our analysis revealed some small, yet positive effects on property values in response to a highway reconstruction and expansion project. In general, studies of new transportation links such as highway or urban rail links tend to find larger increments in property values near the new facility. The presence of this price effect provides an opportunity for local governments or transportation authorities to capture a portion of this increment in value, a practice known as *value capture* (Batt 2001, Stopher 1993). Value capture policies may be a particularly attractive alternative for transportation finance in fast-growing locations, where increases in the demand for travel outstrip the resources available from conventional sources (e.g., fuel or property taxes, etc.) to finance infrastructure improvements (Vadali et al. 2009).

Several types of value capture policies exist that may be applied in the case of highway network improvements. These range from policies that capture the value associated with development on top of a link (e.g., sale of air rights) to policies that attempt to recover a portion of land value increases within a geographically-defined area near an improved transportation link. The latter include policies such as special assessments, tax increment financing, and impact fees. In the United States, there is some recent experience with the use of impact fees on new highway corridors to draw upon (Boarnet and DiMento 2004).

Value capture policies hold promise for improving the equity with which transportation is financed. In particular, they target a restricted group of non-user beneficiaries from investments in transportation networks that under current methods of transportation finance receive benefits that are disproportionately greater than the costs they bear. New transportation projects may generate accessibility benefits that impart windfall gains on owners of nearby property. The use of value capture techniques as one component of financing plans for transportation projects helps to level this playing field by reallocating costs to align more closely with the benefits received across a wider set of beneficiaries.

ACKNOWLEDGMENTS

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Development of Updated Guidelines and Specifications for Roadway Rehabilitation

Charles T. Jahren
Department of Civil, Construction, and Environmental Engineering
Iowa State University
428 Town Engineering
Ames, IA 50011-3232
cjahren@iastaste.edu

Paul Wiegand
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
pwiegand@iastate.edu

Larry J. Stevens
SUDAS
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
lstevens@iastate.edu

Ryan Shropshire
Department of Civil, Construction, and Environmental Engineering
Iowa State University
428 Town Engineering
Ames, IA 50011-3232
rashropshire@gmail.com

ABSTRACT

This presentation will describe an investigation that is reviewing Iowa Department of Transportation (Iowa DOT) and SUDAS guidelines and specifications for roadway rehabilitation techniques and provide recommendations for improvement (Iowa Highway Research Board Project 08-01). In the process, investigators have surveyed practitioners to determine in which areas guidance is most valuable. Furthermore, investigations have reviewed specifications and guidelines from several sources to identify best practices for possible inclusion in Iowa DOT and SUDAS documents. The review included

- Current Iowa DOT and SUDAS Documents
- Neighboring states
- Neighboring local jurisdictions
- National Trade and Professional Associations
- Federal Highway Administration
- Academic and professional literature

Based on the findings of these tasks, investigators will recommend improvements to the target documents. In the proposed presentation, investigators will share findings regarding:

- The type of guidance that was found to be most valuable to practitioners,
- Best practices identified during the document review process, and
- Preliminary recommendations regarding areas for improvement of Iowa DOT and SUDAS documents.

During the presentation, audience members will be requested to give input on current recommended guidelines in accordance with the research plan for this project.

Key words: Iowa DOT—roadway rehabilitation techniques—SUDAS

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Stabilization Procedures to Mitigate Edge Rutting for Granular Shoulders

Charles T. Jahren
Department of Civil, Construction, and Environmental Engineering
Iowa State University
428 Town Engineering
Ames, IA 50011-3232
cjahren@iastaste.edu

David J. White
Earthworks Engineering Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
djwhite@iastate.edu

Thang H. Phan
Department of Civil, Construction, and Environmental Engineering
Iowa State University
394 Town Engineering
Ames, IA 50011-3232
thphan@iastate.edu

Chase Westercamp
Department of Civil, Construction, and Environmental Engineering
Iowa State University
Town Engineering
Ames, IA 50011-3232
cwestercamp@gmail.com

Peter Becker
Department of Civil, Construction, and Environmental Engineering
Iowa State University
394 Town Engineering
Ames, IA 50011-3232
pbecker@iastate.edu

ABSTRACT

An investigation is being conducted to develop stabilization procedures that will mitigate edge rutting for granular shoulders (Iowa Highway Research Board Project [TR-591]). Its objectives are as follows:

1. Determine the relative importance of localized, chronic edge rut issues compared to longer reaches of roadway with more general shoulder edge rut maintenance issues.
2. Develop a series of strategies for mitigating edge rut problems using various mixtures and gradations of granular materials and various stabilization agents.
3. Rate the performance of a subset of the abovementioned strategies by observing test sections.
4. Recommend strategies based on the results of test section performance, cost, and probable future maintenance procedures.
5. Assist the Iowa Department of Transportation in implementing the use of the recommended strategies.

Currently, investigators are executing tasks related to Objective 3. The proposed presentation will provide a progress report on the investigation.

Currently, two sets of test sections have been constructed: one on US 20 east of Waterloo, Iowa, and another on US 75 north of Sioux Center, Iowa. The granular shoulders on these high-volume roads develop edge ruts quickly and serve as a good test location for edge rut mitigation strategies. The US 20

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test sections were constructed in October 2008 and included calcium chloride, magnesium chloride, and sodium silicate. The US 75 test sections were construction using Dust Lock®, a soybean-oil–based dust suppressant. Investigators will describe the process of selecting stabilizing agents for testing, selecting dosage and application procedures, and current performance.

Key words: edge rutting—granular shoulders—mitigation

Identification of Practices, Design, Construction, and Repair Using Trenchless Technology

Charles T. Jahren
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
428 Town Engineering
Ames, IA 50011-3232
cjahren@iastatate.edu

Muhannad T. Suleiman
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
Town Engineering
Ames, IA 50011-3232
suleiman@iastate.edu

Halil Ceylan
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
482B Town Engineering
Ames, IA 50011-3232
hceylan@iastate.edu

Larry J. Stevens
SUDAS
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
lstevens@iastate.edu

Bill Conway
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
Town Engineering
Ames, IA 50011-3232
william.m.conway@gmail.com

ABSTRACT

An investigation (Iowa Highway Research Board Project TR-570) is being conducted to

1. Document the current practices and applications of trenchless technology in the United States and in Iowa, in particular, and
2. Evaluate the effects of trenchless construction on surrounding soil and adjacent structures.

Researchers visited more than 10 projects involving trenchless installations and documented successes and challenges in each one. Field visits included sites with horizontal directional drilling, auger boring, tunneling, and mole installations. In addition, at six sites, researchers conducted field tests to measure changes in lateral earth pressure that occur during trenchless installations. Such measurements are relevant because there is concern that such changes in earth pressure may cause pavements to heave or settle as a result of the trenchless installation process. Field measurements were conducted at horizontal directional drilling and mole installations.

Researchers will present a summary of findings from the field visits and field measurements. Possible types of transportation facility damage that may occur from these installations will be discussed as well as possible quality control techniques that may reduce the likelihood of damage. Also, findings regarding lateral earth pressure changes during installation will be discussed.

Key words: field tests—installation—trenchless technology

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Access Management Considerations for Roundabout Design and Implementation

Mark T. Johnson
MTJ Engineering, LLC
313 Price Place #205
Madison, WI 53703
mtjeng@sbcglobal.net

Hillary N. Isebrands
Department of Civil, Construction, and Environmental Engineering
Iowa State University
P.O. Box 5734
Eagle, CO 81631
hillary.isebrands@gmail.com

ABSTRACT

Roundabouts are compatible with many access management principles. The operational characteristics of roundabouts differ from signalized intersections in many substantial ways. Roundabouts allow for more flexibility in design that can be of significant benefit when balancing the competing objectives of roadway safety, capacity, and the access needs of existing and/or proposed land uses. The literature and case studies are limited when it comes to roundabouts and access management principles. This paper explores case studies of the different ways in which roundabouts can provide flexibility in access management. It specifically addresses business access into and near roundabouts, roundabouts in series, and other access management issues compatible with roundabouts in redevelopment, new development, and constrained urban environments.

Key words: access management—roundabouts

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Iowa's Multi-Level Linear Referencing System

Steven Kadolph
Research and Technology Bureau
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
steven.kadolph@dot.iowa.gov

Ryan Wyllie
Office of Transportation Data
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Ryan.Wyllie@dot.iowa.gov

ABSTRACT

The Iowa Department of Transportation's Linear Referencing System (LRS) has been recognized as an advance in transportation technology by the American Association of State Highway and Transportation Officials' Technology Innovation Group. Iowa is in the first year of a three technology innovation grant to assist states in implementation of the NCHRP 20-27(2) LRS model.

The purpose of an LRS is to accurately place business data located by a Linear Referencing Method (LRM) on a cartographic representation of a transportation feature. Examples of LRMs are milepoint, milepost, coordinate route, and literal description. Iowa's LRS consists of a spatial accurate centerline, temporal location of transportation systems, datum objects, a network layer, and routes.

The PowerPoint presentation will discuss:

1. What is the LRS
    a. Conceptual model
    b. Iowa's implementation
2. Why did Iowa develop the system
    a. Business data integration
    b. Improved accuracy in both space and time
3. What Iowa gained from its development
    a. Centralized location component for transportation system location
    b. Navigable network
4. Lessons learned
    a. What we did right
    b. What we might do differently
5. Questions

Iowa has made multiple presentations at the Geographic Information System (GIS) for Transportation Symposium since 2000, had webinars with the Illinois Tollway Authority and the Montana Department of Transportation, and was visited by the Minnesota Department of Transportation.

Key words: asset management—data needs—GIS applications—integration

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Financing Road Projects in India Using PPP Scheme

Satyanarayana N. Kalidindi
Building Technology and Construction Management Division
Department of Civil Engineering
Indian Institute of Technology Madras
Chennai – 600 036, India
satyakn@iitm.ac.in

L. Boeing Singh
Building Technology and Construction Management Division
Department of Civil Engineering
Indian Institute of Technology Madras
Chennai – 600 036, India
boeing.laishram@gmail.com

ABSTRACT

India has one of the largest road networks in the world, aggregating around 3.3 km. Historically, the budgetary resources from the governments have been the major source of financing for infrastructure such as road projects in India. These investments have resulted in an increase in the length of the road network from 0.4 million km in 1950–51 to 3.3 million km in 1995–96. But the development of the road network has failed to keep pace with the growth in the traffic. The reduction of the budgetary allocation towards roadway upgrades on account of the competing demands from other sectors, such as social and economic infrastructure and the limitations in the traditional public procurement system, have resulted in deficiencies in the road network leading to capacity constraints, delay, congestion, fuel wastage, and higher vehicle operating costs.

In order to remove the deficiencies and upgrade the road network to world-class standards, the governments at the Union and state levels have initiated various measures. For instance, at the national level, the Union Government has introduced various structural reforms and fiscal incentives to promote private sector participation in the development of a National Highway network. The public-private partnership (PPP) models that have been used in procuring the National Highways projects include Build-Operate-Transfer (BOT) (Toll) and BOT (Annuity) models. Besides bringing efficiency gains in the implementation of the projects, the involvement of the private sector through the PPP route also facilitates private investments in the development of road projects. This paper focuses on the various approaches that have been used for financing of PPP road projects in India. The reforms, measures, and procurement strategies that have been initiated to enable financing through PPP route in view of the risk profile associated with such projects are also discussed. The degree of financing from the private sector depends on, *inter alia*, the risk profile and financial viability of the project. This in turn influences the selection of the type of PPP model that is considered most appropriate for the concerned project.

Keywords: BOT—infrastructure financing—PPP—project risks

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INTRODUCTION

Historically, budgetary resources from the governments have been the major source of financing for infrastructure such as road projects in India. Investments in road sector in the post-independence era have resulted in expansion of the road network from 0.4 million km in 1950–51 to 3.32 million km in 1995–96. In the corresponding period, kilometers of roads with the proper surface has increased from 0.156 million km to 1.517 million km (Planning Commission 1997). On the other hand, in the period 1950–51 to 1995–96 the number of passenger buses has gone up 13-fold from 34,000 to 450,000, while the goods vehicle fleet has increased 22-fold from 82,000 to 1,785,000.

In order to keep pace with the growth in traffic, the Central Government's Five Year Plans have emphasized the need for improvement in the road network and the need to overcome inadequacies in the roads. The seventh Five Year Plan (1985–89) indicated the need for further improvement in the road network as significant portions of the network were without proper surface and pavement width, as most of the roads were single lane (Planning Commission 1980). The report also mentioned that even in the case of National Highways, a significant portion of the roads were single lane. The eighth Five Year Plan (1989–91) also reiterated the need to overcome the problems of inadequate road pavement; and breadth, thickness, and presence of old, weak, and narrow bridges and culverts (Planning Commission 1992). The severe deficiencies in the road network and the growing mismatch between traffic needs and available infrastructure have resulted in severe capacity constraints, delay, congestion, fuel wastage, and higher vehicle operating costs.

The decline in the allocation of funds over various plan periods in terms of percentage of the total plan outlay has been identified as one of the factors partly responsible for the inadequacies in the road network. The lack of investment manifested itself in the form of non-replacement of overaged stock, slowing down of modernization, and inadequate attention to maintenance (Planning Commission 1992). Besides the budgetary constraints, the traditional public procurement system has serious weakness in planning and implementation of road projects leading to time and cost overruns. In order to augment resources, the Indian government has emphasized, starting from the seventh Five Year Plan (1985–89), the need to look for resources from nonconventional sources of funds and private sector participation in road sector (Planning Commission 1980).

The steady economic growth due to economic liberalization in the 1990s has resulted in high traffic growth with the highways becoming increasingly congested, thereby driving up the demand for improved road transport. The upgrade of the Indian road network to world-class standards has assumed immense importance in the post-liberalisation era, as the delay on the roads could result in high inventory costs, thus affecting India's competitiveness in the international market (Planning Commission 1997).

In the post-liberalization era, there has been a paradigm shift in the mode of procurement of infrastructure such as road projects in India. The Central Government and state governments have adopted a public-private partnership (PPP) route in place of a traditional public procurement process for development of limited stretches of the road network. One of the reasons governments are opting to use PPPs for the development of infrastructure is to use the skills, innovations, and managerial capability of the private sector to optimize efficiency in infrastructure projects. PPP arrangements are also employed by governments with the objective of using private financing to address the funding needs, in the light of the competing demands on budgetary resources from social and economic sectors. This paper focuses on the various approaches that have been used for financing the National Highways projects through the PPP route in India. The reforms, measures, and procurement strategies that have been initiated to enable financing through the PPP route in view of the risk profile associated with such projects are also discussed.

PPPS—CONCEPT AND MODELS

PPPs aim at financing, designing, implementing, and operating public sector facilities and services through partnerships between public agencies and private sector entities (UNECE 2008). One of the main reasons governments are opting to use PPPs for infrastructure development is to increase the efficiency of infrastructure projects through a long-term collaboration between the public sector and private business (Davies and Eustice 2005). Emphasizing efficiency gains from a PPP perspective, it is stated that the main consideration for public agencies for opting on PPPs should be ensuring monetary value. PPPs facilitate the project to be implemented on time and within budget. The "no service/no pay" principle ensures that the private partner is incentivized for timely delivery and operation of project assets. Better overall governance by private sector entities enables the private partner to have better control of cost overruns contrary to traditional public procurements which are often characterized by significant construction delays and cost overruns. On account of assigning life cycle maintenance obligations to the private sector, private partners are incentivized to optimize capital and maintenance expenses over the project duration. In short, by transferring responsibilities and risks to those best able to manage them under PPPs, overall cost of risk is reduced. This reduced cost of risk is the key means of delivering value for money to the public sector. In fact, in case of successful PPP projects on account of the reduced cost of risk, there is still monetary value in spite of the high cost of finance from private partners vis-à-vis public borrowing (EIB 2004).

PPPs, in the broadest sense, can cover all types of collaboration across the interface between the public and private sectors to deliver policies, services, and infrastructure. The term PPP refers to a wide range of arrangements with simple arrangement such as management contract on one extreme of the spectrum, while arrangements such as full privatization or divestiture remain on the other extreme of the spectrum. Various approaches are in use to classify the arrangements between the two extremes of the spectrum. One of the approaches is to refer to the wide variety of arrangements based on the involvement of the private and public sectors in the various phases of project life cycle (Pakkala 2002). However, the most common way of referring to the different arrangements is based on the extent to which the responsibilities and risks are transferred from public sector to private sector. Figure 1 shows the risk transfer continuum and the characteristics of the various PPP models. The risk transfer to the private sector increases as we move from maintenance management to divestiture (Hammami et al. 2006). Critical risks such as market risk are completely transferred to the private sector in PPP models such as BOT and divestiture.

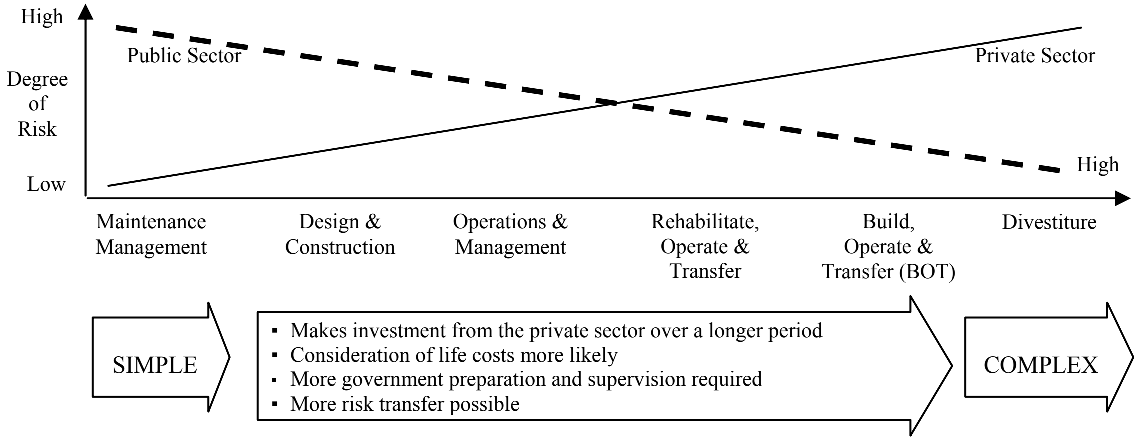
High
Degree of Risk
Low
Public Sector
Private Sector
High
Maintenance Management
Design & Construction
Operations & Management
Rehabilitate, Operate & Transfer
Build, Operate & Transfer (BOT)
Divestiture
SIMPLE
▪ Makes investment from the private sector over a longer period
▪ Consideration of life costs more likely
▪ More government preparation and supervision required
▪ More risk transfer possible
COMPLEX

Figure 1. PPP models risk transfer continuum and their characteristics (adapted from ADB [2000] and World Bank [2004])

PPPS IN NATIONAL HIGHWAYS NETWORK

National highways are the arterial roads that run through the width and breadth of the country connecting state capitals, ports, industrial and tourist centers, and adjacent countries. The National Highways, with a total length of 65,659 km, account for just 2% of the 3.3 million km road network, but carry 40% of the total traffic (DoRTH 2007a). In spite of the fact that National Highways have played a key role in the economic growth of the country, the Central Government has not been able to allocate sufficient budgetary resources to meet roadway needs due to competing demands from other sectors, especially the social sector. The Government of India, which has jurisdiction over the National Highways network regarding its development and maintenance, has sought the involvement of the private sector through the PPP route to meet the galloping resource requirements and overcome the inefficiencies in the traditional public procurement system.

Involvement of the private sector in the development of road projects in the National Highways network are through PPP models such as Build-Operate-Transfer (BOT) and Design and Construction (or Design and Build/EPC) contract. BOT (Toll) and BOT (Annuity) are the two variants of the BOT model through which capital from the private sector is invested in the development of road projects. In a BOT (Toll) Model, the concessionaire (private sector) is required to meet the upfront/construction cost, operational cost, and the expenditure on annual and periodic maintenance. The concessionaire recovers the costs along with the interest and a return on investment out of the future toll collection. A capital grant is also provided in order to bridge the gap between the investment required and the gains arising out of it and increase the viability of the projects. With respect to the BOT (Annuity) Model, the concessionaire is required to meet the entire upfront/construction cost and the expenditure on annual maintenance. The concessionaire recovers the entire investment and a predetermined cost of return out of the annuities payable by the granting authority every year.

The Central Government of India has undertaken the ambitious National Highways Development Program (NHDP) to upgrade the National Highways in seven phases. The Government of India in January 1999 formally launched NHDP to develop the Golden Quadrilateral network (the National Highways network connecting the four metro cities of Mumbai, Chennai, Kolkata, and Delhi) under NHDP Phase I and north–south and east–west (NSEW) corridors under NHDP Phase II. The National Highways Authority of India (NHAI) was mandated to implement this program, which was estimated to cost 540 billion[1] Indian Rupee (INR) (in 1999 prices). NHAI planned private sector participation in certain stretches of the National Highways network under the NHDP project and anticipated private investments to the tune of INR 40 billion (in 1999 prices). NHAI involved the private sector in the NHDP projects through the two PPP models: BOT (Toll) and BOT (Annuity).

The scope of NHDP has been further expanded when the Government of India included five more phases (i.e., NHDP Phase III to NHDP Phase VII) to the program under the government's ambitious plan to upgrade the National Highways in a phased manner in the period 2007–2012 (see Table 1).

The Committee on Infrastructure, Government of India, estimated that investment to the tune of INR 2,272.58 billion, including INR 524.34 billion for completion of NHDP phases I and II, will be required to complete the program (DoRTH 2007a). The major portion of the investment is expected from the private sector since the government, as a matter of policy, has decided that all sub-projects in NHDP Phase III to Phase VII would be taken up on PPP basis, i.e., through BOT mode (Ministry of Finance

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[1] 1 US$ = INR 48.3 on July 20, 2009

2006). Implementation of projects through design-build contract will be only in exceptional cases, where private sector participation is not possible at all.

Table 1. National Highways Development Program for 2007–2012 (Adapted from DoRTH [2007b])

Name of Project	Likely Cost (in INR Billion)
Completion of GQ and EW-NS corridors	524.34
4-laning of 11,113 km under NHDP Phase-III	724.54
2-laning with paved shoulders of 20,000 km of National Highways under NHDP Phase-IV	27.8
6-laning of selected stretches of National Highways under NHDP Phase-V	412.10
Development of 1000 km of expressways under NHDP Phase-VI	166.80
Construction of ring roads, flyovers, and bypasses on selected stretches under NHDP Phase-VII	166.80
Total	2,272.58

ENABLING FRAMEWORK FOR PPPS IN NATIONAL HIGHWAYS

The Central Government of India has introduced various reforms and initiatives in order to create an enabling framework for private sector participation in development of National Highways. One of the important steps taken up by the Central Government in this direction was the constitution of NHAI with the enactment of National Highways Authority of India Act, 1988 (Ministry of Law and Justice 1988). NHAI, which was put into operation in February 1995, has been responsible for the development, maintenance, and operation of the National Highways. The Government of India has also reformed the legal framework and paved the way for private sector participation in development of National Highways with the amendment of the National Highway Act, 1956 in June 1995. This act has enabled private investors to levy toll and allowed participation in construction, maintenance, and operation of National Highways.

Since the Government of India decided in April 1995, to involve private sector in road development, several institutional reforms and fiscal incentives have been introduced besides the legal reforms to encourage private sector participation in the upgrade of the National Highways network. The key broad and road-sector–specific institutional reforms and the fiscal incentives introduced are highlighted below (DoRTH 2009):

1. Government to bear the cost of project feasibility study, land for the right-of-way and wayside amenities, shifting of utilities, environment clearance, cutting of trees, etc.
2. Foreign Direct Investment up to 100% in road sector
3. Provision of subsidy up to 40% of project cost to make projects viable; the quantum of subsidy will be decided on a case-by-case basis
4. 100% tax exemption in any consecutive 10 out of 20 years after commissioning of the project
5. Duty free import of high capacity and modern road construction equipment

6. Road sector has been accorded the status of an industry via Section 18 (1)(12) of the Infrastructure Act
7. Easier external commercial borrowing norms

As part of the initiative to encourage private sector participation, the Central Government of India has developed model concession agreements (MCAs) for PPPs in the road sector, such as MCAs for major road projects costing more than INR 1 billion undertaken under BOT (Toll) basis, MCAs for minor road projects costing less than INR 1 billion undertaken under BOT (Toll) basis, and MCAs for road projects undertaken under BOT (Annuity) route. Another model concession agreement was developed by Planning Commission, Government of India, for road projects taken up on a Design-Build-Finance-Operate (DBFO) basis. These standard concession agreements will facilitate standardization of terms and conditions and ensure uniformity in the various agreements for PPP road projects (Planning Commission 2002). In addition, the model concession agreements also spell out the precise policy and regulatory framework put in place for PPP road projects (Planning Commission 2006a). This framework addresses the issues that are typically important for limited recourse financing of infrastructure projects, such as mitigation and unbundling of risks, allocation of risks and rewards, symmetry of obligations between the principal parties, precision and predictability of costs and obligations, and force majeure and termination.

In addition, standard documents have also been formulated for the two-stage bidding process for PPP projects. The first stage is generally referred to as the Request for Qualification (RFQ) stage, which is to pre-qualify and short-list eligible bidders for stage two of the process (Ministry of Finance 2009). The second and final stage of the bidding process, which is generally referred to as the Request for Proposal (RFP) stage, is aimed at obtaining financial offers from pre-qualified bidders after the RFQ stage (Ministry of Finance 2007). Detailed guidelines for inviting applications for pre-qualification and short-listing of bidders, submission of financial offers, and criteria for selection of bidders are provided in the model RFQ and RFP documents. The Government of India has also created different guidelines for PPP projects of different project costs on the formulation, appraisal, and approval of the projects in order to ensure speedy appraisal of projects, and have uniformity in appraisal mechanism and guidelines (Department of Economic Affairs 2008a). Finally, a Public Private Partnership Appraisal Committee (comprising secretaries of Departments of Economic Affairs, Expenditure, Legal Affairs, Planning Commission, and the department sponsoring the project) has been set up, which will facilitate appraisal and approval of PPP projects of all sectors, including projects under NHDP where the capital costs are above INR 5 billion (Department of Economic Affairs 2005).

The Government of India has taken various initiatives to meet the unique financing needs of infrastructure projects. In order to provide long-term finance (debt or equity) to infrastructure projects, the Ministry of Finance, Government of India evolved the scheme for financing commercially viable infrastructure projects in various sectors such as roads, power, solid waste management, and water supply through a special purpose vehicle called the India Infrastructure Finance Company Limited (IIFCL) in 2006 (Planning Commission 2006b). A corpus fund titled India Infrastructure Project Development Fund (IIPDL), with an initial contribution of INR 1 billion has been set up to provide financial support to the state and central ministries for quality project development activities (Department of Economic Affairs 2008c). Finally, a Viability Gap Funding Scheme was launched in 2004 to meet the funding gap of economically essential projects and make it commercially viable for private sector participation (Department of Economic Affairs 2008b).

BOT VARIANTS USED IN NATIONAL HIGHWAYS NETWORK

Private sector participation in the development of road projects in the National Highways network takes place through the two variants of the BOT model: BOT (Toll) and BOT (Annuity). The risk allocation framework and the bidding process for these models are discussed in detail in the following sections.

BOT (Toll) Model

In the BOT (Toll) model, the commercial and technical risks relating to construction, operation, and maintenance of the projects are allocated to the concessionaire. The traffic revenue risk, which is one of the critical risks associated with PPP road projects in India, is also allocated to the concessionaire. The risk allocation framework as per MCA for the BOT (Toll) project developed by the Planning Commission is presented in Table 2.

Through a two-stage bidding process, the concession is awarded to the concessionaire. In the first stage (also known as qualification stage) bidders provide the information specified in the RFQ. The pre-qualified bidders are then invited to submit their bids. Bids could be invited for the project on the basis of the lowest financial grant that would be required for implementing the project. Instead of seeking a grant, the bidders could also offer to share the revenue or make upfront payment to the granting authority for award of the concession. The grant/revenue sharing constitutes the sole criterion for evaluation of the bids and the concession is awarded to the bidder quoting the highest revenue sharing. In the event that the bidder is not sharing the revenue, then the concession is awarded to the bidder seeking the lowest grant.

BOT (Annuity) Model

BOT (Annuity) is a traffic risk-neutral PPP model. In this PPP model, the concessionaire is selected through the two-stage bidding process. In the first stage, the interested parties are invited to furnish their technical and financial strength. The pre-qualified parties are then invited to submit the financial bid, which is the cost of construction, operation, and maintenance of the facilities, and a percentage of returns thereon quoted on a semi-annual basis throughout the concession period. The contract is awarded to the bidder with the lowest quote of the annuity. The granting authority pays the concessionaire annuities on each annuity payment date as per the annuity payment schedule, after adjusting for non-availability of the lane and delay or early achievement of commercial date.

In this PPP model, the concessionaire assumes risks relating to construction, technical, operation, and maintenance, while the other critical risks relating to land acquisition, permit/approval, traffic risk, and toll collection risk are allocated to the granting authority. The risk allocation framework for BOT (Annuity) projects as per the MCA for BOT (Annuity) is presented in Table 3.

Table 2. Risk allocation framework for BOT (toll) project

Risk	Allocated to	Remarks
Permit/ approval	Granting authority/ Concessionaire	Applicable permits relating to environmental protection and conservation of the site to be obtained by the granting authority, other applicable permits are to be obtained by the concessionaire
Delay in land acquisition	Granting authority	Granting authority shall pay damages calculated at Rs 50 per day for every 1,000 sq. m commencing from 91st day of the date of financial closure and until such right–of-way is procured
Delay in financial closure	Concessionaire	In case the financial closure does not happen within 300 days of signing concession agreement, then concession agreement shall be deemed to have been terminated by mutual agreement of the parties. The granting authority can encash the bid security.
Traffic revenue risk	Concessionaire	MCAs provide for extension of the concession period in the event of a lower than expected growth in traffic. Conversely, the concession period is proposed to be reduced if the traffic growth exceeds the expected level.
Time overrun during construction	Concessionaire	In the event the concessionaire fails to meet the project milestone, he or she has to pay damage at 0.1% of the performance security amount (which is about 5% of the total project cost) for each day of delay. However, the damages paid will be refunded in case the project achieves completion on or before the scheduled completion date. In case of delay in entry into commercial service, the concessionaire shall pay damage at 0.1% of the performance security for delay of each day until COD is achieved.
Change of scope	Granting authority/ Concessionaire	Granting authority will bear all the costs arising out of any change of scope order if the costs exceed 0.25% of the total project cost. Otherwise, the costs shall be borne by the concessionaire.
Operation and maintenance risk	Concessionaire	In case of lane closure beyond the specified time limit, concessionaire shall pay damage calculated at 0.1% of the average daily fee for every stretch of 250 m or part thereof, for each day of delay. In case the concessionaire fails to meet the maintenance requirements, it shall pay damage calculated at higher of (a) 0.5% of average daily traffic, and (b) 0.1% of the cost of rectification.
Competing roads	Granting authority	The granting authority will pay the concessionaire compensation equal to the difference between the realizable fee and the projected daily fee until the breach is cured.
Change in law	Granting authority/ Concessionaire	The effects of the change in law in terms of increase in costs or reduction in costs shall be borne by granting authority and concessionaire as per the agreed schedule.
Force majeure risk	Granting authority/ Concessionaire	The parties shall bear their respective costs.
Indirect political risk	Granting authority/ Concessionaire	One-half of all the costs exceeding the insurance cover shall be reimbursed by the granting authority to the concessionaire in case the events happen after financial closure.
Political risk	Granting authority	All the costs attributable to the event will be reimbursed by the authority to the concessionaire.

Table 3. Risk allocation framework for BOT (annuity) project

Risk	Allocated to	Remarks
Pre-investment	Granting authority	The expenses toward project development have been borne by granting authority with budgetary resources
Resettlement and rehabilitation	Granting authority	Resettlement and rehabilitation of the affected people has been carried out by the granting authority with the assistance of Central and State governments
Permit/approval	Granting authority	Granting authority has obtained all the clearances and grant approvals required for the implementation of the project
Delay in land acquisition	Granting authority	Granting authority has been responsible for acquisition of the project site and handing it over to concessionaire
Delay in financial closure	Concessionaire	Failure to achieve financial closure before or on commencement would be construed as an event of default
Time and cost overrun during construction	Concessionaire	Concessionaire has the right to start construction at its own risk Concessionaire is entitled to receive bonus or incur penalty in early or late completion of construction If the delay of commercial operation date from the scheduled project completion date is in excess of 120 days, then granting authority could terminate the concession agreement and appropriate the performance security
Time and cost overrun during operation and maintenance	Concessionaire	If time overruns during operation and maintenance in an annuity period exceed 1000 lane km hours, then concessionaire shall be deemed to be in material breach of operations and management requirements No provision for escalation of annuity payment leads to cost overrun during operations and maintenance, which is borne by concessionaire
Delay in payment of annuity	Granting authority	Granting authority is under the obligation to make payment to the concessionaire within 90 days. Granting authority provides a revolving irrevocable letter of credit for one instalment of the annuity. Granting authority pays the annuity in two equal semi-annual installments.
Change of scope	Granting authority	Granting authority will bear the additional expenditure incurred due to change of scope.
Traffic revenue risk	Granting authority	Granting authority will assume the risk. Granting authority can exercise its right to levy and collect toll
Change in law	Granting authority Concessionaire	The effect of the change in law in terms of increased capital expenditure and costs/taxes shall be borne by granting authority and concessionaire as per an agreed schedule.
Non-political force majeure	Concessionaire	Concessionaire shall bear this risk through insurance.
Political risk	Granting authority	The Granting authority will bear the political risk due to any political event which has a material adverse effect. If failure to make good the effects of the political events occurs, granting authority will reimburse the affected party as the provisions of the termination event due to political risk
Performance standards	Concessionaire	In case of material breach of the operations and management requirements, the granting authority can terminate the agreement. Some of the circumstances leading to material breach include the following: 1. Riding quality below the prescribed acceptable level 2. Non-availability during an annuity payment period exceeds 1000 lane km hours
Lane availability	Concessionaire	Non-availability of lane for reasons due to concessionaire failure to discharge its obligations leads to deduction in the annuity amount payable to concessionaire
Interest rate risk	Concessionaire	The interest rate risk has been factored in the annuity quoted by the concessionaire

Source: Boeing Singh and Kalidindi (2006)

BOT Models—Applicability

The current policy framework for procurement of road projects through the PPP route is to first offer the project on BOT (Toll). If due diligence indicates the project to be unviable on BOT (Toll) in the first instance itself, then it should be offered on BOT (Annuity) basis.

The BOT (Toll) model is predominantly adopted for those stretches of the National Highways network with high/medium traffic density, which is financially viable. In certain stretches where there is a large number of commercial vehicles and a perceived low level of traffic revenue risk, the concessionaire has even agreed to share the revenue. On the other hand, where there is perceived lack of users' willingness to pay toll and the private sector is reluctant to assume traffic revenue risk, the BOT (Annuity) model is used in those projects.

ISSUES

The Government of India has undertaken to create an enabling framework for private sector participation in the development of the National Highways network. There are, however, certain issues limiting greater participation of the private sector in the development of road projects through the PPP route. Some of the issues include the following:

- The current policy of offering the project first on BOT (Toll), then on BOT (Annuity) and then on engineer procure and construct (EPC) contract is likely to introduce delay in the implementation of the project since government approval is required at each stage.
- In case of the BOT (Toll) model, the degree of risk exposure to the concessionaire is high and the private sector is reluctant to take high-risk exposure. On account of this, there has been very low private sector participation in bidding of projects that are to be developed through BOT (Toll) route.
- Though BOT (Annuity) exposes the concessionaire to a lower level of risk, the cost of the project procured through the BOT (Annuity) route is higher. The cost of private capital is comparatively higher compared with the sovereign cost of borrowing.
- The bidding process for PPP road projects has been standardized with the introduction of model RFQs and RFPs. There has been a lack of investor interest in the PPP road projects on account of certain clauses in both the model documents. For instance, as per the model RFQ, only six applicants will be short-listed for the bidding stage based on their respective aggregate experience score. And, as per the model RFP, the bidder will be ineligible for bidding if the bidder was: (1) pre-qualified for the bid stage (second stage of bidding process) in relation to eight or more projects, (2) declared as the selected bidder for undertaking four or more projects, or (3) unable to achieve financial close for two projects within the stipulated time during the period of two months preceding the bid due date.
- As per the MCA, risk allocation has been based on the underlying principle of allocating the risks to the parties best suited to manage them. However, there are certain risks such as land acquisition risk, which in spite of being allocated to the party best suited to manage the risks, has been a major cause for delay in timely completion of the project.

CONCLUSIONS

National highways play a key role in the economic growth of the country. The Union Government of India has taken various measures to upgrade the capacity and quality of the National Highways network. PPP routes have been adopted by the government to meet the funding gap and use techno-managerial efficiencies of the private sector to obviate the inefficiencies in the traditional public procurement system. Various reforms have been introduced by the Union Government of India to create an enabling environment for participation of the private sector in the development of the road projects through the PPP route. Model concession agreements have been developed to facilitate standardization of terms and conditions and ensure uniformity in the various agreements for PPP road projects.

BOT (Toll) and BOT (Annuity) are the two PPP models that have been used in procuring the National Highways projects in India. The BOT (Toll) model is predominantly used for development of projects in stretches with high traffic density and financial viability. On the other hand, BOT (Annuity) is the more attractive PPP model for development of road projects in those stretches of the National Highway network with medium/low traffic density. Hence, the risk profile of the projects and financial viability of the project influences the selection of the type of PPP models. The risk allocation framework for each of these models has been discussed.

In spite of the various initiatives taken by the Government, the participation of the private sector has not been up to the expectations of the Government due to a number of perceived risks by the private sector.

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Evaluation of Centerline Rumble Strips for Prevention of Highway Crossover Accidents in Kansas

Daniel Edgard Karkle
Department of Industrial and Manufacturing Systems Engineering
Kansas State University
237 Durland Hall
Manhattan, KS 66506-5101
dkarkle@ksu.edu

Eugene R. Russell
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506-5000
geno@ksu.edu

Margaret J. Rys
Department of Industrial and Manufacturing Systems Engineering
Kansas State University
237 Durland Hall
Manhattan, KS 66506-5101
malrys@ksu.edu

ABSTRACT

Centerline rumble strips (CLRS) are raised or indented patterns installed in the centerline of undivided two-way highways. The function of the CLRS is to alert drivers who encroach on or cross the centerline by producing noise and vibration to reduce crossover accidents, which accounted for 12.5% of total U.S. highway fatalities in 2007. The objective of this study was to investigate the effectiveness of milled in CLRS in reducing crossover accidents in Kansas. The before-and-after empirical Bayes (EB) method and the Naïve before-and-after method were applied to existing accident data and compared. Two sections of highway were analyzed in this paper. The first was a 15.2-mile section of US-50, between Newton and Hutchinson. The second was a 10.8-mile section on US 40 between Lawrence and Topeka. The Naïve method showed an overall 50.69% reduction in the total number of accidents per mile year and a 92.1% reduction of crossover accidents after installation of CLRS. The EB method indicated an overall 49.4% reduction of total number of accidents and an 89.2% reduction of the crossover accidents. Results of the two methods were statistically comparable. This study showed that installing CLRS reduced crossover accidents in the sections of US 50 and US 40 in Kansas. Results from this study were comparable with results reported by other states, showing that installing CLRS is an effective countermeasure to reduce crossover accidents and potentially other accidents as well.

Key words: centerline rumble strips—empirical Bayes method—Naïve method—safety effectiveness

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PROBLEM STATEMENT

In the United States, 60% of fatal accidents occur on rural roads. Among these, 90% occur on two-lane roads, and 20% of these accidents involve two vehicles traveling in opposite directions, totaling 4,500 fatal accidents per year (Suzman 1999; cited by Russell and Rys 2005). Data from 2007 showed that 21,433 fatal accidents, or 57.5% of the total number of fatal accidents, occurred on undivided two-lane roads in the country (NHTSA 2007). Head-on (HO) and opposite-direction sideswipes (OPP SW) accidents represent 12.5% of fatal accidents and 10.3% of the total number of accidents in the country (NHTSA 2007).

In order to reduce the number of accidents, several state departments of transportation have installed rumble strips and other accident countermeasures on U.S. highways. Rumble strips are grooved or raised indentations placed on the shoulder or on the pavement of a travel lane. The primary purpose of the strips is to prevent accidents by providing noise and vibration when crossed by vehicles, warning drivers. In the United States, the following four different types of rumble strips have been used: milled, raised, rolled, and formed (Richards and Saito 2007).

- Milled—type of rumble strips made by a machine that cuts grooves in the pavement surface
- Rolled—type of rumble strips made by a roller machine that presses into hot asphalt surfaces to create grooves(they must be installed during pavement compaction)
- Formed—type of rumble strips that are formed in concrete pavements during the finishing process
- Raised—type of rumble strips installed over pavements by the adherence of various materials

Centerline Rumble Strips

Centerline rumble strips (CLRS) are primarily installed on the centerline of undivided two-way highways, and their main purpose is reducing crossover accidents, specifically head-on and opposite direction sideswipe accidents, which are usually caused by driver inattention and drowsiness.

After CLRS were accepted as an efficient method in reducing crossover accidents, use of them in the United States has been increasing over the years. Chen and Cottrell (2005) reported that 24 states in United States have installed CLRS. In addition, according to Richards and Saito (2007), there are more than 2,400 miles of CLRS installed in the country, and the most common pattern dimensions are length (dimension perpendicular to the travel lane) of 16 in., width (dimension parallel to the travel lane) of 7 in., depth of 0.5 in., and spacing (center to center) of 12 in. Several authors have reported advantages other than reducing accidents in installing CLRS, such as high benefit-cost ratio, improvement of lateral vehicle position to the right, low interference in passing maneuvers, versatile installation conditions, and public approval. However, some concerns involving CLRS, such as disturbing noise for nearby residents, decreased visibility of the painted strips, faster pavement deterioration, potential driver erratic maneuvers to the right after encountering CLRS, and ice formation in the grooves have been cited in the current literature (Russell and Rys 2005). For this reason, a better understanding of the balance needed between safety and practical effects of CLRS depends on future investigation of these concerns.

In Kansas, two different types of milled-in CLRS have been used: rectangular and football-shaped. The first two installations of CLRS in Kansas are the sections studied in this report. After these initial installations, the Kansas Department of Transportation (KDOT) has been increasing the number of sections of rural highways receiving CLRS. Currently, more than 230 miles of CLRS have been installed in 29 different locations in Kansas (Buckley 2009). In 2007, KDOT adopted an official policy of installation of the strips.

Naïve Before-and-After Method

The Naïve before-and-after method consists of a comparison between the number of accidents on a treated section in the after period and the number of accidents in the same section during the before period. This type of comparison is known to be biased due to the regression to the mean phenomenon, i.e., a section of highway that presented an elevated number of accidents in a period tends to have decreased number of accidents in a future period and vice versa, even with no improvement on the section. Although the Naïve method does not account for the regression to the mean bias, this method has been used in several studies of the effectiveness of CLRS in reducing accidents. Some results of studies that used the Naïve method are the following:

- Fitspatrick et al. (2000) reported a 90% reduction in fatal head-on accidents and 42% reduction in total head-on accidents after the installation of CLRS in a 23 mile section of a two-lane rural highway in California. The total period analyzed was 59 months.
- Outcalt (2001) reported a 34% reduction in head-on accidents (divided per million vehicles), and 36.5% in sideswipe opposite-direction accidents (divided per million vehicles), after the installation of CLRS in a 17 mile section of a two-lane rural highway in Colorado, even with an increase of 18% of annual average daily traffic (AADT). The total period was 44 months.
- Monsere (2002), cited by Russell and Rys (2005), reported an overall 69.5% reduction in crossover accidents after the installation of CLRS in two sections of approximately 8.5 miles each, one a four-lane rural highway and the other a two-lane rural highway in Oregon, using the Naïve before-and-after method.
- The Delaware Department of Transportation (DelDOT (2003) showed a 95% reduction in the average number of head-on accidents per year, 60% in the average per year of "drove left of the center" type of accident, and 8% in the average per year of all types of accidents after the installation of CLRS in a 2.9-mile section of a two-lane rural highway in Delaware. However, this study showed a 4% increase in the average per year of total number of accidents involving injuries and 13% in the average per year number of accidents involving only property damage. The AADT increased 4% from the before to the after period. The total period analyzed was 10 years.
- According to Kar and Weeks (2009), Arizona DOT reported a decrease in the number of fatal and serious injury head-on and opposite-direction sideswipe accidents from 18, in the before period (2000–2002), to seven in the after period (2003–2005). Crashes per million vehicle miles traveled (MVMT) were calculated as follows: MVMT = (number of accidents * 1,000,000) / (AADT * 365 * segment length). In the before period, MVMT was approximately 0.025, and in the after period, it was approximately 0.011. Thus, there was approximately a 56% reduction of these types of accidents after the installation of CLRS.

Empirical Bayes Method

The empirical Bayes (EB) method estimates the number of accidents for the after period, based on linear regression analysis, using information of sections with similar characteristics to the treated sections, and on historical information. The estimated number of accidents is compared to the actual number of accidents counted in the after period.

Even though the EB method can be considered the most acceptable method to evaluate the characteristics of a treatment in reducing accidents over time, only one study that applied the EB method to investigate the safety-effectiveness of CLRS in reducing accidents was found in the literature. In this study, Persaud et al. (2004) used data from seven states and found an estimated reduction of approximately 21% (95% CI

= 5-37%) in frontal and sideswipe opposing-direction types of accidents in treated sections of undivided two-lane rural highways after the installation of CLRS. When injuries were involved in the same types of accidents, the reported reduction was estimated to be 25% (95% CI = 5-45%). Considering all types of accidents, the authors reported an estimated 14% (95% CI = 8-20%) reduction of injury accidents. All types of accidents were reduced by an estimated 15% (95% CI=15-25%). The total length of treated sections was 210 miles at 98 sites.

RESEARCH OBJECTIVE

The objective of this study was to investigate the effectiveness of milled-in CLRS in reducing the number of total and targeted crossover accidents in Kansas. The before-and-after EB method and the Naïve before-and-after method were applied and compared.

RESEARCH METHODOLOGY

The first installation of CLRS in Kansas occurred in June 2003, on approximately 15.2 miles of US 50, between Newton and Hutchinson, in Harvey County. Two different patterns of rectangular CLRS were installed in this location, alternated and continuous. In this report, this section will be referred to as section A. It consists of a two-lane undivided rural highway with lane width of 12 ft, and some passing zones on hills, on a generally straight alignment. Surface type of the lanes and shoulders was bituminous. Width of the shoulders ranged between 5 to 10 ft, and the AADT on this section ranged from 4,000 to 6,000. The second section studied in this report had football-shaped CLRS installed in May 2005 in a segment of approximately 10.8 miles on US 40 between Lawrence and Topeka. In this report, this section will be referred to as section B. It was a two-lane undivided rural highway with lane width of 11 ft. It had a high percentage of no-passing zones with many horizontal and vertical curves. Surface type of the lanes was bituminous, while the three ft shoulders had turf surface.

Naïve Before-and-After Method

This method calculates the proportion of accidents in the after period compared to the before period. For section A on US 50, the before period analyzed ran from January 1998 to June 2003. The after period considered for this section ran from July 2003 to December 2007. For section B on US 40, the before period analyzed was from January 1998 to May 2005. The after period studied for this section was from June 2005 to December 2007.

Since the before and after periods for the two studied sections had different durations, the counted number of accidents was divided by the duration of the period and by the length of the section. The comparable results were stated in annual accidents per mile.

Empirical Bayes Method

The methodology described in this section was based on Hauer (1997, 2002) and Harwood et al. (2002).

According to Patel et al. (2007), the concept of the EB method is to estimate the number of accidents that the sections of interest would have had in the after period if no treatment had been used and compare this number to the actual number of accidents in the after period on the section submitted to treatment. Therefore, it is possible to estimate the influence of the treatment (CLRS) on the final result. In this report, the expected number of accidents in the after period was corrected due to differences in traffic volume over the periods and due to the differences between the duration of the periods.

The estimated number of accidents in the after period if no treatment was used is not only based on historical information (accidents that occurred in the section of interest in the before period), but also uses data from a group of similar sites—highways without treatment with similar characteristics to the treated section (AADT, geometry, rate of accident per year, etc.) to calibrate a safety performance function (SPF). According to Hauer et al. (2002), methods that estimate the safety-effectiveness of a treatment, based only on the counted accidents in the section of interest in the before period, show results that can be inflated due to regression to the mean bias.

The SPFs can be obtained by performing a regression analysis, generally using a negative binomial distribution. In this report, the regression analyses were done using AADT as the only predictor for the total number of accidents in one mile of a specific group of highways. As a result, the SPF should predict the number of accidents per mile on a highway with determined characteristics, according to the volume of traffic. The most difficult task for a researcher, in order to apply the EB method, is to find sections that are comparable in terms of traffic, number of accidents, and geometry to the studied sections. In this report, the geometry and volume of traffic were the most important factors used to choose the similar sites.

KDOT provided the total number of accidents from sections that were used as potential similar sites to match section A on US 50 and section B on US 40. The AADT were obtained from KDOT historical traffic count maps, available online (KDOT 2009).

In order to match section A on US 50, there were three potential similar sites available with the following characteristics: two-lane rural highways; lane width of 12 ft; surface type for lane and shoulder: bituminous; shoulder width ranging from 5 to 10 ft; AADT ranging from 3,800 to 6,500; and accident rate ranging from 1.926 to 4.259 accidents per 100 MVM (million vehicle-miles of travel). The three locations were the following:

- US 54 in Kiowa County from the junction of US 54 and US 400 to the Pratt county lane, excluding the cities of Greensburg and Haviland; it was divided into two sections, called Kiowa A, from county reference posts 6.443 to 14.410; and Kiowa B, from county reference posts 15.666 to 30.355
- US 75 in Montgomery County from SJCT US 75/US 166 to US 75/US 400; it was divided into three sections, called Montgomery A, from county reference posts 1.697 to 4.695; Montgomery B, from county reference posts 17.980 to 20.664; and Montgomery C, from county reference posts 26.311 to 33.493
- US 281 in Barton County from RS-981 to the transition 2L/4L undivided, about 1.5 miles north of RS-42; it was divided into three sections, called Barton A, from county reference posts 2.100 to 5.320; Barton B, from county reference posts 8.622 to 10.272; and Barton C, from county reference posts 12.330 to 17.059

In order to match section B on US 40, four potential similar sites were available, and they had the following characteristics: two-lane rural highways, lane width of 11 ft, surface type for lane was bituminous, turf shoulder with a width of 3 ft and AADT ranging from 2,000 to 5,100, and accident rate ranging from 0 to 3.375 accidents per 100 MVM (not considering US-24). The four locations were the following:

- Called Douglas, US 24 from Douglas/Leavenworth County line to Tonganoxie south city limit; county reference posts 0.000 to 8.625.
- Called Brown, KS-20 in Brown County from the JCT KS-20/RS-1265 to west city limit of Horton
- Called Cherokee, KS-7 in Cherokee County from the JCT KS-7/RS-1166 to JCT US 400/KS-7

- Called Franklin, KS-33 in Franklin County from the JCT I-35/KS-33 to south city limit of Wellsville

In this report, the GENMOD procedure in the commercial Statistical Analysis System (SAS) software version 9.1 was used to compute the SPF functions. Since SAS 9.1 uses the natural logarithm as a link function in the GENMOD procedure, the model of the SPF function is exponential, as presented by equation (1).

$$ACC = e^{\beta_0} * e^{(AADT_{Before} * \beta_1)}, \tag{1}$$

where ACC = expected number of accidents (per mile per year) in a section with the same characteristics to the section of interest, $AADT_{Before}$ = average AADT for the before period, and $\beta 0$ and $\beta 1$= intercept and slope of the regression analysis.

The negative binomial regression analysis also gives the overdispersion parameter (k) per mile. It is discussed in more details in Hauer (2001).

The result of the EB method depends on how much "weight" is given to accidents in similar sites (first part of equation [2]), and to the counted accidents in the treated sections during the before period (second part of equation [2]).

$$estimated\ \ ACC = \rho * ACC + (1-\rho) * ACC\ \ Before \quad , \tag{2}$$

where *estimated ACC* = expected number of accidents in a period with the same duration of the before period, ρ = weight or how much influence is due to historical data or similar sites. This parameter was calculated using equation (3).

$$\rho = \frac{1}{1+(ACC/k)} \tag{3}$$

The standard deviation of e*stimated ACC*, denoted by σ_{EST}, was calculated by equation (4).

$$\sigma_{EST} = \sqrt{estimated\ ACC * (1-\rho)} \tag{4}$$

The corrected number of accidents that would have occurred in the after period if no treatment had been made was calculated by equation (5).

$$\mu = estimated\ \mathrm{ACC} * \mathrm{C}_1 * C_2\ , \tag{5}$$

where C_1 = ratio between the result of equation 1, using $AADT_{After}$,and the result of the same equation, using $AADT_{Before}$. It is clear that the relation between AADT and the number of accidents in a period is not linear (it follows the function given by equation 1). For this reason, C1 was used to correct e*stimated ACC,* instead of the simple ratio between AADTs. C_2 = ratio between the duration of the after period and the duration of the before period.

The variance of μ was calculated by equation (6).

$$Var\ \mu = C_1^2 * C_2^2 * \sigma_{Est}^2 \tag{6}$$

An estimate of the safety-effectiveness of the CLRS treatment can be obtained by the ratio between *ACC After* and μ. However, Hauer (1997) claims that an estimate like this is biased. A better estimate is obtained using equation (7).

$$\omega = \frac{ACC\ After / \mu}{1 + (Var\ \mu / \mu^2)}, \tag{7}$$

where ω = unbiased estimate of safety effectiveness of a treated site, or more than one site (using sums of the terms).

The variance of ω was obtained by equation (8).

$$Var\ \omega = \omega^2 * [\frac{(Var\ \text{ACC After}/\text{ACC After}^2) + (Var\ \mu/\mu^2)}{1 + (Var\ \mu/\mu^2)}]^2, \tag{8}$$

where *Var ACC After* = Variance of *ACC After* = ∑ *ACC After*.

Similar sites chosen and used to match section B on US 40 were US 24 in Douglas County and KS-33 in Franklin County. These were chosen the only ones used due to lack of convergence on the SAS algorithm when using the data of other potential similar sites available.

Data from all potential similar sites were used to calculate the SPF for section A on US 50, since the algorithm on SAS converged.

Parameters of the regression analysis and goodness-of-fit of the SPF functions are presented in Table 1. The goodness-of-fit was evaluated by deviance, scaled deviance, Pearson chi-square, and log likelihood statistics. The closer to 1 that value/DF reaches, the better the goodness-of-fit.

Table 1. Parameters of regressions and evaluation of goodness-of-fit of the SPF functions

SPF for US-40—Total Accidents				SPF for US-50—Total Accidents			
Parameter	**Estimate**	**SE**	**Pr > Chi Sq.**	**Parameter**	**Estimate**	**SE**	**Pr > Chi Sq.**
β_0	-1.2229	0.6328	0.0533	β_0	-1.4019	0.5569	0.0118
β_1	0.0007	0.0001	< 0.0001	β_1	0.0004	0.0001	< 0.0001
K	-0.0793	0.0245		K	-0.1475	0.0335	
Criterion	**DF**	**Value**	**Value/DF**	**Criterion**	**DF**	**Value**	**Value/DF**
Deviance	16	17.1240	1.0703	Deviance	62	48.8752	0.7883
Scaled Deviance	16	17.1240	1.0703	Scaled Deviance	62	48.8752	0.7883
Pearson Chi-Square	16	17.0100	1.0631	Pearson Chi-Square	62	48.7775	0.7867
Log Likelihood		92.6221		Log Likelihood		-24.3943	

In order to compute the head-on and opposite-direction sideswipe accidents that would be corrected by CLRS, all police accident reports for accidents in the treated sections of US 40 and US 50 were analyzed. Since this paper uses the total number of accidents to calibrate SPFs, police accident reports from the similar sites were not analyzed. The potentially correctable crossover accidents considered occurred on non-intersection zones due to drivers' inattention, influence of alcohol, and drivers that fell asleep. All accidents that occurred due to other factors were not considered as CLRS correctable, and were not computed in the analysis of HO + OPP SW accidents.

Another analysis was done using the total number of accidents, excluding those involving animals, intersections, or related to intersections. The effect of the treatment could be over or under estimated due to the presence of these types of accidents on the calculation, because the incidence of animals and the number of intersections per mile of highway can be very different from one section to another, and there was no data available revealing these numbers.

In summary, two analyses were done. The first used total number of accidents excluding intersections and animals. The second computed only HO + OPP SW types of accidents. Both analyses had the SPFs generated using total number of accidents (all types) as input data.

KEY FINDINGS

Naïve Before-and-After Method

Table 2 shows results of the Naïve before-and-after method.

Table 2. Results of the Naïve before-and-after method

Total	Section	Length (miles)	Installation	Years	# Acc Before	# Acc After		Rate Before	Rate After	Reduction
				Before	After					
	A on US 50	15.18	June, 2003	5.5	4.5	75	38	0.90	0.56	**38.07%**
	B on US 40	10.76	May, 2005	7.42	2.58	205	32	2.57	1.15	**55.11%**
	Overall	25.94				280	70	3.47	1.71	**50.69%**
HO + OPP SW	**Section**	**Length (miles)**	**Installation**	**Years**		**# Acc Before**	**# Acc After**	**Rate Before**	**Rate After**	**Reduction**
				Before	After					
	A on US 50	15.18	June, 2003	5.5	4.5	6	1	0.07	0.01	**79.63%**
	B on US 40	10.76	May, 2005	7.42	2.58	9	0	0.11	0.00	**100.00%**
	Overall	25.94				15	1	0.18	0.01	**92.07%**

During the before period on US 50 (section A), there were six HO or OPP SW accidents. Four of these were caused by drivers' inattention, one occurred due to alcohol interference, and one due to driver falling asleep. During the after period in the same section, there was only one accident, caused by driver's inattention. Considering the section of US 40, nine HO or OPP SW accidents occurred in the before period. Five of these occurred due to drivers' inattention, two due to alcohol influence, and two due to drivers that fell asleep. No HO or OPP SW accident occurred in this section during the after period.

Results of the Naïve method showed that in section A on US 50, the number of total accidents per mile per year (excluding animals and intersections) decreased 38.07% in the after period compared to the before period. The number of crossover accidents (head-on and opposite-direction sideswipe) decreased 79.63% in this section after the installation of CLRS. In section B on US 40, the number of total accidents per mile per year decreased 55.11%. The number of crossover accidents decreased 100%, since no crossover accidents occurred in this section after the installation of CLRS. Overall, the number of total accidents per mile per year decreased 50.69%. The number of crossover accidents decreased 92.07%. It is clear that CLRS were not created with the purpose of reducing all types of accidents. However, results of the Naïve method provide evidence that CLRS are potentially effective in reducing total number of accidents, and are particularly effective in reducing head-on and opposite-direction sideswipe accidents.

Empirical Bayes Method

Table 3 presents the analysis of safety effectiveness in the sections treated with CLRS, using the EB method.

Table 3. Safety effectiveness in sections treated with CLRS – EB method

Section	AADT Before	AADT After	Counted Accidents During After Period with Treatment		Expected Accidents During After Period in case of No Treatment		Reduction	
			Total	HO + OPP SW	Total (σ)	HO + OPP SW (σ)	Total (95% CI)	HO + OPP SW (95% CI)
A on US 50	5524	5036	38	1	53.93 (6.23)	4.22 (1.74)	30.47% (27.93% - 33.01%)	79.75% (68.03% - 91.46%)
B on US 40	4255	4465	52	0	83.83 (5.86)	3.65 (1.22)	62.01% (61.44% - 62.58%)	100.00%
Overall			**90**	**1**	**137.76 (8.55)**	**8.68 (2.21)**	**49.38% (47.58% - 51.18%)**	**89.18% (66.70% - 111.67%)**

Section A on US 50 presented a statistically significant reduction, estimated as 30.47%, in number of total accidents and 79.75% in number of HO+OPP SW accidents. Considering section B, the reduction of total accidents was significant and estimated as 62.01%. The targeted crossover accidents were reduced from 3.65 to zero (100% of reduction). Section B, treated with football-shaped CLRS, presented a potentially better effect when compared with section A, treated with rectangular CLRS. However, this comparison may have limited validity due to differences between all other variables relative to the sections.

Overall, considering both sections, reduction of total accidents (excluding animals and intersections) was significant and estimated as 49.38%, and reduction of the targeted crossover (HO + OPP SW) accidents was also significant and estimated as 89.18%. Thus, the treatments were effective in decreasing the number of total and crossover accidents.

Table 4 summarizes the results of this report, comparing the two methods.

Table 4. Comparison between results of the Naïve and EB methods

Section	Naïve Total	EB Total (95% CI)	Naïve HO + OPP SW	EB HO + OPP SW (95% CI)
A on US 50	38.07%	30.47% (27.93% - 33.01%)	79.63%	79.75% (68.03% - 91.46%)
B on US 40	55.11%	62.01% (61.44% - 62.58%)	100.00%	100.00%
Overall	**50.69%**	**49.38% (47.58% - 51.18%)**	**92.07%**	**89.18% (66.70% - 111.67%)**

The comparison between the results of the two methods reveals that the reduction in the total number of accidents, calculated by the Naïve method, is comparable with the reduction found by the EB method. Considering the targeted crossover accidents, results of the two methods are also comparable.

In summary, both methods showed a significant reduction in accidents after the installation of CLRS. It is concluded that the treatments had a positive effect in reducing both crossover accidents and total accidents in the two analyzed sections.

Comparison to Other States

Although there are a considerable number of studies about the effectiveness of CLRS in reducing accidents in the United States, types of accidents evaluated in these studies are not consistent. Therefore, in order to achieve a better comparison between the results found in Kansas and other studies, extra calculations were necessary in this study, using the same methodology previously cited for the Naïve and EB methods.

Table 5 summarizes the reduction of accidents per state after the installation of CLRS, calculated for other types of accidents.

Table 5. Reduction of accidents per state after installation of CLRS

State / Accident Type	Naïve Method					EB Method	
	Fatal HO	HO	OPP SW	HO + OPP SW	Fatal and Serious Injuries HO + OPP SW	HO + OPP SW	Total (all Types)
Arizona					56%		
California	90%	42%				12%	14%
Colorado		34%	37%			31%	11%
Delaware		95%				81%	23%
Maryland							19%
Minnesota						-12%	0%
Oregon				70%			46%
Washington						21%	25%
Kansas	**80%**	**81%**	**78%**	**80%**	**59%**	**85%**	**33%**

Based on this limited data available from other states, it can be assumed that overall results found in Kansas are comparable to results found by other states, which reinforces the evidence that CLRS are effective in preventing crossover and possibly other types of accidents as well.

CONCLUSIONS

The results showed that installing CLRS reduced head-on and opposite-direction sideswipe types of accidents in the treated sections of US 50 and US 40. Although the CLRS were not expected to reduce all types of accidents, it appears they may have an effect on decreasing the total number of accidents as well. Results of the EB method were comparable to results found by the Naïve method for the analyses of CLRS correctable head-on, and sideswipe accidents and for the total number of accidents, excluding those involving animals and intersections.

The limitation of this study was the use of SPFs obtained for all types of accidents for predicting the number of the crossover accidents and total number of accidents.

Results of this report are comparable with results found in other states and evidence found in the literature, showing that installing CLRS is an effective way to reduce crossover accidents.

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Improving On-Site Construction Productivity Using the WRITE System

Seonghoon Kim
Department of Construction Management and Civil Engineering Technology
Georgia Southern University
P.O. Box 8047, 1006C Carruth Bldg.
Statesboro, GA 30458
shkim@georgiasouthern.edu

Yong Bai
Department of Civil, Environmental, and Architectural Engineering
University of Kansas
1530 W. 15th Street, 2150 Learned Hall
Lawrence, KS 66045
ybai@ku.edu

ABSTRACT

Existing construction productivity measurement techniques are not capable of providing the real-time productivity data to project managers and engineers for analyses and sharing the data among participants involved in construction operations. As a result, actions to address the on-site productivity problems cannot be taken just in time. To address these shortfalls, the **W**ireless **R**eal-t**i**me Produc**t**ivity M**e**asurement (WRITE) System was developed to measure the on-site construction productivity in real time. In addition, an on-site construction productivity improvement model using the WRITE System and the benchmark data was developed. Field experiments were conducted on a bridge construction project to determine the accuracy of the developed model. During the experiments, the real-time productivity data measured by the WRITE System was compared to the benchmark productivity data. The results of the comparison provided the necessary information for project managers and engineers to determine if immediate actions should be taken to improve the on-site construction productivity. The success of this research project made several major contributions to the advancement of the construction industry. First, it advanced the application of wireless technology in construction operations. Second, it provided an advanced technology for engineers and project managers to determine on-site construction productivity in real time. Thus, actions on improving on-site construction productivity could be taken just in time if needed. These advancements enhance the contractors' capability of managing construction projects.

Key words: benchmark—construction—measurement—productivity—wireless

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INTRODUCTION

Productivity data have been widely used as performance indicators to evaluate construction operations throughout the entire phase of construction. Construction companies must continuously track productivity in order to estimate their performance to maintain profitability and to prepare future biddings (Ghanem and Abdelrazig 2006). Hence, measuring productivity at a project site has been an important task in the construction industry (Chang 1991). Over the years, on-site productivity measurement techniques have been developed, including questionnaires, stopwatch studies, photography, time-lapse videos, and video-taping (Oglesby, Parker, and Howell 1989).

In recent years, real-time monitoring systems have become key methods to reduce the gap between actual and planned production rates in a timely manner (Navon and Shpatnitsky 2005; Sacks et al. 2005; Peddi et al. 2009). A real-time video system was developed by Everett and Slocum to monitor lifting activities of crane in attempts to improve both productivity and safety of crane operations (Everett and Slocum 1993). Since 2000, wireless technologies, such as global positioning system (GPS) and radio frequency identification (RFID) system, were utilized to track the current status of the resources and activities. A GPS technology was used to automatically measure earthmoving performance by identifying the locations of equipment at regular time intervals and converting the information into project productivity (Navon and Shpatnitsky 2005). A web-based camera was used to monitor interior construction operations. This web-based network technology produced an opportunity to avoid using a wired network connection in such congested construction jobsite (Kang and Choi 2005).

Existing on-site construction productivity measurement methods have some common limitations in providing the real-time productivity data to project managers and engineers for analyses and sharing the data among participants involved in construction operations. As a result, opportunities to improve construction operations are lost. To address these shortfalls, there is an urgent need to develop new technologies that can be used to collect and analyze the on-site construction productivity data in real time.

OBJECTIVES

The first objective of this research project was to develop the Wireless Real-time Productivity Measurement (WRITE) System that could be used to collect and analyze the on-site construction productivity data in real time. Using the real-time productivity data, engineers and project managers may be able to accurately determine the bridge replacement progress and easily share the information with all parties involved in the bridge replacement project. Thus, the wireless real-time productivity measurement technology has a great promise to improve construction schedule forecasts and to increase emergency response capability after extreme events. The second objective was to develop a model for improving on-site construction productivity in real time utilizing the data collected by the developed WRITE System and the benchmarking data gathered from experts in the construction industry.

RESEARCH METHODOLOGY

The first phase of this research project was to conduct the literature review followed by the development of the WRITE System. During the development phase, the researchers identified the necessary hardware and software for the system and outlined a framework to show the connection of major hardware components. The third phase was to develop the model for construction productivity improvement using the WRITE System and the productivity benchmark data. A survey methodology was used to obtain the productivity benchmark data from bridge construction experts. The fourth phase was to conduct the field experiments to test the developed model. During the field experiment, productivity data collected using the WRITE System at the construction site was compared to the benchmark data to form the basis for the project managers to make productivity improvement decision in real time. Finally, research findings and recommendations for future research were outlined. Field experiments were conducted in a bridge reconstruction project to demonstrate how this procedure was used by the construction project managers to identify on-site productivity problems and to take immediate action to address these problems.

DEVELOPMENT OF WRITE SYTEM

Overview of the WRITE System

The developed WRITE system includes a video camera, a digital camera, a data processor, an AC transformer, two antennas, and a laptop computer as shown in Figure 1. The preliminary test results indicated that the developed system can measure the on-site construction productivity accurately (Kim 2008).

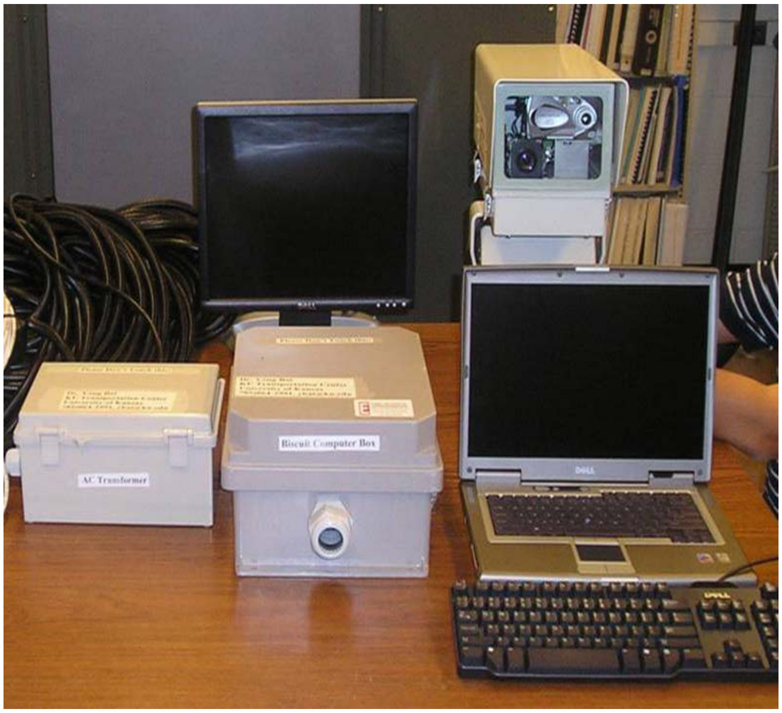


Figure 1. Major components of the WRITE system

WRITE System Framework

Figure 2 presents the framework of the WRITE System that was developed during the process of this research project. Once the video camera takes pictures from the construction site, the data processor immediately saves the pictorial data into files. Then, these files are transmitted in real time via wireless modems. An engineer or a project manager with the IP address at another location can access the data files via a wireless modem or a local area network (LAN) to conduct productivity analyses using a computer with VM View software. After finishing the data analysis, productivity data and live pictures can be presented in a website so that other users, such as the owner, engineers, contractors, and material suppliers, can share the information.

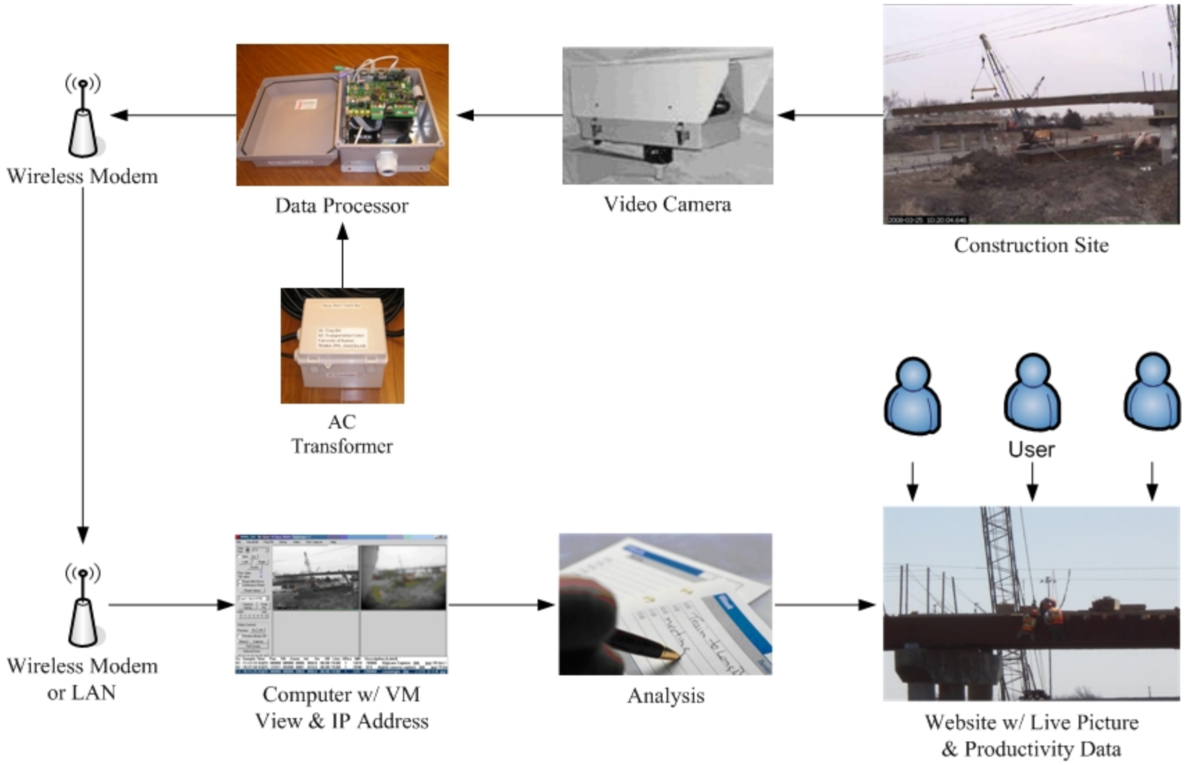

Wireless Modem
Data Processor
Video Camera
Construction Site
AC Transformer
User
Wireless Modem or LAN
Computer w/ VM View & IP Address
Analysis
Website w/ Live Picture & Productivity Data

Figure 2. Framework of the WRITE System

Development of the Productivity Improvement Model

After building the WRITE System, a model for the on-site construction productivity improvement was developed as shown in Figure 3. In this model, the first task is to collect pictorial data in the construction site using the WRITE System. The second task is to determine the real-time productivity data which is the ratio of working and nonworking time. The third task is to compare the real-time productivity data with the productivity benchmark data. During the comparison, a project manager or an engineer will answer two questions, and then make productivity improvement decisions accordingly. The first question is whether the real-time productivity data is higher than the benchmark data at which action should be taken. If the answer for this question is no, management needs to take action immediately to improve the on-site productivity. If the answer for the first question is yes, management goes to the next stage to compare the real-time data with the acceptable benchmark data. If the real-time data is higher than the acceptable benchmark data, no action is needed. Otherwise, management needs to be aware that action may be needed in the near future and close monitoring is necessary at the construction site. The developed model can be utilized for the entire period of construction or for the segments of construction.

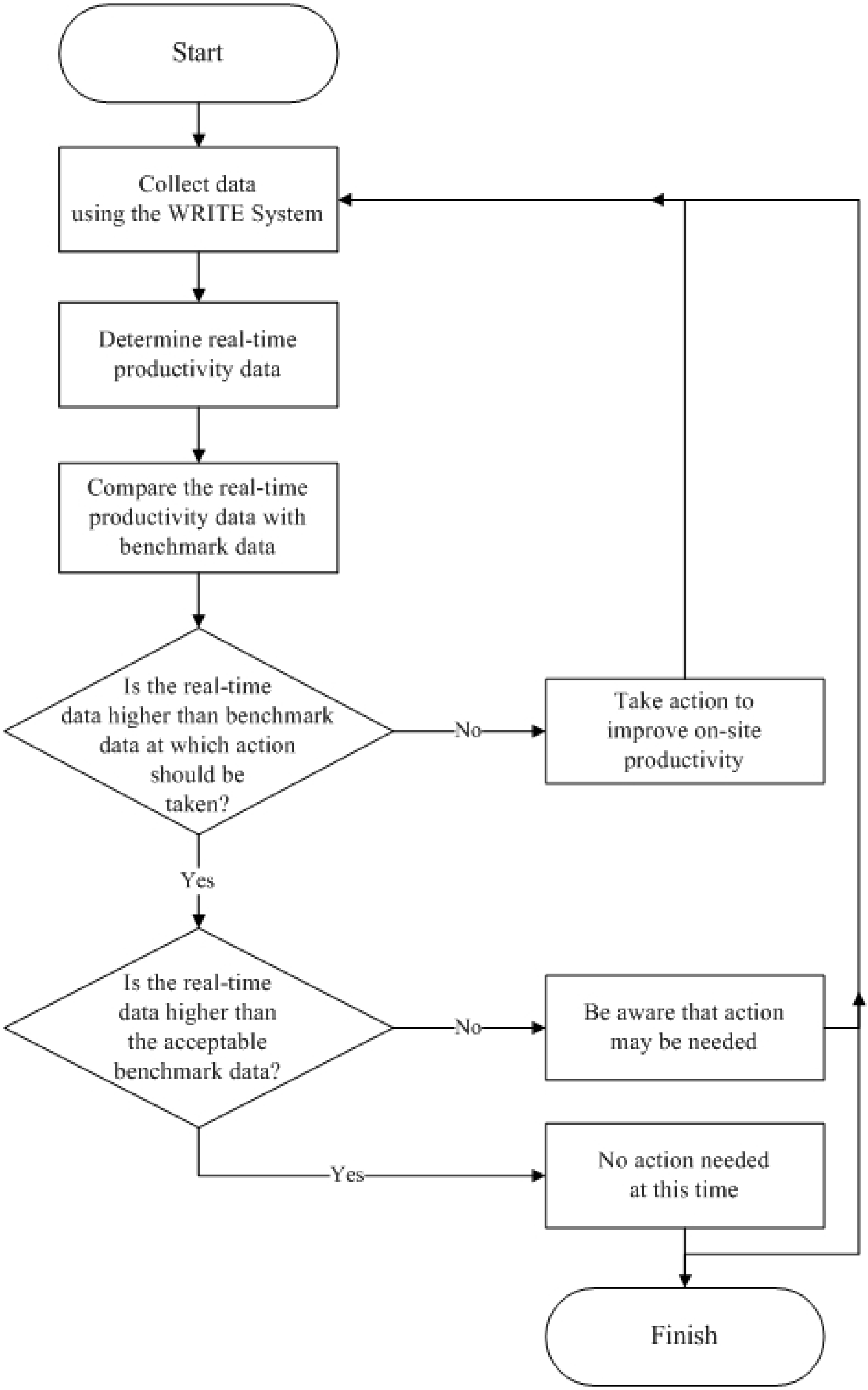

Start
Collect data using the WRITE System
Determine real-time productivity data
Compare the real-time productivity data with benchmark data
Is the real-time data higher than benchmark data at which action should be taken?
No
Take action to improve on-site productivity
Yes
Is the real-time data higher than the acceptable benchmark data?
No
Be aware that action may be needed
Yes
No action needed at this time
Finish

Figure 3. Productivity improvement model using the WRITE System and benchmarking data

FIELD EXPERIMENTS

The researchers conducted field experiments to demonstrate how the developed model can be utilized to improve on-site construction productivity. The field experiment was performed in a steel girder bridge reconstruction project over Interstate 70 in Lawrence, Kansas. On-site productivity data were collected for two months from March 24 to May 23 in 2008. Prior to the field experiments, the researchers reviewed plans, specifications, the project cost and schedule, and other publications to obtain information on project uniqueness, crew size, and historical daily productions. In addition, the researchers visited the jobsites to gather the geographical information in order to develop the field experimental plan.

Work Breakdown Structure (WBS)

The work breakdown structure (WBS) has been widely used to manage the project. WBS is defined as “a deliverable-oriented grouping of project elements,” which organizes and defines the hierarchical structure of the entire project (Jung and Woo 2004). It is often used in the complex construction projects to identify project information and improve the efficiency of control processes (Chua and Godinot 2006). A WBS shows the relationship of all project elements at different levels and makes them more manageable and measurable. The number of levels depends on the size and complexity of the projects (U.S. Department of Energy 1997). The bridge reconstruction project used in the field experiments was broken down into four levels, including Level 1 (project), Level 2 (work zone), Level 3 (activity), and Level 4 (operation). Examples of the levels of steel girder bridge WBSs are shown in Table 1.

Table 1. WBS for steel girder bridge reconstruction

Level 1 (Project)	Level 2 (Work Zone)	Level 3 (Activity)	Level 4 (Operation)
Steel Girder Bridge	General	Mobilization	Set up Crane
	Abutment	Traffic Control	Moving concrete safety barrier
	Pier 1	Demolition	Driving pile
	Pier 2	Excavation	Forming
	Pier 3	Abutment 1	Structural excavation
	North side	Abutment 2	Slope protection (filter fabric and rock)
	South side	Pier Drill Shafts	Set bearing devices
	Span 1	Pier Columns	Unload beams
	Span 2	Pier Cap	Set beams
	Span 3	Slope protection	Install diaphragms
	Span 4	Beam Setting	Bolting and tightening splice
		Deck Forming	Ground splice
		Reinforcing Deck	Prepare deck material
		Bridge Barrier Rail	Prepare deck forming
		Concrete Barrier	Overhangs
		Backfill Abutments	Strip
		Approach road	Place backwall (strip drain & backfill)
			Tying rebar
			Pouring and curing
			Strip and check elevation
			Others

Determining Productivity Benchmark Data

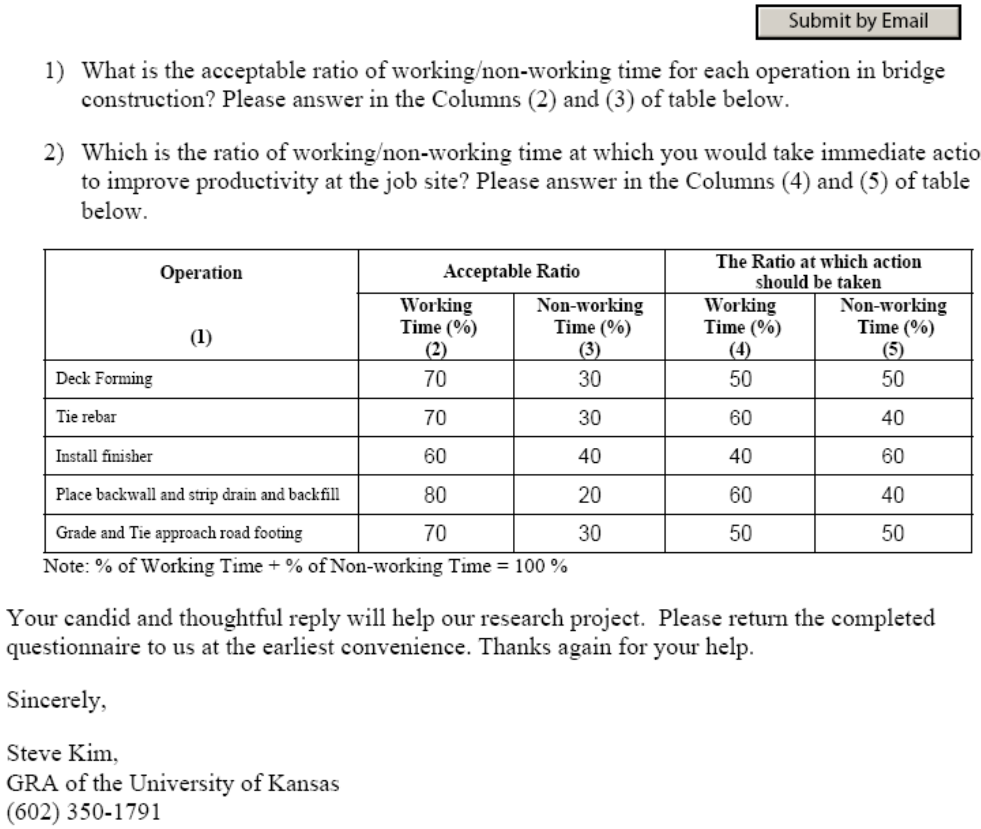
Benchmarking has been used as a tool to improve productivity since the early 1980s (Jackson et al. 1994). The Construction Industry Institute (CII) has established construction productivity metrics and a reporting format for construction productivity benchmarking and improvement (Han et al. 2005). Actual working time of construction workers is at 56% in nuclear plant construction projects (Hewage and Ruwanpura 2006). Christian and Hachey (1995) studied concrete-placement operations, and the finding was 61% working time and 39% nonworking time. According to the previous research projects, the ratio of working time and nonworking time ranges approximately from 50:50 to 60:40. However, there is no consensus on the acceptance ratio of working time verse nonworking time in the construction industry because construction projects have different natures such as different types of projects, activities, and operations.

In this research project, working and nonworking time for five bridge operations were identified, including deck forming, tying rebar, installing finisher, backfilling, and placing approach road footing. A total of 66 hours of video tapes were recorded using the WRITE System to determine the productivity rates for the five bridge operations. These videos were all taken zoomed-in so that the researchers could clearly identify the working and nonworking time for each operation. The ratio of working and nonworking time was 86% and 14% on average as shown in Table 2.

Table 2. Ratio of working and nonworking time determined by the WRITE System

Operation	Time (Second)		Percentage (%)	
	Working Time	Nonworking Time	Working Time	Nonworking Time
Deck forming	24,720	2,160	92	8
Tying rebar	40,320	5,880	87	13
Installing finisher	71,230	21,100	77	23
Placing backwall, strip drain, and backfill	44,850	1,950	96	4
Grade and tie approach road footing	21,275	2,725	89	11
Total	202,395	33,815	86	14

To identify the benchmarking data, an e-mail survey form shown in Figure 4 was developed and distributed to four construction professionals in the bridge construction. Table 3 shows names of construction professionals and their company names, specialties, and positions. The benchmark data were based on professional intuitions about rates of working time and nonworking time for each of the five bridge construction operations.



1) What is the acceptable ratio of working/non-working time for each operation in bridge construction? Please answer in the Columns (2) and (3) of table below.

2) Which is the ratio of working/non-working time at which you would take immediate action to improve productivity at the job site? Please answer in the Columns (4) and (5) of table below.

Operation (1)	Acceptable Ratio		The Ratio at which action should be taken	
	Working Time (%) (2)	Non-working Time (%) (3)	Working Time (%) (4)	Non-working Time (%) (5)
Deck Forming	70	30	50	50
Tie rebar	70	30	60	40
Install finisher	60	40	40	60
Place backwall and strip drain and backfill	80	20	60	40
Grade and Tie approach road footing	70	30	50	50

Note: % of Working Time + % of Non-working Time = 100 %

Your candid and thoughtful reply will help our research project. Please return the completed questionnaire to us at the earliest convenience. Thanks again for your help.

Sincerely,

Steve Kim,
GRA of the University of Kansas
(602) 350-1791

Figure 4. Survey form for collecting benchmark data

Table 3. List of survey construction professionals

Name	Company	Construction Specialty	Position
Ken Johnson	BRB contractors, Inc.	Bridge	Project Manager
Mike Laird	BRB contractors, Inc.	Bridge and Plant	Project Manager
Ray Rinne	A.M. Cohron & Son, Inc.	Bridge	Superintendent
Christopher J. Rech	A.M. Cohron & Son, Inc.	Bridge	Project Manager

Table 4 shows acceptable ratios provided by four survey participants. The overall average ratio for working time (WT) was 81% and overall average ratio for nonworking time (NWT) was 19%. Tying rebar had the highest nonworking ratio of 21%, while deck forming had the lowest rate of 16%. According to the survey participants, they can accept the working time ratio of at least 79% for these bridge operations. Table 5 presents ratios at which action should be taken by project managers to improve on-site construction productivity. The overall average ratio for WT was 75% and overall average for NWT was 25%. Tying rebar had the highest nonworking time rate of 28%, while deck forming had the least nonworking time rate of 23%.

Table 4. Acceptable ratio

Operation	**BRB 1**		**BRB 2**		**A.M. Cohron 1**		**A.M. Cohron 2**		**Average**	
	WT (%)	**NWT (%)**	**WT (%)**	**NWT (%)**	**WT (%)**	**NWT (%)**	**WT (%)**	**NWT (%)**	**WT (%)**	**NWT (%)**
Deck forming	85	15	80	20	85	15	85	15	84	16
Tying rebar	80	20	75	25	80	20	80	20	79	21
Installing finisher	85	15	75	25	85	15	85	15	82	18
Placing backwall, strip drain, and backfill	70	30	80	20	85	15	85	15	80	20
Grade and tie approach road footing	80	20	80	20	85	15	85	15	82	18
Average	80	20	78	22	84	16	84	16	**81**	**19**

Note: WT–Working Time; NWT–Nonworking Time

Table 5. Ratio at which action should be taken

Operation	BRB 1		BRB 2		A.M. Cohron 1		A.M. Cohron 2		Average	
	WT (%)	**NWT (%)**	**WT (%)**	**NWT (%)**	**WT (%)**	**NWT (%)**	**WT (%)**	**NWT (%)**	**WT (%)**	**NWT (%)**
Deck Forming	75	25	75	25	80	20	80	20	77	23
Tying rebar	70	30	70	30	75	25	75	25	72	28
Installing finisher	75	25	70	30	80	20	80	20	76	24
Placing backwall, strip drain, and backfill	60	40	75	25	80	20	80	20	74	26
Grade and tie approach road footing	70	30	75	25	80	20	80	20	76	24
Average	70	30	73	27	79	21	79	21	**75**	**25**

Note: WT–Working Time; NWT–Nonworking Time

Table 6 presents the results of the comparison between the benchmarking data from the survey and the real-time productivity data determined by the WRITE System. For the operation of installing finisher, the nonworking ratio of 24% was equal to the ratio at which action should be initiated by the construction manager. The rest of operations had larger working time ratios than the minimum required working ratios.

Table 6. Data comparison between the WRITE System and the benchmarks

Operation	Acceptable Ratio		Ratio at which action should be taken		WRITE System	
	WT (%)	**NWT (%)**	**WT (%)**	**NWT (%)**	**WT (%)**	**NWT (%)**
Deck Forming	84	16	77	23	92	8
Tying rebar	79	21	72	28	87	13
Installing finisher	82	18	76	24	76	24
Placing backwall, strip drain, and backfill	80	20	74	26	96	4
Grade and tie approach road footing	82	18	76	24	89	11
Average	81	19	75	25	88	12

Note: WT – Working Time; NWT – Nonworking Time

By comparing the rates from the WRITE System to the benchmark data, project managers can take actions for improving on-site construction productivity in real time. As shown in Table 7, there are three cases that project managers can make decision using the developed model. First, if the productivity ratio measured by the WRITE System is higher than the acceptable ratio, then, no action is required. Second, if the ratio is between acceptable ratios and ratios at which action should be initiated, then, management needs to be aware that an action may be needed in the near future. Finally, if the ratios are lower than the minimum required rate, then, the project manager needs to take actions immediately.

Table 7. Making management decisions using the WRITE System

No.	Ratios from the WRITE System	Action
1	Higher than acceptable ratios	No action needed
2	Between acceptable ratios and ratios at which action should be taken	Aware that action may be needed
3	Lower than ratios at which action should be taken	Action is required

CONCLUSIONS AND RECOMMENDATIONS

This research project made several major contributions to the advancement of knowledge in the construction industry. First, it advances the applications of wireless technologies in construction operations. As a result, all participants involved in construction projects can monitor construction projects at any location and any time. In addition, productivity data gathered by the WRITE System can be sent to a website so that owners, engineers, contractors, and material suppliers can share data and make necessary actions in remote locations. Second, due to the fact that the developed WRITE System is capable of continuously collecting and sending the on-site construction productivity data, construction managers and engineers now have a new technology to determine the on-site construction productivity in real time. Third, integrating the benchmarking data and the WRITE System data makes it possible for the construction managers and engineers to continuously improve the on-site construction productivity in real time. With advancements made by utilizing the WRITE System, communication and coordination will be improved at the construction project sites, which enhance the contractors' capability of managing construction projects.

This research project can be extended in several ways. First, because the process of determining working status using the live images obtained from the WRITE System is time-consuming and subject to human error and bias, there is a need to develop an algorithm to automatically classify working or nonworking status. Second, currently no database has been established for real-time productivity improvement using the WRITE System and benchmark data. There is no standard process for management to increase or decrease the crew size during the construction. Third, validation for the developed procedure needs to be performed on other construction operations in the future research projects.

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Development of Mix Design Process for Cold In-Place Recycling

Hosin “David” Lee
Department of Civil and Environmental Engineering
University of Iowa
Public Policy Center
227 South Quadrangle
Iowa City, IA 52241-1192
hlee@engineering.uiowa.edu

Yongjoo “Thomas” Kim
Department of Civil and Environmental Engineering
University of Iowa
Public Policy Center
227 South Quadrangle
Iowa City, IA 52241-1192
yongjkim@engineering.uiowa.edu

ABSTRACT

Cold in-place recycling using emulsified asphalt (CIR-emulsion) is a recycling process that evolved during the late 1980s. The need for a CIR-emulsion mixture with specific engineering properties calls for the use of a mix design. However, no standard mix design is available for CIR-emulsion in Iowa. Some cold in-place recycling using engineered emulsion (CIR-EE) mix design procedures are complex and require special equipment that is not commonly available. Recently, a new mix design procedure was developed for CIR. As a part of CIR mix design process, the simple performance tests, which include dynamic modulus test, dynamic creep test, and static creep test, were conducted to evaluate the performance of CIR mixtures at various testing temperatures and loading conditions. The optimum emulsified asphalt contents were found near 1.0%. Dynamic modulus, flow number, and flow time of CIR-emulsion mixtures using CSS-1h were generally higher than those of HFMS-2p. Flow number and flow time of CIR-emulsion using recycled asphalt pavement (RAP) materials with softer residual asphalt was higher than those of CIR-emulsion using RAP materials with harder residual asphalt. The dynamic modulus, flow number, and flow time values were affected by both emulsion types and residual asphalt stiffness of RAP materials.

Key words: cold in-place recycling—dynamic modulus—emulsified asphalt—flow number—flow time

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Modeling of Chip Seal Performance on Kansas Highways

Litao Liu
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
litao@ksu.edu

Rick Miller
Kansas Department of Transportation
Materials & Research Center, 2300 Van Buren
Topeka, KS 66611
rick@ksdot.org

Mustaque Hossain
Department of Civil Engineering
Kansas State University
2124 Fiedler Hall
Manhattan, KS 66506
mustak@ksu.edu

ABSTRACT

Due to traffic loading and weathering, highway pavements deteriorate starting immediately after construction. Many rehabilitation methods are available to bring deteriorated pavements back to an appropriate level of service for road users. One of these methods, chip seal, has been widely used in many states, including Kansas. This study evaluated the performance of chip seal treatments in Kansas. The study used pavement condition data from the Pavement Management Information System (PMIS) database. The data include detailed distress information of most chip seal rehabilitation projects on Kansas highways from the last two decades. The multiple linear regression method was used to develop linear models that can predict distress progression and variation in performance of chip-sealed pavement. The effect of age of pavements and traffic loading on the performance of chip seals was also evaluated. Appropriate models for predicting roughness and rutting have been developed and validated. Models for transverse and fatigue cracking are not encouraging.

Key words: chip seal—distress progression—PMIS

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INTRODUCTION

Timely preventive maintenance (PM) can preserve pavements above minimum acceptance serviceability level. The National Cooperative Highway Research Program (NCHRP) defines PM as "a program strategy intended to arrest light deterioration, retard progressive failures, and reduce the need for routine maintenance and service activities" (Gransberg and James 2005). Chip seal is one of the PM techniques that can play an important role in deferring the need for major rehabilitation (Abdul-Malak, Fowler & Meyer 1993).

In Kansas, the PM treatments adopted by the Kansas Department of Transportation (KDOT) include thin overlay, ultra-thin bonded bituminous surface (Novachip), chip seal, and slurry seal. The better the pavement condition when these treatments are applied, the longer the treatments will last, and the more cost-effective these treatments are. Chip seal is a relatively cost-effective preventive maintenance technique compared to other thin surface treatments since the cost of chip seal is about one-fifth to one-half the cost of a 2-in. overlay (Chen, Lin, and Luo 2005).

PROBLEM STATEMENT

Estimating the effectiveness of PM techniques is useful for PM purposes. By modeling the performance of a treatment, it is possible to determine the timing at which subsequent activities would be necessary. To achieve that, a long-term, continuous monitoring of distresses, such as roughness and cracking, is needed to determine relative effects of certain external factors and to predict future pavement performance. Performance models for flexible PM treatments, chip seal, flush seal, and sand seal treatments had been developed in the past (Sebaaly, Lani, and Hand 1995), but because of the localized nature of the materials, the models are not transferable.

Pavement design engineers can use these models to check the validity of their design procedures and the appropriateness of various assumptions that are made during design processes. Material engineers can verify whether a given type of material is appropriate for anticipated traffic and environmental conditions. As a result, design and construction practices can be altered to produce longer-lasting pavements. Pavement management engineers will gain the most from such studies because they are usually responsible for recommending various maintenance alternatives for specific applications.

KDOT maintains a comprehensive Pavement Management Information System (PMIS) database that contains detailed information related to section characteristics, historical distress data, performance estimation data, and traffic-related data. The PMIS database is generated from the information collected during the annual pavement condition survey conducted by KDOT staff. These condition survey data for the highway sections that have been treated with chip seal were extracted from the PMIS database for performance modeling in this study. The test sections in this study are consistent with respect to the distribution of age, pavement structure, pavement conditions prior to treatment, traffic loading, and environmental conditions. A total of 280 chip seal projects, distributed among the six administrative districts of KDOT, were identified and studied.

OBJECTIVE

The objective of this research project was to develop models capable of predicting progression of important distresses that have significant effects on the performance of chip seal-treated pavements. The distresses selected are roughness (International Roughness Index [IRI]), rutting, and cracking.

CHIP SEALS IN KANSAS

In Kansas during 1962 to 2006, at least 754 chip seal projects were completed, and 7,004 highway segments (1 mile length) have been maintained with chip seals. Many of these chip seal treatments had a 10-year design life as per the PMIS database. Because distress data on the chip seals before 1992 were not available in the database, only projects from 1992 to 2006 were studied in this research.

A total of 4,156 chip seal treatments were performed on 3,532 segments of 280 highways in Kansas from 1992 to 2006. About 16% of these segments were treated twice with chip seals, and 1% of them were treated three times, as can been seen in Figure 1.

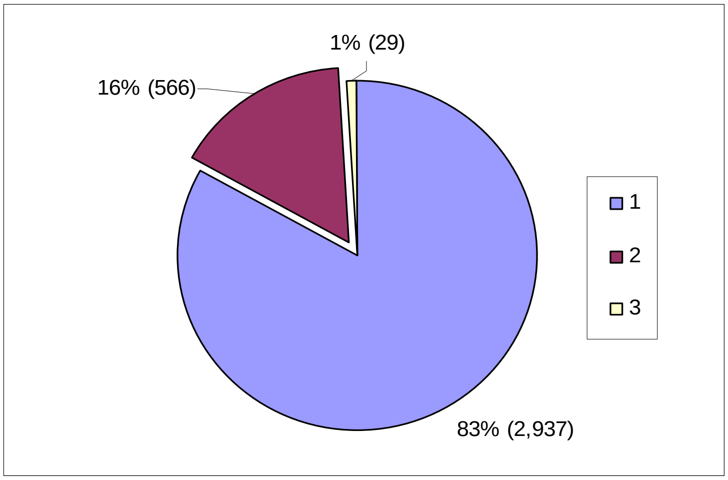


Figure 1. Proportion of segments by number of chip seal treatments

CONDITION SURVEY IN KANSAS

The distresses that are surveyed during the annual condition survey of asphalt-surfaced pavements include roughness, rutting, transverse cracking, and fatigue cracking. Though block cracking is also surveyed, this distress was found to be quite negligible because the majority of test sections could be observed to have hardly any block cracking. Therefore, it was not considered in this study. The effect of chip seals on distress progression was examined for three classes of highways: Interstate highway, U.S. highway, and Kansas state highway.

Roughness

Road roughness is an important attribute in evaluating pavement condition because of its effect on ride quality and vehicle operating costs (Miller, Vedula, and Hossain 2004). Currently, KDOT employs a South Dakota-type profilometer equipped with laser sensors to collect roughness data in terms of IRI.

Since IRI values are computed on both wheel paths, the average value expressed in in./mile was taken in this study.

Rutting

Currently, most state highway agencies integrate the measurement of rut depth as a part of the condition survey of bituminous and composite pavements mostly using the profilometer. The measurement of rut depth can be automatically conducted with a rut bar mounted on a vehicle with three or five or more sensors that are capable of measuring the profile data of road surfaces (Miller, Vedula, and Hossain 2004). In Kansas, KDOT use a three-point system in which data are collected in each wheel path and at mid-lane. In that case, the rut depth is calculated as the difference in elevation between the mid-lane measurement and the wheel path measurement.

Transverse Cracking

In the annual KDOT pavement condition survey, transverse cracks are manually measured by selecting three, 100 ft test sections from each 1-mile highway segment and counting the number of full lane-width cracks (centerline to edge on a two-lane road). The average crack number of the three, 100 ft sections is recorded as the extent of transverse cracking, which might be a one or two digit number, to the nearest 0.1 cracks. A transverse crack is judged and falls into one of the following four categories: T0, T1, T2, and T3, based on severity conditions that are coded as the following:

- T0: Sealed cracks with no roughness and sealant breaks less than 1 ft per lane
- T1: No roughness, 0.25 in. or wider with no secondary cracking; or any width with secondary cracking less than 4 ft per lane, or any width with a failed seal (1 or more ft per lane)
- T2: Any width with noticeable roughness due to depression or bump,also, cracks that have greater than 4 ft of secondary cracking but no roughness
- T3: Any width with significant roughness due to depression or bump,secondary cracking will be more severe than Code T2.
- Transverse cracking for a segment then expressed as EqTCR, which is the equivalent number of “T3” cracks expected per 100 ft segment

Fatigue Cracking

In Kansas, fatigue cracking is measured manually by observing the amount of fatigue cracking on three, 100 ft test sections for each 1 mile (normally) highway segment during annual pavement condition surveys. Alligator cracking is recorded in the unit of linear feet/100 ft, and the extent must exceed five feet to be counted. The average value is reported for each segment with one or more of the four severity levels (FC1, FC2, FC3, and FC4), which are coded as the following:

- FC1: Hairline alligator cracking, pieces not removable
- FC2: Alligator cracking, pieces not removable, cracks spalled
- FC3: Alligator cracking, pieces are loose and removable, pavement may pump
- FC4: Pavement has shoved forming a ridge of material adjacent to the wheel path

The fatigue cracking is then described as EqFCR, which is the equivalent number of “FC4” cracks per 100 ft segment.

METHODOLOGY

The multiple linear regression method and the Statistical Analysis System (SAS) software were used in this research. Multiple linear regression is an extension of simple linear regression and can be used to account for the effects of several independent variables simultaneously. The general multiple linear regression model can be defined in terms of X variables in the following form (Weisherb 2005):

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \ldots + \beta_p X_p\,,$$

where Y = dependent variable; β_0 = equation constant; $\beta_1, \ldots, \beta_p$ = partial regression coefficients; and $X_1, \ldots, X_p$ = independent variables.

R squared (R^2) value, which is the coefficient of determination in linear regression, gives the proportion of variability in Y explained by regression on a set of explanatory variables. It can also be interpreted as the square of correlation between observed values of Y versus fitted values. The value of R^2 is in a range of 0 to 1, with 1 indicating that a fitted model perfectly explains the response and 0 indicating that a fitted model cannot explain the response. The stepwise variable selection method was used in this study to eliminate those variables that did not meet the 10% significance level. In the regression process, outliers, defined as the data point that has students' residuals over 3.0, were eliminated from the data set to make the fitted model more precise.

DATA

Data used for the multiple linear regression analysis was extracted from the PMIS database by selecting highway segments treated with chip seals. Traffic data corresponding to each highway segment were also obtained. The Pavement Management System (PMS) segments in Kansas (usually 1 mile) belonging to the same road were combined and average distress and traffic values were calculated. Data on a single road during different service years were treated as different records by pavement age. The distribution of chip seal projects by annual average daily traffic (AADT) was also calculated. Three categories were used to classify AADT: low (≤400 veh/day), intermediate (between 400 and 4,000 veh/day), and high (>4000 veh/day). Ninety-five percent of the chip seal projects were done on low and intermediate volume roads.

RESULTS AND ANALYSIS

The variables used in the modeling process and their description are shown in Table 1. The models developed in this study were proposed to predict progression of distresses, not the initiation mostly caused by the problems from structural design and materials. Therefore, the first year distress data after chip-sealing were used as a variable.

Table 1. Description of variables used in linear regression

Variable	Description	Attribute
IRI	International roughness index, in./mile	Dependent variable
RD	Rut depth, in.	Dependent variable
EqTCR	Equivalent number of full-width transverse tracks per 100-ft highway segment	Dependent variable
EqFCR	Equivalent fatigue cracks per 100 ft highway segment, ft/100 ft	Dependent variable
InitIRI	The first year IRI value after chip-sealing	Independent variable
InitRD	The first year rut depth value after chip-sealing	Independent variable
InitTCR	The first year transverse crack value after chip-sealing	Independent variable
InitFCR	The first year fatigue crack value after chip-sealing	Independent variable
AGE	The service year of a chip seal treatment	Independent variable
AADT	Annual average daily traffic, vehs/day	Independent variable
ESAL	Cumulative equivalent 18 kip single axle loads	Independent variable
CLASS	Highway class, “1” for interstate highways, “2” for US highways, and “3” for “K” or state highways	Independent variable

Models

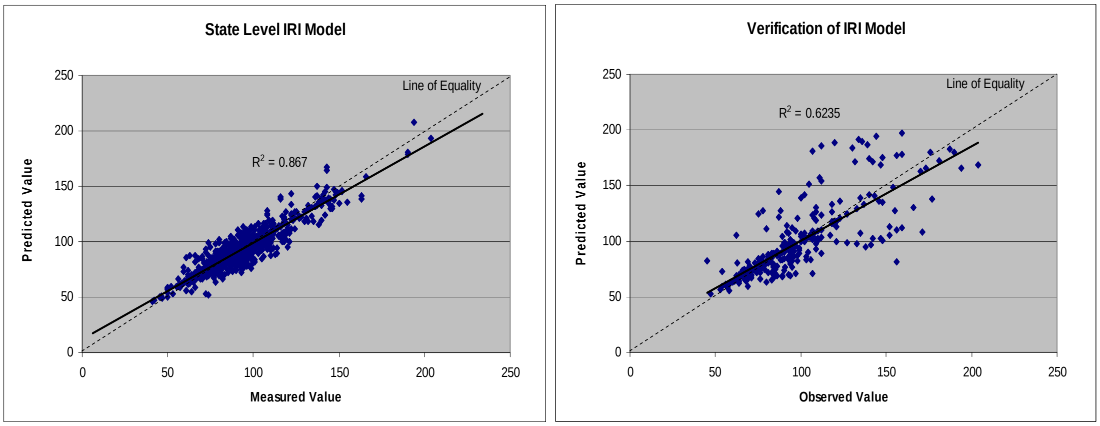
Roughness (IRI)

The fitted state-wide roughness model is shown in equation (1). InitIRI, age and class are the significant variables. The R^2 value is 0.867, indicating that 86.7% of the variation in the IRI values can be explained by these three variables in equation (1). Figure 2a displays the plot of the predicted IRI values against the measured IRI values.

$$\text{IRI} = 3.97091 + 0.89323\,(\text{InitIRI}) + 2.87797\,(\text{Age}) + 1.29244\,(\text{Class}) \qquad (1)$$

$$(n = 844;\ R^2 = 0.867)$$

Based on the parameter coefficients in equation (1), a one-year increment of chip seal age would probably cause about a 3 in. increase of roughness. Road class also has a direct relationship with roughness condition. For example, in the same year after chip sealing, a “K” or state highway is likely to become 1.29 in. rougher than a U.S. highway. The fitted model in equation (1) was validated using data from Districts 5 and 6, as shown in Figure 2b. An R^2 value of 0.624 was achieved, so 62.4% of the variability of IRI values in Districts 5 and 6 can be explained using equation (1). Data points are evenly distributed around the line of equality and fairly close to the equality line for the data values below 100 in./mile. The fitted model is adequate for predicting the IRI progression after chip sealing on highways in Kansas, but it should be used with caution for those roads where severe roughness problems are observed.



(a) **(b)**

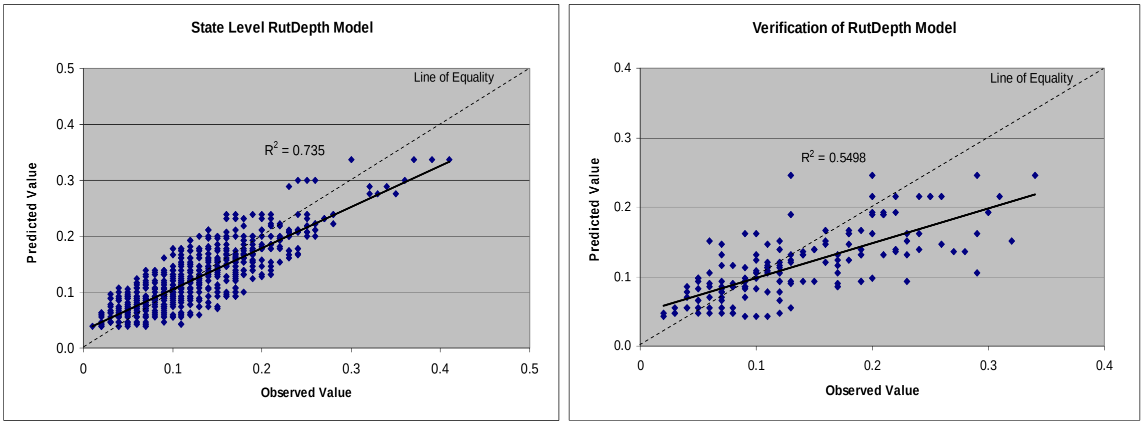
Figure 2. Fitted IRI model and validation

Rutting

The derived model for predicting progression of rut depth is illustrated in equation (2). There are two significant parameters, InitRD and Road Class, at a 10% level of significance. The equation shows that the rut depth of chip-seal–treated pavements is dependent on rutting at the time of chip seal but independent of pavement age and traffic loading. The R^2 value shows that 73.5% of the variations in rut depth of a project can be explained by the model in equation (2) if the first-year rut depth value and road class are known. Figure 3a shows the predicted and observed values.

$$RD = 0.03621 + 0.76501\,(InitRD) - 0.00404\,(Class) \qquad (2)$$
$$(n = 848;\ R^2 = 0.735)$$

Again, the rut depth data from Districts 5 and 6 were used to validate the model in equation (2). Figure 3b illustrates the calculated and measured rut depth values. About 58% of the calculated values matched the observations in the database.



(a) **(b)**

Figure 3. Fitted rutting model and validation

In order to predict the progression of rutting on chip seal-treated pavements more precisely, more factors, such as material type, pavement thickness, etc., need to be considered. This may also indicate that chip seal is not a very good treatment for mitigating rutting on asphalt-surfaced pavements.

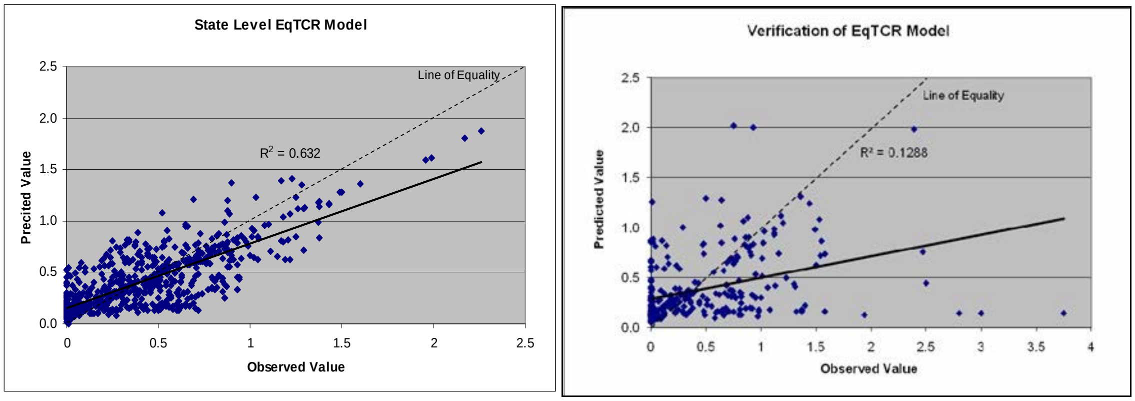
Transverse Cracking

Equation (3) shows the prediction model for predicting development of transverse cracks (designated as EqTCR) after chip seal applications.

$$\text{EqTCR} = -0.0765 + 0.7833\,(\text{InitTCR}) + 0.0175\,(\text{Age}) + 0.0561\,(\text{Class}) \quad (3)$$
$$(n = 722;\ R^2 = 0.633)$$

Besides the first-year transverse cracking value, age and road class are also significant at a 10% level of significance. The R^2 value is 0.632. Thus, 63.2% of variations in the observed transverse cracking values can be explained by the first-year EqTCR value, age, and class of highways. The equation shows that transverse crack development has a direct relationship with the age of chip seal. Lower functional class highways tend to have more transverse cracks than higher class ones. The plot of predicted EqTCR values versus observed values is shown in Figure 4a. The trend line implies that the developed model is very likely to underpredict transverse cracking conditions when actual values are larger than 0.5.

Figure 4b shows the validation plot for equation (3) using data from Districts 5 and 6. A number of data points are far away from the line of equality, resulting in a very low R^2 value of 12.9%. Therefore, this model has a very limited ability for predicting progression of transverse cracks on chip-seal–treated pavements. This model needs to be used with caution, especially for highway segments with large first-year EqTCR values. More factors, such as material types, pavement thickness, and characteristics prior to chip-sealing, might need to be considered for developing a better prediction model for EqTCR.



(a) **(b)**

Figure 4. Fitted EqTCR model and verification

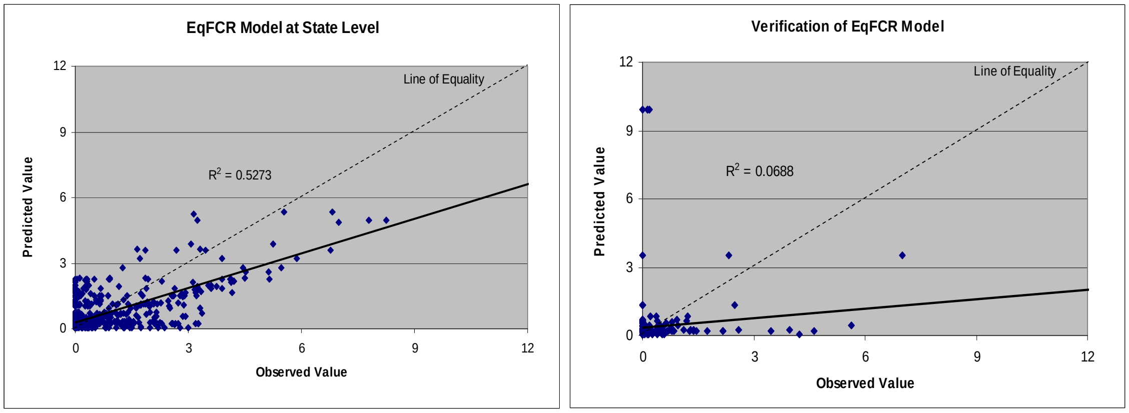
Fatigue Cracking

Equation (4) is the linear model for the other cracking distress variable in PMIS, fatigue cracking (designated as EqFCR).

$$\text{EqFCR} = -0.24839 + 0.49664\,(\text{InitFCR}) + 0.00008\,(\text{ESAL}) + 0.15381\,(\text{Class}) \quad (4)$$
$$(n = 804;\ R^2 = 0.527)$$

Besides the first-year fatigue cracking value, equivalent single axle loads (ESAL) and highway class are the significant variables to predict fatigue cracking deterioration after chip sealing. Age is not significant. This may imply that fatigue cracking is more dependent on traffic loading than time. An R^2 value of 0.527 was obtained for this model. Figure 5a shows the plot of predicted EqFCR values versus observed values. A somewhat large scatter of data can be observed.

Figure 5b displays the predicted EqFCR values versus observed values for Districts 5 and 6. As can be seen, a very low R^2 value of 6.9% was obtained, suggesting a lack of applicability of this model to fit the whole data set. A number of roadways had no or very slight fatigue crack deterioration after chip-sealing, while some roadways had very significant fatigue crack development. This caused difficulties in modeling progression of fatigue cracking for chip seal-treated pavements. Again, these results may indicate that chip seal is not an appropriate treatment for fatigue-cracked pavements.



(a) **(b)**

Figure 5. Fitted EqFCR model and validation

CONCLUSIONS

Multiple linear regression models were developed for predicting distress progression on chip-sealed pavements using data available in the PMIS database. The models were validated by data not used in the regression process. The IRI and rutting models appear to be reasonable. More variables need to be included in the transverse cracking and fatigue cracking models for better prediction. Chip seal does not appear to be a viable treatment for pavements with extensive transverse and fatigue cracking.

ACKNOWLEDGEMENT

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Determining Pavement Damage Cost Attributed to Truck Traffic in Southwest Kansas

Chunxiao Liu
Department of Civil, Environmental and
Architectural Engineering
University of Kansas
1530 W. 15th Street, 2150 Learned Hall
Lawrence, KS 66045
springlcxs@yahoo.com

Yingfeng Li
Texas Transportation Institute
Texas A&M University
3500 NW Loop 410, Suite 315
San Antonio, TX 78229
y-li@tamu.edu

Yong Bai
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
1530 W. 15th Street, 2150 Learned Hall
Lawrence, KS 66045
ybai@ku.edu

ABSTRACT

The southwest Kansas region is one of the centers in the United States for the production of processed beef and related industries. Trucks utilized to support these industries cause noteworthy damages to the regional highway pavements. The objective of this research project was to determine the pavement damage cost of a typical highway section in southwest Kansas attributed to the truck traffic for the processed beef and related industries. To achieve the project objective, the researchers first collected pavement data from the Kansas Pavement Management Information System. Then, they estimated the truck vehicle miles traveled that were generated by the industries on the selected highway section. Finally, using these data, the researchers determined the damage cost in 2007 dollar value of the selected highway section associated with the industries. During data analysis, the researchers adopted a systematic pavement damage estimation procedure that incorporated a time-decay model and a traffic-related damage model developed by American Association of State Highway and Transportation Officials and used in the Highway Economic Requirements System. Results of the analysis indicated that the damage cost was as high as $1,727 per mile per year on the 41-mile highway section attributed to the beef and related industries. Outcomes of this research will help highway agents assess highway pavement maintenance needs and set up maintenance priorities. In addition, the analysis results will be valuable for the determination of reasonable user costs.

Key words: cost—damage—Kansas—pavement—truck

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INTRODUCTION

Kansas is one of the leading states in the processed meat and related industries in the United States. It ranks first in number of cattle slaughtered nationwide, second in total number of cattle, and third in the number of cattle on feed and in red meat production by commercial slaughter plants in 2004 (USDA 2005). In Kansas, the southwest region plays a key role in the industries by having more than three hundred feed yards and several of the biggest meat processing plants in the nation. The processed meat and related industries use trucks as the dominant transportation mode, which speeds up highway pavement damage in the region and causes concerns including air pollution, fuel consumption, safety, and congestion.

Previously, the Kansas Department of Transportation (KDOT) initiated a research project to study the transportation modes used in the processed beef and related industries in the southwest Kansas region and their impacts on local and regional economies (Bai et al. 2007). The researchers of that project collected related transportation data and estimated truck vehicle miles traveled (VMT) generated by the industries in the region. It became particularly interesting for government agencies and the industries to determine the highway pavement damage costs associated with the VMT, which will be valuable for stakeholders to understand the highway pavement damage costs attributable to the processed beef and related industries and to promote the utilization of other transportation modes. The knowledge can be also used to assess highway pavement maintenance needs and to set up maintenance priorities. In addition, the analysis results will be valuable for the determination of reasonable user costs.

LITERATURE REVIEW

The highway pavement damage cost analysis required knowledge of various subjects, including the processed meat and related industries in southwest Kansas, highway maintenance theories and practices, heavy-vehicle impact on pavement damage, fundamentals of pavement management systems, and pavement performance/condition predictions. To date, various agencies and individuals have performed studies that involved estimation of highway damage associated with certain types of heavy vehicles. For instance, some researchers have studied the road damage costs due to abandoned short-line railroads (Babcock et al. 2003; Russell et al. 1996), changes in regulations governing truck weights and dimensions (Hajek et al. 1998), and proposed drawdown usage of major waterways (Lenzi et al. 1996). The authors did not find studies that analyzed pavement damage attributed to a certain type of industry, such as the processed beef industry.

Some of the popular models currently used fall in different categories based on the model development methodologies, including Bayesian models, probabilistic models, empirical models, mechanistic-empirical models, and mechanized models (AASHTO 2002). Among these, empirical models have been widely used in pavement damage studies because of their maturity and reasonable accuracy. Tolliver (2000) developed a cost estimation procedure that utilized empirical models relating the physical lives of pavements to truck-axle loads and environmental factors. These empirical models were originally developed from American Association of State Highway Officials (AASHO) road test data and later incorporated into the pavement design procedure developed by AASHTO and followed by many state Departments of Transportation (DOT) including KDOT. In addition, the equations and functions used in these models have also been embedded in the pavement deterioration model of Highway Economic Requirements System (HERS), a comprehensive highway performance model used by the Federal Highway Administration (FHWA) to develop testimony for Congress on the status of the nation’s highways and bridges (FHWA 2002). The data required for the analysis procedure were available in the Pavement Management Information System (PMIS) database managed by each state DOT.

PAVEMENT PERFORMANCE PREDICTION MODELS

Two types of deterioration models were utilized in this study: a time decay model and an equivalent single axle load (ESAL). The former addresses the pavement damage caused by environmental factors, and the latter analyzes the pavement damage due to truck traffic. These two models are briefly described below. A detailed description of the equations used in the models can be found in (Liu 2007).

Traffic-Related Pavement Damage Model

Calculation of ESAL Factor

The ratio of decline in pavement serviceability relative to the maximum tolerable decline in serviceability can be used as a damage index to measure pavement damage:

$$\frac{P_I - P}{P_I - P_T} = \left(\frac{N}{\tau}\right)^{\beta}, \tag{1}$$

where P_I = initial pavement serviceability rating, P_T = terminal pavement serviceability rating, P = current pavement serviceability rating, N = the number of passes of an axle group of specified weight and configuration (e.g., a single 18-kip axle), τ = the number of axle passes at which the pavement reaches failure (i.e., the theoretical life of the pavement), and β = deterioration rate for a given axle.

For flexible pavements, the unknown parameter β in equation (1) can be estimated through regression equations developed based on AASHO road test data, as shown in equation (2). Using the single 18-kip axle as a reference axle, the parameters can then be computed, as shown in equation (3). Based on these parameters, equations (4) and (5) are derived to compute the equivalent rate of flexible pavement deterioration caused by a single axle load in comparison to an 18-kip axle load and by a tandem axle group.

$$\beta = 0.4 + \frac{0.081(L_1 + L_2)^{3.23}}{(SN+1)^{5.19}\, L_2^{3.23}}, \tag{2}$$

$$\beta_{18} = 0.4 + \frac{0.081(18+1)^{3.23}}{(SN+1)^{5.19} 1^{3.23}} = 0.4 + \frac{1094}{(SN+1)^{5.19}}, \tag{3}$$

$$\log_{10}(ESAL) = 4.79 \log_{10}\left(\frac{L_1+1}{18+1}\right) + \frac{G}{\beta_{18}} - \frac{G}{\beta}, \tag{4}$$

$$\log_{10}(ESAL) = 4.79 \log_{10}\left(\frac{L_2+2}{18+1}\right) - 4.33 \log_{10}(2) + \frac{G}{\beta_{18}} - \frac{G}{\beta}, \tag{5}$$

where L_1 = axle load in thousand pounds or kips, L_2 = axle type (1 for single, 2 for a tandem, and 3 for triple axles), SN = structural number of flexible pavement section, and β_{18} = deterioration rate for a single 18-kip axle load. In both equations (4) and (5),

$$G = \log_{10}\left(\frac{P_I - P_T}{P_I - 1.5}\right). \tag{6}$$

The ESAL factor *n* is computed by taking the inverse logarithm of the appropriate expression, as shown in equation (7):

$$n = 10^{\log_{10}(ESAL)}. \tag{7}$$

The above equations are derived for flexible pavements. For rigid pavements, equation (8) is used to convert rates of deterioration to rigid pavement ESAL for single axle loads, and equation (9) is utilized to compute the equivalent rate of pavement deterioration caused by a given tandem axle group. *G* is computed using equation (10), and *n* is computed using equation (11).

$$\log_{10}(ESAL) = 4.62\log_{10}\left(\frac{L_1+1}{18+1}\right) + \frac{G}{\beta_{18}} - \frac{G}{\beta}, \tag{8}$$

$$\log_{10}(ESAL) = 4.62\log_{10}\left(\frac{L_2+2}{18+1}\right) - 3.28\log_{10}(2) + \frac{G}{\beta_{18}} - \frac{G}{\beta}, \tag{9}$$

$$G = \log_{10}\left(\frac{P_I - P_T}{P_I - 1.5}\right), \tag{10}$$

$$n = 10^{\log_{10}(ESAL)}. \tag{11}$$

Calculation of ESAL Life

The ESAL life of a pavement is the cumulative number of ESAL that the pavement can accommodate before it needs to be rehabilitated. The following equations were included in HERS and can be used to calculate the cumulative ESAL, or *LGE*:

$$LGE = XA + \frac{XG}{XB}, \tag{12}$$

$$SNA = SN + \sqrt{\frac{6}{SN}}, \tag{13}$$

$$XB = 0.4 + \left(\frac{1{,}094}{SNA}\right)^{5.19}, \tag{14}$$

$$XG = \log_{10}\left(\frac{P_I - P_T}{3.5}\right), \tag{15}$$

$$XA = 9.36\log_{10}(SNA) - 0.2, \tag{16}$$

where *LGE* = cumulative ESAL that a pavement section can accommodate before reaching its terminal serviceability rating (in logarithmic form), *XB* = rate at which a pavement's life is consumed with the accumulation of ESAL, *XG* = pavement serviceability loss in terms of the maximum tolerable pavement Present Serviceability Rating (PSR) loss (from P_I to P_T), *XA* = theoretical life of newly constructed pavement in ESAL, and *SNA* = converted pavement structural number.

The actual life cycle of a flexible pavement is computed by taking the inverse logarithm of *LGE*:

$$ESALlifecycle = 10^{LGE}\,. \tag{17}$$

For rigid pavements, the theoretical life is a function of the thickness of the concrete slab (*d*) and can be calculated using the following equations:

$$ESALlifecycle = 10^{LGE}\,, \tag{18}$$

where

$$XA = 7.35\log_{10}(d+1)0.06\,, \tag{19}$$

$$XB = 1 + \frac{16{,}240{,}000}{(d+1)^{8.46}}\,, \tag{20}$$

$$XG = \log_{10}\frac{(P_I - P_T)}{3.5}\,, \tag{21}$$

$$LGE = XA + \frac{XG}{XB}\cdot \tag{22}$$

Time-Related Deterioration of Pavements

As discussed in (Tolliver 2000), the decay rate due to environmental conditions can be estimated using the following equation:

$$\delta = \frac{-\ln\left(\dfrac{P_T}{P_I}\right)}{L}\,, \tag{23}$$

where δ = decay rate due to environmental losses, P_T = terminal PSR, P_I = initial PSR, and *L* = maximum feasible life of pavement section.

Given the decay rate, the PSR due to the environmental impact can be computed as

$$P_E = P_I \times e^{(-t\delta)}\,, \tag{24}$$

where P_E = PSR due to the environment impact and *t* = typical pavement performance period.

Calculation of Structural Numbers

For flexible pavements, the structural number, or *SN*, can be determined using equation (25) (Tolliver 2000):

$$SN = a_1 d_1 + a_1^* d_1^* + a_2 d_2 + a_3 d_3 . \tag{25}$$

The structural number for a composite pavement, particularly for asphalt concrete (AC) overlay of portland concrete cement (PCC) slab, can be calculated by equation (26) (AASHTO 1993):

$$SN = SN_{ol} + SN_{eff} = \sum a_{oli} d_{oli} + a_{eff} D_{eff} m_{eff} \ , \tag{26}$$

where SN_{ol} = overlay structural number; SN_{eff} = effective structural number of the existing slab pavement; d_1, d_1^*, d_2, d_3, and d_{oli} = thickness of surface layer, base course, base layer, subbase layer, and overlay layer; D_{eff} = thickness of fractured PCC slab layer (inches); a_1, a_1^*, a_2, a_3, a_{oli}, and a_{eff} = layer coefficient; and m_{eff} = drainage coefficients for a fractured PCC slab.

DATA COLLECTION

This project focused on the highway section of US 50/400 between Dodge City to Garden City, Kansas, one of the major highway sections carrying heavy truck traffic generated by the beef and related industries (Bai et al. 2007). Figure 1 shows the selected highway section, which was further divided into four pavement segments (PS) based on different pavement characteristics.

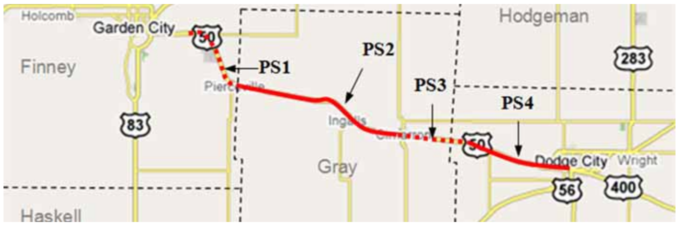

Figure 1. Location of highway segment under study

The estimation of highway pavement damage costs required several types of data, including truck parameters, pavement characteristics data, and pavement maintenance cost data. The previous study (Bai et al. 2007) showed that 3-S2 tractor-trailer combinations were the predominant truck used by the processed beef and related industries. In addition, 82% of the truck combinations on the nationwide highway system are 3-S2 trucks (USDOT 2000). Therefore, the 3-S2 model was used as the truck type for this study. This configuration has two axles on the semi-trailer and three axles on the tractor. It has a loading configuration of 10/35/35 meaning that the tractor unit applies a 10,000 pound load to the front axle, and each of two tandem axle groups under the trailer support 35,000 pounds of weight—a maximum legal gross vehicle weight (GVW) is 80,000 pounds. The pavement characteristics data of the four segments were collected from KDOT's PMIS. KDOT also provided information about the recent maintenance costs for the four pavement segments. Table 1 describes each of the pavement segments and recent maintenance projects.

Table 1. US-50/400 pavement basic data in Finney County

PS	Beginning Point	Ending Point	Length	Type	SN	Cost Data		
						Year	Project	Total Cost ($)
1	1.4 km E Garden City	ECoL	16.3 km/10.13 mi	FDBIT	5.4	2005	K-6374-01	15,908,221
2	WCoL	WCL Cimarron	29.2 km/18.14 mi	PDBIT	3.05	1985	K-1764-01	3,074,770
						1997	K-6190-01	999,522
						2004	K-9324-01	1,653,059
3	ECL Cimarron	ECoL	6.9 km/4.29 mi	FDBIT	N/A	1992	K-4038-01	1,685,548
						2001	K-8146-01	746,771
4	WCoL	Jct. US-50 & US-400	13.8 km/8.57 mi	COMP	N/A	1981	K-1228-01	3,595,654
						1989	K-3643-01	272,433
						1992	K-4039-01	627,261
						1992	K-4609-01	448,390
						2001	K-8145-01	220,173
						2003	K-8145-02	1,730,826

ECoL: East County Line; WCoL: West County Line; WCL/ECL Cimarron: West/East City Limits of Cimarron; FDBIT: full depth bituminous pavement; PDBIT: partial depth bituminous pavement; and COMP: composite pavement.

TRUCK VMT ASSOCIATED WITH THE PROCESSED BEEF AND RELATED INDUSTRIES

The study of pavement damage costs involved two critical steps: the estimation of truck VMT generated by the processed beef and related industries and the computation of the damage on the selected highway segments caused by truck VTM. The truck VMT was determined by Bai et al. (2007) in the following six categories:

- Transporting feeder cattle to feed yards in southwest Kansas
- Transporting feed grain to feed yards in southwest Kansas
- Transporting finished cattle to meat processing plants in southwest Kansas
- Transporting boxed beef to customers in the United States
- Transporting meat byproducts
- Transporting boxed beef to oversea market

To estimate the processed beef and related truck traffic in southwest Kansas, the origins and destinations of each shipment component were identified first. Based on the identified origins and destinations, the beef-related truck traffic was then distributed to the major highways in the southwest Kansas area using TransCAD software. Routes were selected based on shortest distance, giving priority to the principal highways. There were 369 feed yards within the 24 counties of the southwest Kansas region that served as major origins or destinations for shipments. To facilitate the analyses, the feed yards within each county were aggregated to a centroid on a principal highway located in the county considering factors such as feed yard sizes and distribution. The shipments generated by the industries from major highways to each individual feed yard used only local roadways, and thus, were not considered in the study. Figure 2 shows the procedure for estimating the total VMT generated by the processed beef and related industries, followed by the estimated truck VMT in Table 2 (Bai et al. 2007, Liu 2007).

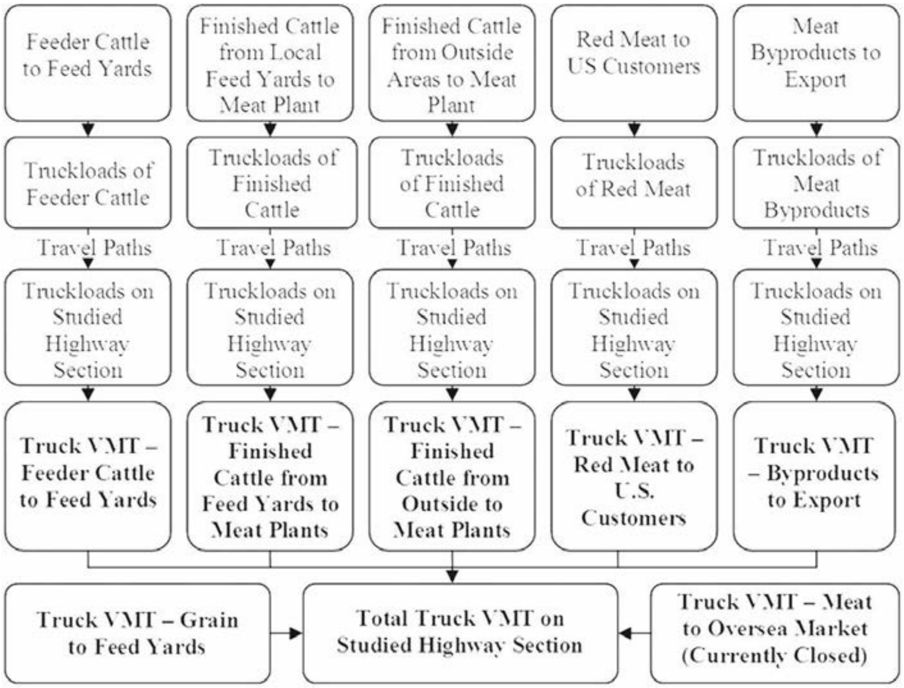
Feeder Cattle to Feed Yards
Finished Cattle from Local Feed Yards to Meat Plant
Finished Cattle from Outside Areas to Meat Plant
Red Meat to US Customers
Meat Byproducts to Export
Truckloads of Feeder Cattle
Truckloads of Finished Cattle
Truckloads of Finished Cattle
Truckloads of Red Meat
Truckloads of Meat Byproducts
Travel Paths
Travel Paths
Travel Paths
Travel Paths
Travel Paths
Truckloads on Studied Highway Section
Truckloads on Studied Highway Section
Truckloads on Studied Highway Section
Truckloads on Studied Highway Section
Truckloads on Studied Highway Section
Truck VMT – Feeder Cattle to Feed Yards
Truck VMT – Finished Cattle from Feed Yards to Meat Plants
Truck VMT – Finished Cattle from Outside to Meat Plants
Truck VMT – Red Meat to U.S. Customers
Truck VMT – Byproducts to Export
Truck VMT – Grain to Feed Yards
Total Truck VMT on Studied Highway Section
Truck VMT – Meat to Oversea Market (Currently Closed)

Figure 2. Procedure for estimating total annual truck VMT

Table 2. Total annual truck VMT on the studied pavement segments (one-way)

PS	Shipment	Annual Truckloads per PS	Total Annual Truckloads	Annual VMT per PS	Total Annual VMT
1	Feed cattle to feed yards	4,209	55,539	42,637	562,610
	Finished cattle to meat processing facilities	41,044		415,776	
	Boxed beef to U.S. customers	4,473		45,312	
	Byproducts to export destinations	5,813		58,886	
2	Feed cattle to feed yards	4,209	55,539	76,351	1,007,477
	Finished cattle to meat processing facilities	41,044		744,538	
	Boxed beef to U.S. customers	4,473		81,140	
	Byproducts to export destinations	5,813		105,448	
3	Feed cattle to feed yards	5,748	56,477	24,659	242,282
	Finished cattle to meat processing facilities	40,443		173,500	
	Boxed beef to U.S. customers	4,473		19,189	
	Byproducts to export destinations	5,813		24,938	
4	Feed cattle to feed yards	5,748	56,477	49,259	484,006
	Finished cattle to meat processing facilities	40,443		346,596	
	Boxed beef to U.S. customers	4,473		38,334	
	Byproducts to export destinations	5,813		49,817	

DAMAGE ATTRIBUTED TO TRUCK TRAFFIC OF BEEF INDUSTRIES

Cost Estimation Procedure

Two types of deterioration models were utilized in this study: a time decay model and an ESAL, or pavement damage model. The former addresses pavement damage caused by environmental factors, and the latter analyzed the pavement damage due to truck traffic. The loss of pavement serviceability attributed to the environmental factors was estimated first, and the rest of the serviceability loss was then assigned to truck axle loads. The total pavement damage cost associated with truck traffic generated by the processed beef and related industries was calculated in seven major steps (see Figure 3).

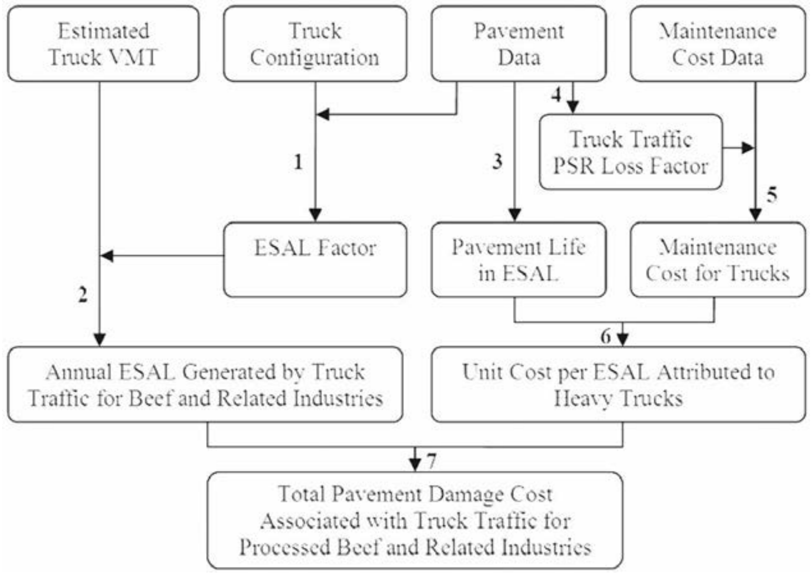
Estimated Truck VMT
Truck Configuration
Pavement Data
Maintenance Cost Data
4
Truck Traffic PSR Loss Factor
1
3
5
ESAL Factor
Pavement Life in ESAL
Maintenance Cost for Trucks
2
6
Annual ESAL Generated by Truck Traffic for Beef and Related Industries
Unit Cost per ESAL Attributed to Heavy Trucks
7
Total Pavement Damage Cost Associated with Truck Traffic for Processed Beef and Related Industries

Figure 3. Pavement damage cost analysis procedure

Calculation of ESAL Factors and Annual ESAL (Steps 1 and 2)

To calculate the pavement damage costs due to trucks associated with processed beef and related industries, it was necessary to calculate ESAL factors for the study truck type and pavements. Pavement structural numbers are key inputs for the calculation of ESAL factors. The numbers for pavement segments 1 and 2 were obtained directly from KDOT PMIS system as 5.4 and 3.05. However, the structure numbers for segments 3 and 4 had to be computed based on their pavement structural information. PS 3 is a flexible pavement, which is a full-depth asphalt pavement without a base layer. The subbase layer of this pavement segment is the subgrade (natural soil). According to this information, the layer coefficients a_1 and a_1^* were determined as 0.4 and 0.26, respectively, based on (Tolliver 2000). PS 4 is a composite pavement segment, which has a surface layer of 38 mm (1.5 in) Bituminous Mixtures (BM)-1T, a 151 mm (5.95 in) base course of hot mix asphalt (HMA), and a 178 mm (7.01 in) subbase layer of concrete pavement on the subgrade (natural soil). The layer coefficients a_{o1}, a_{o1}^* and a_{eff} for this pavement configuration were 0.4, 0.26, and 0.22, respectively (Liu 2007).

With the structural numbers known, the front axle ESAL factor was calculated first. For the 3-S2 trucks used in this study, the load applied to this axle was 10 kips. The initial and terminal PSR values of the study highway segments were 4.2 and 2.5, as used by KDOT for pavement management. A rear tandem axle ESAL factor for the 3-S2 truck was computed in the same manner as for the single axle, with a different load of 35 kips. The total ESAL factor value for a standard truck was the sum of the front single axle and two rear tandem axle groups (Σn). The annual ESAL for each pavement segment was then computed as the truck ESAL factor multiplied by the estimated annual truck VMT generated by the processed beef and related industries. Notice that the VMT values used for this calculation reflected round trips of the truck traffic by doubling those in Table 2. According to Bai et al. (2007), the majority of the trucks generated by the processed beef and related industries would carry other freight on the way back to maximize their profits. The VMT generated by the returning truck traffic were assumed to be the same as the one-way values calculated above. The final results of the ESAL factors and annual ESAL generated by the processed beef and related industries are listed in Table 3.

Table 3. Calculation of ESAL factors and annual ESAL

Pavement Segment	**PS 1**	**PS 2**	**PS 3**	**PS 4**
SN (equations 25 & 26)	5.4	3.05	4	3.69
P_I	4.2	4.2	4.2	4.2
P_T	2.5	2.5	2.5	2.5
L_1	10	10	10	10
	35	35	35	35
L_2	1	1	1	1
	2	2	2	2
β_{18} (equation 3)	0.472	1.17	0.658	0.759
β_{10} (equation 2]	0.412	0.532	0.444	0.461
β_{35} (equation 2)	0.466	1.106		0.73
$log_{10}^{(ESAL)}$ (equation 4)	-1.076	-0.931	-0.99	-0.966
	0.088	0.093	0.093	0.094
G (equation 6)	-0.201	-0.201	-0.201	-0.201
n (equation 7)	0.084	0.117	0.102	0.108
	1.226	1.239	1.24	1.241
Σn	1.31	1.356	1.342	1.349
VMT	1,125,220	2,014,954	484,564	968,012
Annual ESAL	1,473,852	2,731,694	650,209	1,306,016

Determination of the Pavement ESAL Lives (Step 3)

The maximum life of a pavement was defined in terms of tolerable decline in PSR. The highway segments that were studied were designed by KDOT at an initial PSR of 4.2 and a terminal PSR of 2.5—a maximum tolerable decline in PSR of 1.7. The life of the studied pavement segments in terms of traffic, or ESAL life, was determined using this maximum tolerable PSR decline. ESAL life is the total number of axle passes that would cause the pavement to decline to its terminal PSR irrespective of the time involved. The results of this step are shown in Table 4.

Table 4. Calculation of Pavement ESAL Life

Pavement Segment	**PS 1**	**PS 2**	**PS 3**	**PS 4**
SN (equations 25 & 26)	5.4	3.05	4	3.69
P_I	4.2	4.2	4.2	4.2
P_T	2.5	2.5	2.5	2.5
SNA (equation 13)	6.454	4.453	5.225	4.965
XA (equation 16)	7.38	5.871	6.521	6.314
XB (equation 14)	3.71E+11	2.55E+12	1.11E+12	1.45E+12
XG (equation 15)	-0.314	-0.314	-0.314	-0.314
LGE (equation 12)	7.38	5.871	6.521	6.314
ESAL Life (equation 17)	2.40E+07	7.43E+05	3.32E+06	2.06E+06
L	30	30	30	30
t	14	14	14	14
δ (equation 23)	0.008	0.008	0.008	0.008
P_E (equation 24)	3.78	3.78	3.78	3.78

Calculation of Annual per-Mile Maintenance Cost

The researchers first converted the maintenance data from KDOT to reflect annual per-mile maintenance cost in current dollars. In addition, although a maintenance project was performed in a specific year, the pavements actually decayed gradually. It was necessary to allocate the total maintenance costs to annual splits. Therefore, the researchers first converted the maintenance costs to the current 2007 dollar value, and then distributed the total project costs as annual per mile cost for each pavement segment.

Based on economic theories (Sullivan et al. 2003), the expense ($M_{ti}^{S} current\$$) of a pavement maintenance activity in the activity year (*ti*) on pavement segment *S* can be converted to the current 2007 dollar value (M_{ti}^{S}) given an interest rate (*r*) by equation (27)

$$M_{ti}^{S} = M_{ti}^{S} current\$ \times (1+r)^{2007-ti} \quad . \tag{27}$$

For this study, the interest rate was determined based on the Producer Price Index (PPI) data from 1981 to 2006 (USDL 2007). The average of the PPI change rate per year for construction materials and components is 2.68%, and the average of the PPI change rate per year for construction machinery and equipment is 2.62%. Therefore, 3% was used as the rounded average interest rate.

To compute average annual maintenance costs, it was necessary to determine the time period covered by each maintenance expense. In this study, the maintenance time period of each expense (M_{ti}^{S}) was considered as the interval in years (I_i) between two contiguous maintenance activities. Using the constant dollar smoothing method, annual maintenance spending (A_t^{S}) at time *t* on a pavement segment was computed using equation (28)

$$A_t^{S} = \frac{M_{ti}^{S}}{I_i} = \frac{\sum M_{ti}^{S}}{t_{i+1} - t_i} \cdot \tag{28}$$

According to the KDOT pavement management policy, the maximum feasible life of a pavement is 30 years. From KDOT’s latest Pavement Management System data (2007), the anticipated design life for full-depth asphalt pavement was 14 years before a maintenance action was needed. The anticipated life was six years before an action was needed after a light rehabilitation with any overlay less than 1.5 inches or surface recycle actions. The performance period of the studied pavement segments, in terms of the number of years after a new pavement segment is resurfaced, was considered as 14 years because none of the four segments had overlays less than 1.5 inches. Based on the information and equations described above, the researchers converted the maintenance costs to the 2007 value and then calculated the annual per-mile maintenance cost. The results of these calculations are summarized in Table 5.

Table 5. Maintenance costs in year 2007 value

Pavement Segment	Maintenance Costs				
	Year	**Project**	**Previous Dollar ($)**	**2007 Dollar ($)**	**Cost/mile/year ($)**
1	2005	K-6374-01	15,908,221	16,887,032	119,003
2	1985	K-1764-01	3,074,770	5,891,577	15,103
	1997	K-6190-01	999,522	1,343,274	
	2004	K-9324-01	1,653,059	1,806,342	
3	1992	K-4038-01	1,685,548	2,626,029	35,651
	2001	K-8146-01	746,771	891,684	
4	1981	K-1228-01	3,595,654	7,754,356	39,236
	1989	K-3643-01	272,433	463,799	
	1992	K-4039-01	627,261	977,252	
	1992	K-4609-01	448,390	698,577	
	2001	K-8145-01	220,173	262,898	
	2003	K-8145-02	1,730,826	1,948,060	
Overall Average Annual Per-Mile Cost in 2007 Value					$47,864

Per-ESAL Unit Cost and Total Cost Attributed to the Industries (Steps 4 to 7)

The annual per-mile maintenance expenditure for each of the segments calculated here was due to both environmental factors and truck traffic. It is necessary to identify the proportion of the pavement damage caused by truck traffic only. The PSR loss of each segment due to environmental factors for the design period of 14 years was determined using the time decay model (equations [23] and [24]), given KDOT's policy for initial PSR of 4.2 and terminal PSR of 2.5, with a maximum feasible life of 30 years. As a result, the percent of the pavement maintenance costs due to truck traffic was estimated as 75%.

Thus, the average annual maintenance cost per mile of each pavement segment needs to be adjusted by a factor of 75% to isolate the damage solely attributed to truck traffic. Knowing the costs attributed to truck traffic, the unit cost per ESAL for each pavement segment was computed as the average per-mile maintenance cost divided by the ESAL life of the same segment. In addition, the total maintenance cost that is attributed to the truck traffic of the beef and related industries in the region was calculated as the per-axle cost multiplied by the total ESAL generated by the truck traffic associated with the industries. Table 6 shows the calculated average annual per mile cost caused by heavy trucks and the unit cost per ESAL for each pavement segment. As listed in the table, the average annual maintenance cost on the study highway section that was due to the damage caused by truck traffic associated with the processed beef and related industries was estimated as $71,019, an average annual per-mile cost of $1,727.

Table 6. Average annual per-mile cost, per ESAL cost, and total cost attributed to the industries

Pavement Segment	Segment Length (mi)	AAPMC[1] ($)	Pavement ESAL Life	PEMC[2]	Annual ESAL[3]	Pavement Damage Cost[4] ($)
1	10.13	89,252	23991498	0.004	1,473,852	5,483
2	18.14	11,328	743019	0.015	2,731,694	41,645
3	4.29	26,738	3319627	0.008	650,209	5,237
4	8.57	29,427	2060297	0.014	1,306,016	18,653
Total	41.13 mi	-	-	-	-	$71,019

[1] Average annual per-mile maintenance cost in 2007 dollar value that was caused by heavy trucks;

[2] Per-ESAL maintenance cost in 2007 dollar value

[3] The annual ESAL generated by the truck traffic of the processed beef and related industries

[4] Pavement damage cost in 2007 dollar value that was caused by the truck traffic associated with the processed beef and related industries.

CONCLUSION

The southwest Kansas region plays a key role in the processed beef and related industries by having more than three hundred feed yards and several of the biggest meat processing plants in the nation. Traditionally, the industries have been primarily using heavy trucks (e.g., tractor-trailers) for transporting processed meat, meat byproducts, grain, and other related products. With the continuous growth of these industries, the truck traffic has become one of the major causes for the damage of the local highway network. This paper presents a study of the highway pavement damage cost attributed to the processed beef and related industries. The results of this study can be of particular interest to stakeholders including KDOT for better understanding highway damage sources and assessing highway user costs. The results can also be valuable for identifying and promoting more cost-effective transportation modes for the beef processing and related industries.

The researchers analyzed the damage of a pavement section on US 50/400 that was attributable to the heavy trucks of the southwest Kansas processed beef and related industries. In the study, the researchers used the truck VMT generated by the industries, and then determined the associated pavement damage costs using a systematic procedure that accommodated models developed by AASHTO and used in HERS. The analysis results showed that the total pavement damage cost on the 41-mile highway section was $71,019 per year, or $1,727 per mile per year, that was associated with processed beef and related industries in the region. If the same truck traffic were carried on other major highways in the region (1,835 miles), the total damage cost attributed to the processed meat and related industries would be as high as about $3.2 million per year.

It should be pointed out that a few factors may affect the accuracy of this study. In the analysis, the researchers assumed that all trucks carried standard loads. During the data collection, it was found that a nontrivial percentage of the trucks were overloaded to reduce the shipping costs, which would cause much more severe damage to highways. However, the accurate overloading information could not be obtained and thus the factor was not considered in this study. In addition, the study used the estimated truck VMT data from a previous research project because the actual counts of the truck traffic generated by the processed beef and related industries on the selected highway section were not available. For future studies, more reliable data should be used to increase the accuracy of analysis results.

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Methods at Iowa DOT—Flooded Backfill and Plastic Pipe

Deanna Maifield
Office of Design
Iowa Department of Transportation
800 Lincoln Way
Ames, Iowa 50010
deanna.maifield@dot.iowa.gov

Dean F. Herbst
Office of Construction
Iowa Department of Transportation
800 Lincoln Way
Ames, Iowa 50010
Dean.Herbst@dot.iowa.gov

ABSTRACT

This presentation discusses methods used at the Iowa Department of Transportation (Iowa DOT) to address the problem of flooded backfill and plastic pipe.

Key words: flooded backfill—Iowa DOT—plastic pipe

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Development of Accelerated Superpave Mix Testing Models

Chandra Manandhar
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
cmanandh@ksu.edu

Paul Nelson
Department of Statistics
Kansas State University
104 Dickens Hall
Manhattan, KS 66506
nels@ksu.edu

Mustaque Hossain
Department of Civil Engineering
Kansas State University
2124 Fiedler Hall
Manhattan, KS 66506
mustak@ksu.edu

Cliff Hobson
Kansas Department of Transportation
2300 Van Buren
Topeka, KS 66611
cliff@ksdot.org

ABSTRACT

The Hamburg wheel-tracking device (HWTD) has gained popularity for testing rutting and stripping potential of asphalt pavements. The life of Superpave pavements with out-of-specification density and air-void mixtures can be estimated based on the HWTD test results. However, a typical HWTD test takes about 6 to 6.5 hours to complete. This study focused on reducing the test duration by developing accelerated-mix testing models based on statistics. Six fine-graded Superpave mixtures with 12.5 mm nominal maximum aggregate size (NMAS) were selected for this study with air voids at N_{design} of 4%, simulated in-place density of 93%, two test temperatures (50°C and 60°C), and three load (158, 168, and 178 lb.) levels. Six-inch field cores from three projects were also tested in HWTD at two temperatures (50°C and 60°C) and three load (158, 168, and 178 lb.) levels for model evaluation. The average number of wheel passes to 20 mm rut depth, creep slope, stripping slope, and stripping inflection point derived from the HWTD test results were used in the statistical analysis to build accelerated-mix testing models. The results show that good consistency between the predicted and the observed test results was obtained when higher temperature and load levels are used. The test duration of HWTD can thus be reduced to two hours or less using accelerated testing (statistical) models. It is expected the use of HWTD will tremendously increase and it would be more effective for the quality control and quality assurance (QC/QA) of Superpave mixtures.

Key words: asphalt pavements—Hamburg wheel-tracking device (HWTD)—hot mix asphalt (HMA)—life of defective moisture damage—rutting

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INTRODUCTION

The Kansas Department of Transportation (KDOT) is increasingly using Superpave mixtures that may be susceptible to moisture damage. The moisture susceptibility is currently evaluated by the Kansas Standard Test Method KT-56, which closely follows American Association of State Highway and Transportation Officials (AASHTO) test method AASHTO T 283, "Resistance of Compacted Bituminous Mixture to Moisture-Induced Damage." KDOT's current specified sampling- and testing-frequency chart for the bituminous construction items for the quality control and quality assurance (QC/QA) project requires that one KT-56 test be performed by the contractor on the first lot, and then one test per week or 10,000 tons (Mg). KDOT's specifications also require that the bituminous mixture shall have a minimum tensile strength ratio (TSR) of 80%. Since this test is time consuming, it often happens that the contractor might have paved a substantial area of the pavement that might have the mixture that does not satisfy this criterion. As of now, there is no "rapid" method available to find out mixtures that are susceptible to moisture damage.

The Hamburg wheel-tracking device (HWTD) was introduced into the United States in the early 1990s by the pavement engineers and officials after a European Asphalt Study Tour for technology transfer (Asphalt Tour 1991; Yildrim and Kennedy 2001). HWTD is now gaining popularity for testing rutting and stripping potential of asphalt pavements. However, a single HWTD test takes about 6 to 6.5 hours. If the test duration can be reduced to two hours or less, the use of HWTD will tremendously increase and it would be more effective for the QC/QA of Superpave mixtures.

OBJECTIVE

The objective of this project was to develop accelerated-mix testing models using the HWTD. It was assumed that two predominant distresses that would occur due to non-conforming mixtures are stripping and rutting.

ACCELERATED TEST MODELING

Overstress testing consists of running a product at higher than normal levels of some accelerating stress(es) to shorten product life or to degrade product performance faster. Typical accelerating stresses on asphalt pavement are temperature, mechanical loads, and traffic loads. Accelerated-degradation testing involves overstress testing. Instead of life, product performance is observed as it degrades over time. A model for performance degradation is fitted to such performance data and used to extrapolate performance and time of the failure. Thus, the failure and the life can be predicted before any specimens fails (Nelson 1990).

Statistical Cubic Model

In an attempt to better capture the curvature of the degradation curves of log(rut) versus log(loadings) in the HWTD tests, third-degree polynomials (cubic models) to the degradation paths can be fitted.

The following form of equation denotes the cubic model, where *Y* denotes the log-transformed rut-depth values and *L* represents the log-transformed number of loadings, as shown in equation (1):

$$Y = \alpha + \beta_1 L \underset{\cdot}{+} \beta_2 L^2 \underset{\cdot}{+} \beta_3 L^3 + e_{ij}. \tag{1}$$

The parameters α, β_1, β_2, and β_3 are the coefficients for the intercept, linear, quadratic, and cubic terms, respectively. The cubic model was used in this study because of its superior residual behavior. Model choice and comparison can be done using the Akaike information criterion and the Bayesian (Schwarz) information criterion. Both criteria utilize the log likelihood of the data, yet punish for the number of parameters in accord with the parsimony principle. Smaller values of these criteria indicate better models.

EXPERIMENTAL DESIGN

Six fine-graded Superpave mixtures with 12.5 mm nominal maximum aggregate size (NMAS) have been selected for this study. Four mixtures were sampled from four different projects, each located in one KDOT administrative district and done by one contractor. Two mixtures with modified binders were selected from the pavements of the accelerated-pavement testing (APT) program at the Civil Infrastructure Systems Laboratory (CISL) at Kansas State University. Replicate test specimens were prepared at target air voids of 4% at N_{design} gyrations. Simulated in-place density of 93% of the theoretical maximum specific gravity (G_{mm}) was taken (i.e., in-place air voids of 7%) for the Superpave gyratory compactor-compacted samples. Samples were tested at two temperature levels (50°C and 60°C) and three load levels (158, 168, and 178 lb.). Thus, the experiment involved a total of 36 sets (6 projects x 2 temperature levels x 3 load levels) of samples. Table 1 shows the characteristics of the mixtures under this study. The binder grade for four mixtures was PG 64-22, one mixture had PG 64-28, and one mixture had PG 70-22. The asphalt contents of the base design mixtures (4% air voids at N_{design}) varied from 4.9% to 5.4%. All properties satisfied Superpave and current required KDOT criteria. Figure 1 shows the aggregate gradations of the mixes used in this study. It is observed that only one mixture (US-24, District III) had a much finer gradation compared to others.

HAMBURG WHEEL-TRACKING DEVICE (HWTD) TESTING

Test Specimen Preparation

Test specimens were compacted with a Superpave gyratory compactor. The target air voids at N_{design} of 4% and the simulated in-place density of 93% were selected.

Table 1. Properties of the Superpave mixes

Route	KDOT District	Design ESALs (millions)	N_{design}	PG Binder Grade	Asphalt Content (%)	Air Voids (%) at N_{des}	VMA (%)	VFA (%)	Dust-Binder Ratio	%Gmm at Nini	% Gmm at Nmax
K-4	I	0.40	75	PG 64-22	4.90	4.36	13.9	68	0.7	88.8	96.6
US-24	III	0.70	75	PG 64-22	5.00	3.62	14.1	74	0.9	90.4	97.1
US-50	V	4.50	100	PG 64-22	5.40	4.10	14.6	70	0.6	88.4	96.9
US-83	VI	2.20	75	PG 64-22	4.90	4.38	13.9	68	1.1	89.7	96.4
CISL-A	I	2.9	75	PG 64-28	4.90	4.36	14.0	69	0.7	88.8	96.6
CISL-B	MO	NA	100	PG 70-22	5.40	4.00	14.5	73	1.1	NA	NA

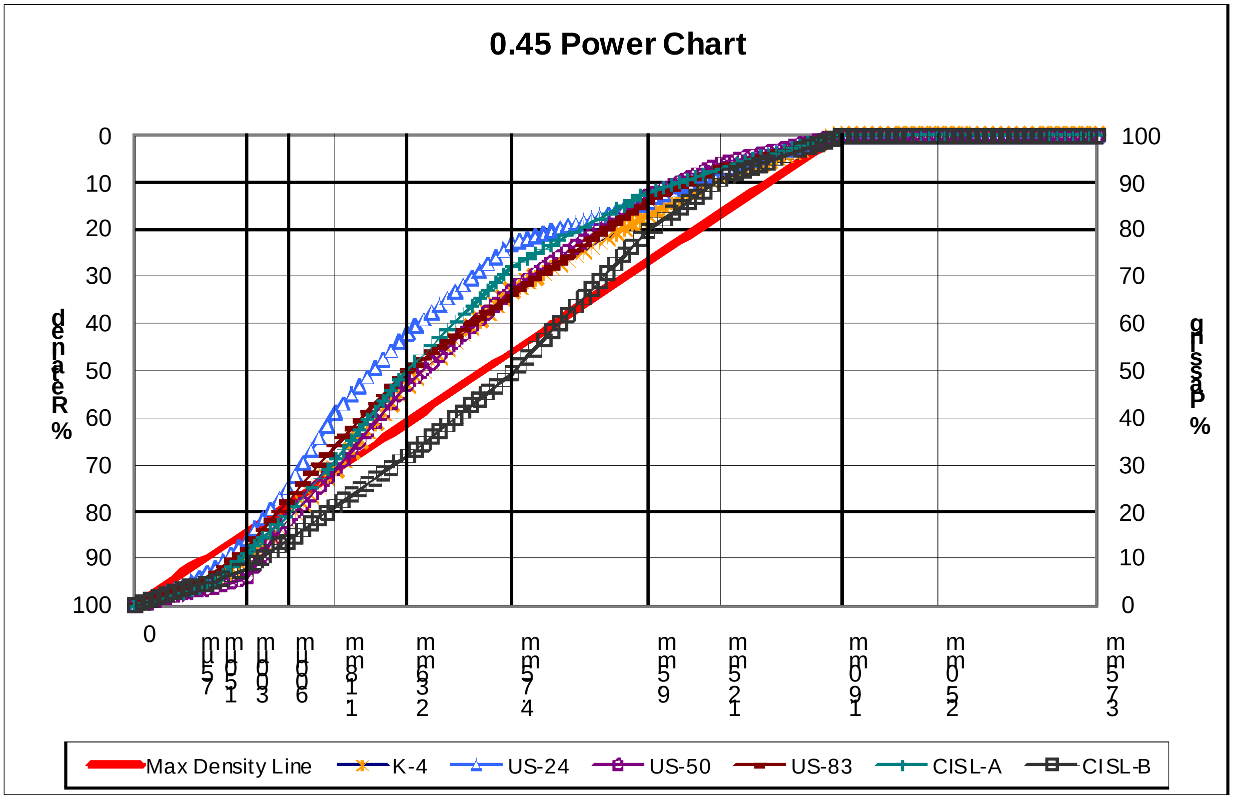


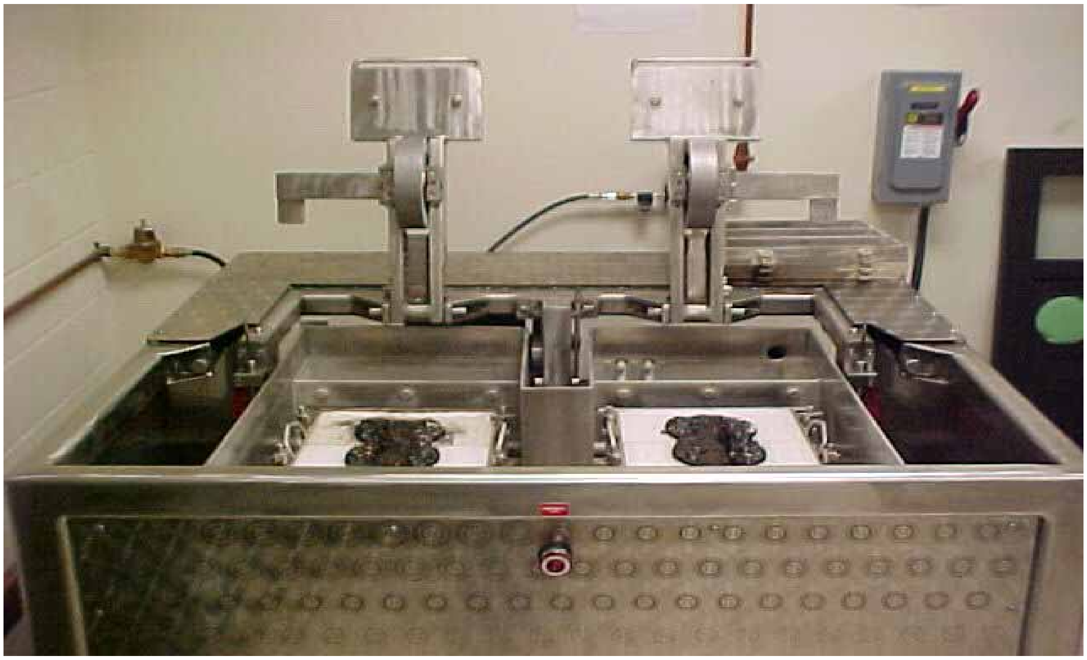
Figure 1. Aggregate gradation charts for the mixtures

Six sets of samples were prepared at 7 ±1% air voids at two temperature levels of 50°C and 60°C and three load levels of 158, 168, and 178 lb. For each mix, the replicate specimens for the HWTD were compacted at the same target air-void content (or density). Theoretical maximum specific gravity (G_{mm}) of the loose mixtures and bulk specific gravity (G_{mb}) of the compacted specimens were also determined. KDOT standard test methods KT-39 (AASHTO T 209) and KT-15 (AASHTO T 166) Procedure III were used to determine G_{mm} and G_{mb}, respectively. The air voids in the compacted specimen were calculated using equation (2):

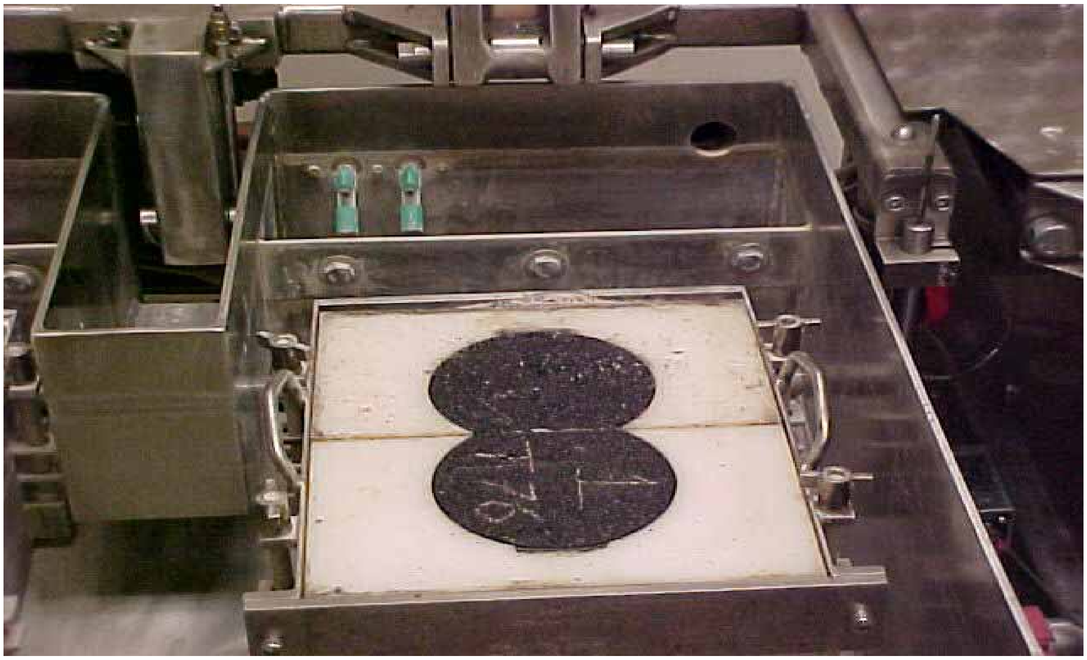
$$\% AirVoids = \frac{100 \times (G_{mm} - G_{mb})}{G_{mm}} \tag{2}$$

Test Equipment

HWTD can be used to predict both the rutting and the stripping potential of asphalt mixes and has recently been used in a number of studies (Mohammad et al. 2007; Hrdlicka and Tandon 2007; Aguiar-Moya et al. 2007; Brown et al. 2002). HWTD used in this study has been manufactured by PMW, Inc. and is capable of testing a pair of samples simultaneously. Figure 2(a) shows the Hamburg wheel tester at Kansas State University. The sample tested was 150 mm diameter and 62 mm tall plugs fabricated by the Superpave gyratory compactors and placed together in special molds as shown in Figure 2(b). In this study, Texas test method TX-242 was followed (Izzo and Tahmoressi 1999). Samples were submerged in water at 50°C or 60°C. The wheel of the tester is made of steel and is 47 mm wide. The wheel applied a load of 705 N and made 52 passes per minute. Each sample was loaded for 20,000 passes or until 20 mm vertical deformation (rut depth) occurred at any point on the sample. Around 6 to 6.5 hours were required for a test for a maximum of 20,000 passes. Rut depth or deformation was measured at 11 different points along the length of each sample with a linear variable differential transformer (LVDT).



(a) Hamburg wheel tester



(b) Test samples

Figure 2. Hamburg wheel tester

Various results that can be interpreted from the Hamburg wheel tester are the number of passes to 20 mm rut depth, creep slope, stripping slope, and the stripping inflection point, as depicted in Figure 3 (Aschenbrener 1995).

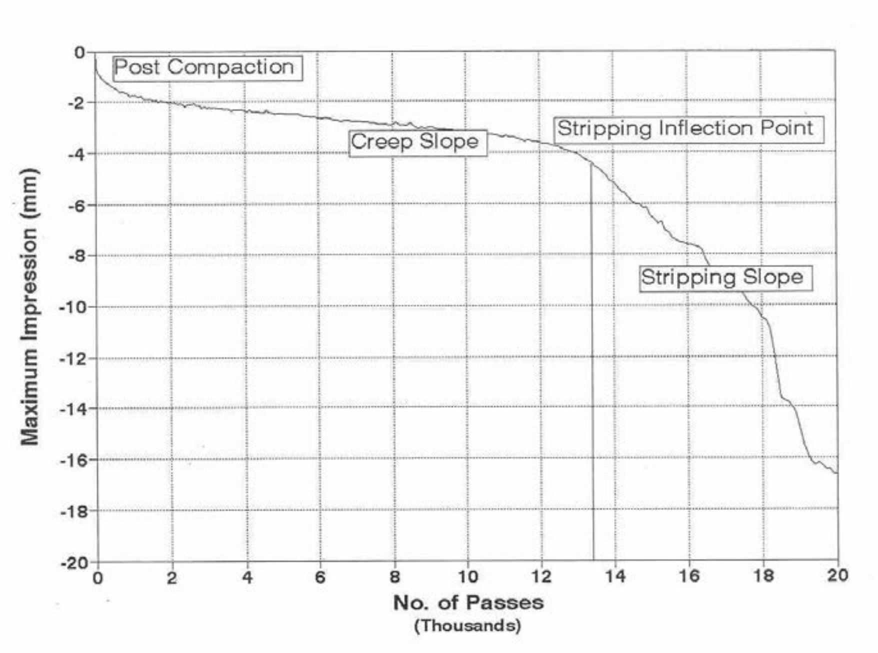


Figure 3. Interpretations of results from the Hamburg wheel tester (Yildirim and Kennedy 2001)

Results

Table 2 shows the Hamburg wheel-tester results in terms of the average number of passes. The best performing mixture in terms of number of passes to reach 20 mm rut depth is the CISL PG 70-22 mixture.

Table 2. Summary of Hamburg wheel-test results (average number of wheel passes)

Route	District	Temp. (^{0}C)	Wheel Passes at Load (158 lb.)	Wheel Passes at Load (168 lb.)	Wheel Passes at Load (178 lb.)
K-4	I	50	13,700	18,730	15,950
		60	7,230	4,075	3,995
US-24	III	50	17,625	17,390	16,650
		60	3,535	2,565	3,180
US-50	V	50	20,000	20,000	20,000
		60	5,355	9,150	4,295
US-83	VI	50	20,000	20,000	20,000
		60	7,145	3,970	7,025
CISL-A	I	50	20,000	18,070	15,055
		60	5,390	4,020	4,625
CISL-B	MO	50	20,000	20,000	20,000
		60	20,000	20,000	20,000

This mixture did not reach the failure condition (as indicated by 20,000 passes on the table) under both temperature and load levels. The effect of modified binder on better performance is quite evident. Values on the table are the number of wheel passes to 20 mm rut depth with failure criteria of 20 mm maximum rut depth or 20,000 passes, whichever comes first.

Table 3 shows the other output parameters (creep slope, stripping slope, and stripping inflection point) of the mixtures under test. The trends in these parameters closely follow the trends shown by the total number of passes to 20 mm rut depth. From the results, it is evident that the number of wheel passes in the HWTD test decreases rapidly as the temperature increases from 50°C to 60°C, but the decrease in the number of wheel passes is not significant for the higher load level at the same temperature.

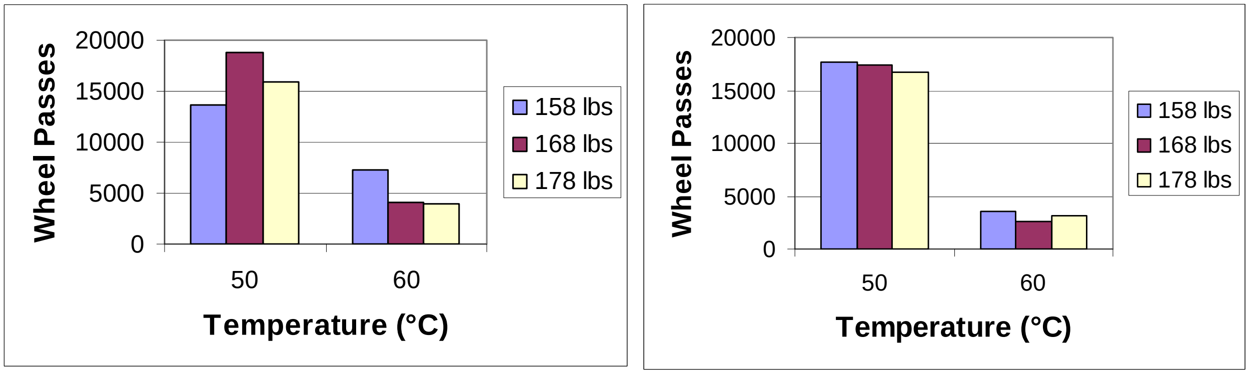
Table 3. Summary of Hamburg wheel-test results (output parameters)

Route	**Parameter**	**Load Level at**					
		158 lbs		**168 lbs**		**178 lbs**	
		Temperature Level at		**Temperature Level at**		**Temperature Level at**	
		50 (^{0}C)	**60 (^{0}C)**	**50 (^{0}C)**	**60 (^{0}C)**	**50 (^{0}C)**	**60 (^{0}C)**
K-4	Creep Slope	2,040	1,370	3,508	830	3,304	700
	Stripping Inflection Point	7,050	3,800	8,950	2,150	9,350	1,900
	Stripping Slope	460	613	613	123	440	165
US-24	Creep Slope	3,615	492	3,012	152	3,266	432
	Stripping Inflection Point	12,650	1,300	12,850	960	9,550	1,700
	Stripping Slope	365	135	203	105	430	99
US-50	Creep Slope	11,000	1,305	7,281	1,742	13,590	717
	Stripping Inflection Point	14,600	2,900	15,650	3,000	13,850	1,450
	Stripping Slope	5,778	154	870	300	405	185
US-83	Creep Slope	2,563	284	1,516	158	3,084	316
	Stripping Inflection Point	13,950	1,050	14,900	1,150	NA	1,000
	Stripping Slope	2,017	401	1,000	123	NA	419
CISL - A	Creep Slope	4,000	475	3,093	530	7,458	764
	Stripping Inflection Point	13,400	1,400	12,300	1,550	10,700	1,800
	Stripping Slope	560	200	508	172	271	168
CISL - B	Creep Slope	19,725	10,211	21,000	5,935	25,358	22,750
	Stripping Inflection Point	NA	NA	NA	17,750	NA	NA
	Stripping Slope	NA	NA	NA	486	NA	NA

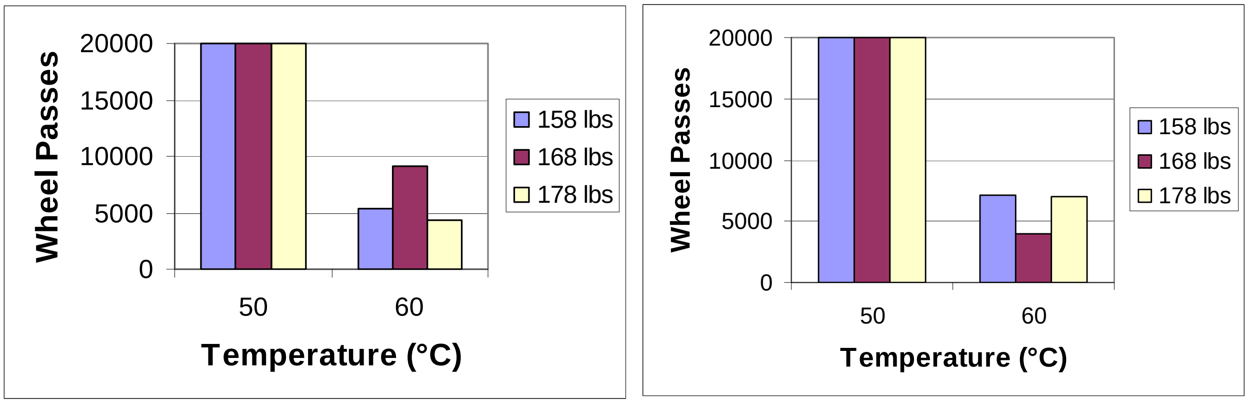
STATISTICAL ANALYSIS

Influence of Temperature and Load

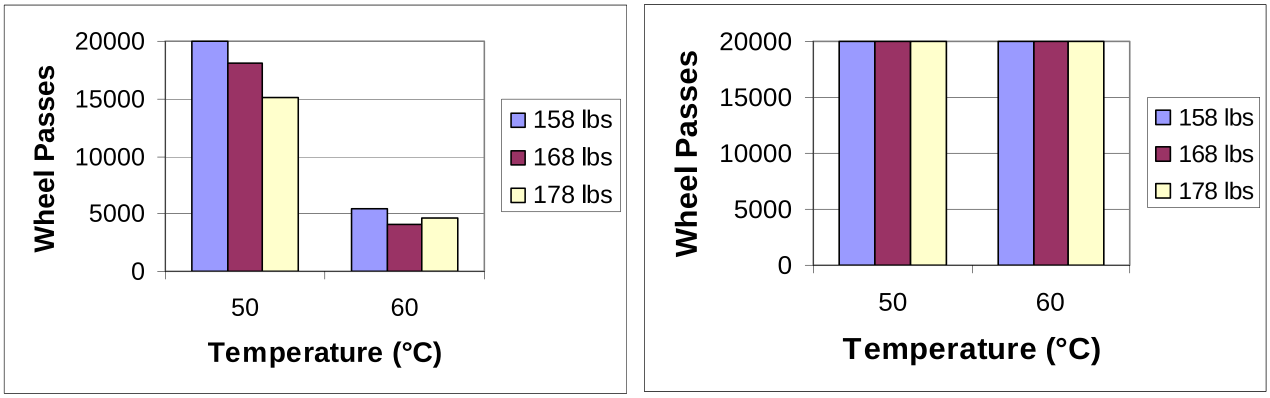
The effect of temperature and load levels on the HWTD test results was studied using the analysis of variance (ANOVA) technique and the software analysis software (SAS 1982). LIFEREG procedure was performed for the development of accelerated-testing model using HWDT test data to test the effect of different factors on the dependent (response) variable. Figure 4 illustrates the comparison of the number of wheel passes to reach 20 mm rut depth at different temperatures and load levels.



(a) K-4 route project **(b) US-24 route project**



(c) US-50 route project **(d) US-83 route project**



(e) CISL-A project **(f) CISL-B project**

Figure 4. Hamburg test results

PROC LIFEREG procedure fits parametric accelerated failure time models to the survival data that may be left, right, or interval censored *(SAS Online 2008)*. The LIFEREG procedure estimates the parameters by the maximum likelihood method using a Newton-Raphson algorithm. Figure 5 shows that the trend of rutting in the HWTD test of the US-24 project at various temperature and load levels. The steeper slope of the plot at 60°C indicates that temperature change has more significant effect when compared to the load change.

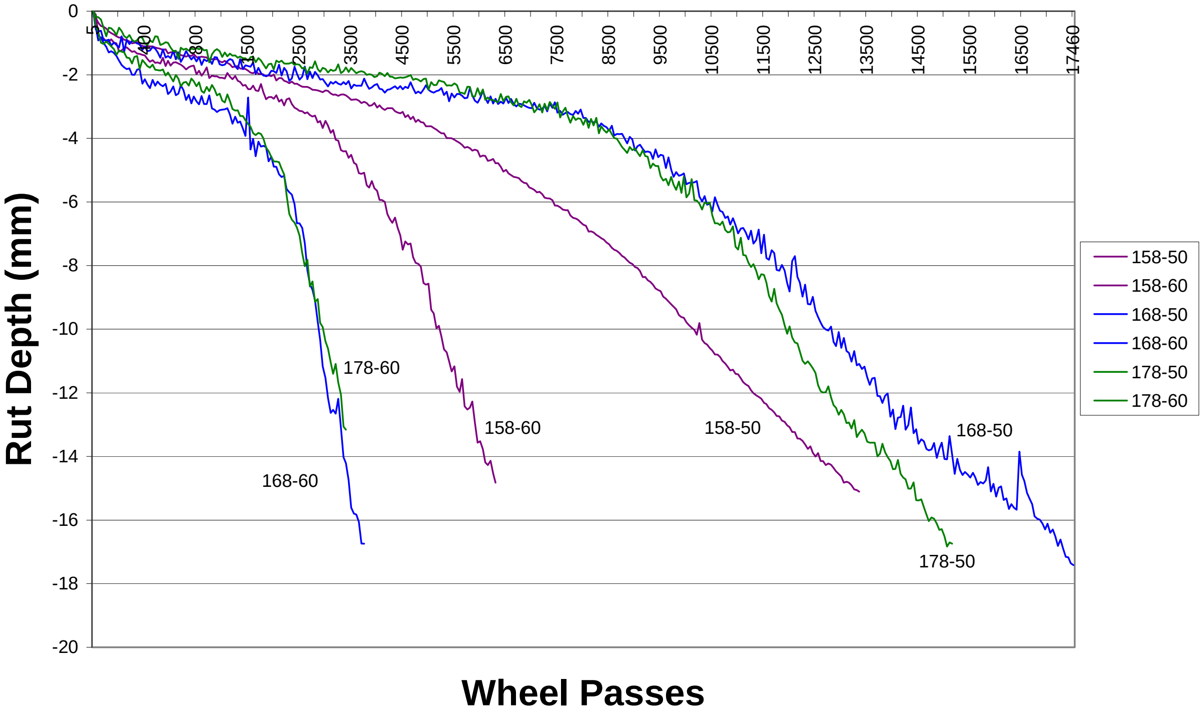


Figure 5. HWTD test results for K-4 project

The procedure also estimates the standard errors of the parameter estimates from the inverse of the observed information matrix (SAS Online 2008). In this research, four response variables were studied: (a) number of wheel passes to reach a 20 mm maximum rut depth, (b) creep slope, (c) stripping slope, and (d) stripping inflection point. The ANOVA model is shown in equation (3):

$$\text{Log}_e(\text{Response Variable})_{ijk} = \mu + \text{Temperature}_i + \text{Load}_j + \varepsilon_{ijk}\,, \qquad (3)$$

where $(Response\ Variable)_{ijk}$ = the various response variables studied, μ = overall mean, $Temperature_i$ = ith temperature effect, $Load_j$ = jth load effect, and $\varepsilon_{ij\,k}$ = error term.

The analysis was done by combining data for all projects. Table 4 shows the statistical analysis results. Temperature level has more significant effects compared to load level. Table 5 shows the comparison of the model predicted and observed wheel passes in the HWTD tests for US-50. An exponential accelerated-life model has been fitted to the data. The results show that good prediction can be made at higher load and temperature using models from the test results. The results show that data for only 5,000 repetitions are good enough to fit an exponential accelerated-life model. This would translate into approximately two hours of testing time in the HWTD test.

Table 4. Summary of statistical analysis

Parameter	Estimate	p-value	Significant
Wheel passes			
Intercept	22.8874	< 0.0001	*
Temperature	-0.1315	< 0.0001	*
Load	-0.0350	< 0.0001	*
Creep slope			
Intercept	41,290.7	0.0078	*
Temperature	-405.6	0.0227	*
Load	-85.2	0.1986	
Stripping inflection point			
Intercept	70,611.0	< 0.0001	*
Temperature	-826.3	< 0.0001	*
Load	-106.8	0.0067	*
Stripping slope			
Intercept	7,359.7	0.0019	*
Temperature	-60.6	0.0261	*
Load	-20.1	0.0437	*

Note: * Significant at 5% level of significance

FIELD VERIFICATION

Field-coring sites were selected at three KDOT district projects (District I, III, and VI). Table 6 presents HWTD results of model-predicted versus field cores. In most of the cases, the number of wheel passes at temperature level of 60°C is less or equal to 5,000, which will translate to the test duration of two hours or less. Figure 6 shows the comparison of laboratory-prepared samples and field cores for various load and temperature levels. In the histogram, it is clearly observed that the number of wheel passes at temperature level of 60°C is less or equal to 5,000 passes.

Table 5. Comparison of model predicted versus observed wheel passes

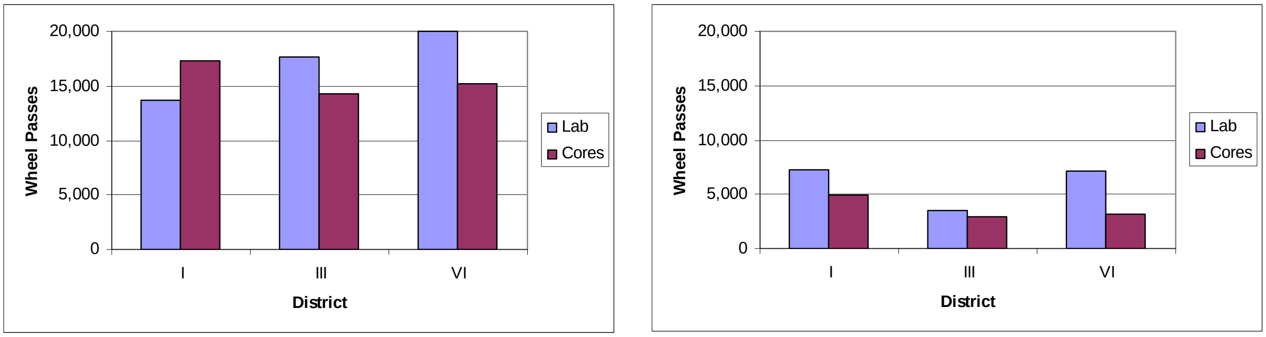
Route	Parameter	Description	Estimate	p-value
US-50	Intercept	Intercept	22.8874	< 0.0001
	T	Temperature	-0.1315	< 0.0001
	L	Load	-0.0350	< 0.0001

Wheel Passes = exp (22.8874 – 0.1315 Temp – 0.0350 load)

Temp. (^{0}C)	Load (lb.)	Calculated Wheel Passes using model	Observed Wheel Passes in Laboratory
50	158	44,166	20,000
50	168	33,942	20,000
50	178	23,918	20,000
60	158	12,931	5,355
60	168	9,112	9,150
60	178	6,421	4,295

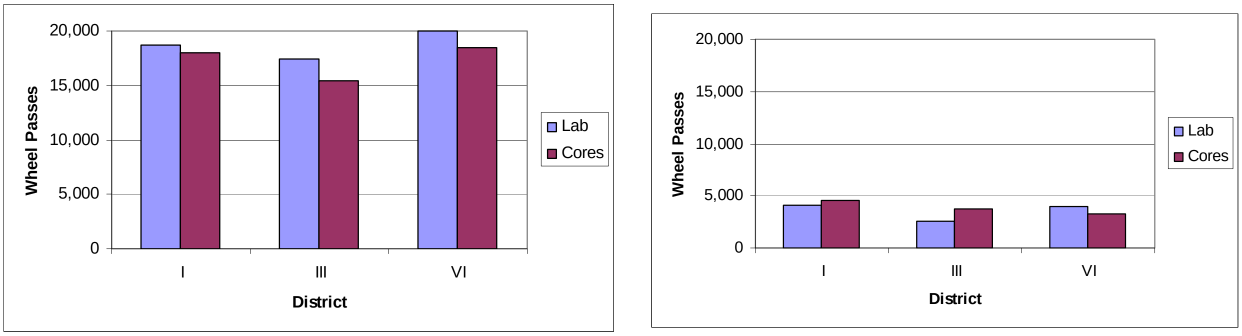
Table 6. Comparison of HWTD results of model predicted versus field cores

Temperature (^{0}C)	Load (lb.)	Model Calculated Wheel Passes	Average Number of Wheel Passes of Field Cores		
			K-4	K-258	US-83
50	158	48,166	17,290	14,270	15,210
50	168	33,942	18,015	15,420	18,525
50	178	23,918	15,130	10,745	15,065
60	158	12,931	4,855	2,870	3,145
60	168	9,112	4,585	3,685	3,310
60	178	6,421	3,655	2,445	2,715



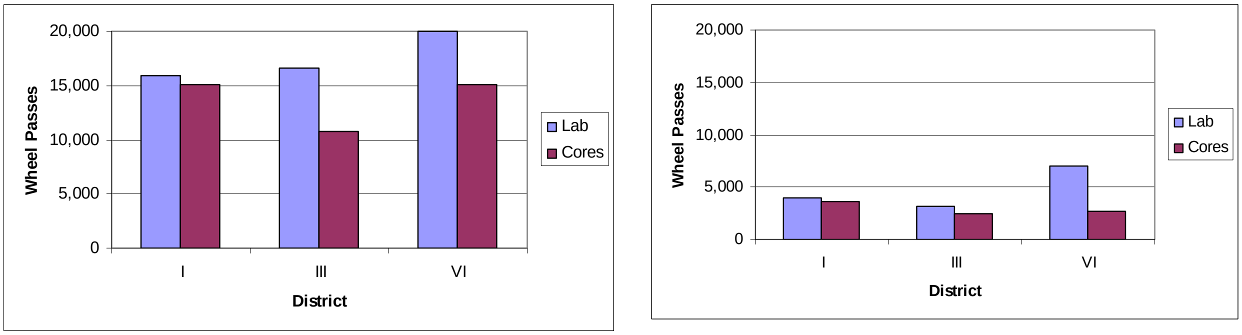
(a) 158 lb. & 50°C

(b) 158 lb. & 60°C



(c) 168 lb. & 50°C

(d) 168 lb. & 60°C



(f) 178 lb. & 60°C

(e) 178 lb. & 50°C

Figure 6. Comparison of HWTD test results of laboratory samples versus field cores

CONCLUSIONS

Based on this study the following conclusions can be made:

1. The accelerated-mix testing models developed shows consistency between the predicted and the observed test results when higher temperature and load levels are used.
2. The test duration can be reduced to two hours or less using accelerated testing (statistical) models developed in this study.

ACKNOWLEDGEMENTS

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Connecting Self-similarity in Channel Network Topology to Scaling of Flood Data

Ricardo Mantilla
IIHR-Hydroscience & Engineering
University of Iowa
423-2 C. Maxwell Stanley Hydraulics Laboratory
Iowa City, IA 52242-1585
ricardo-mantilla@uiowa.edu

ABSTRACT

Recent studies have revealed the existence of power laws, or scaling, in the magnitude of peak flows for individual rainfall-runoff events with respect to drainage area. These findings offer a new theoretical framework to understand the physical basis of power laws in annual flood quantiles with respect to basin areas that arise in flood frequency regional statistical analyses. Research in the last decade has led to the hypothesis that scale invariance, via self-similarity in the topology of river networks, is a fundamental ingredient to explain the existence of power-law behavior in flood data. Insights from these theoretical developments are used to explore the physical mechanisms that led to very extreme flooding in eastern Iowa in June 2008 and to solve the hydrological puzzle of "why were these floods not preceded by extreme rainfall?" In addition, several results that constitute the building blocks for a geophysical theory of floods will be presented. In particular, it is shown that the assumptions of the random self-similar networks (RSNs) model are valid on 28 basins across the United States and observational evidence for a scale-invariant formulation of runoff-transport in river networks.

Key words: floods—river networks—self-similar networks

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An Overview of Passenger Transportation in Iowa

Amanda Martin
Office of Systems Planning
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
amanda.martin@dot.iowa.gov

ABSTRACT

This presentation will provide an overview of current passenger rail service as well as the vision for passenger rail in Iowa. There will be a discussion about the feasibility studies that have been performed by Amtrak for future passenger rail service. There will also be a discussion on the various funding opportunities that are available for intercity passenger rail and high-speed passenger rail service.

Key words: Amtrak—feasibility studies—passenger rail service

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An Owner's Perspective on Implementing an Accelerated Bridge Construction Program

James C. McMinimee
Utah Department of Transportation
4501 South 2700 West
Mail Stop 141200
Salt Lake City, UT 84114-1200
jmcminimee@utah.gov

ABSTRACT

The Utah Department of Transportation has successfully implemented prefabricated bridge systems and accelerated bridge construction on 80 bridges utilizing different elements and technologies from full-depth prefabricated precast concrete deck panels, prefabricated precast concrete bent caps, total superstructure systems, self-propelled modular transport practice, and temporary bridge use. This presentation discusses the organizational lessons learned from implementing an Accelerated Bridge Construction Program.

Key words: accelerated bridge construction—precast—prefabricated bridge systems

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ACCELERATED BRIDGE CONSTRUCTION PROGRAM

The Utah Department of Transportation (UDOT) first implemented prefabricated bridge systems (PBS) and accelerated bridge construction (ABC) in 2002. Since then, Utah has completed 80 bridges utilizing different elements and technologies of PBS and ABC that cover the range of full-depth prefabricated precast concrete deck panels, prefabricated precast concrete bent caps, total superstructure systems, self propelled modular transport (SPMT) practice, and temporary bridge use.

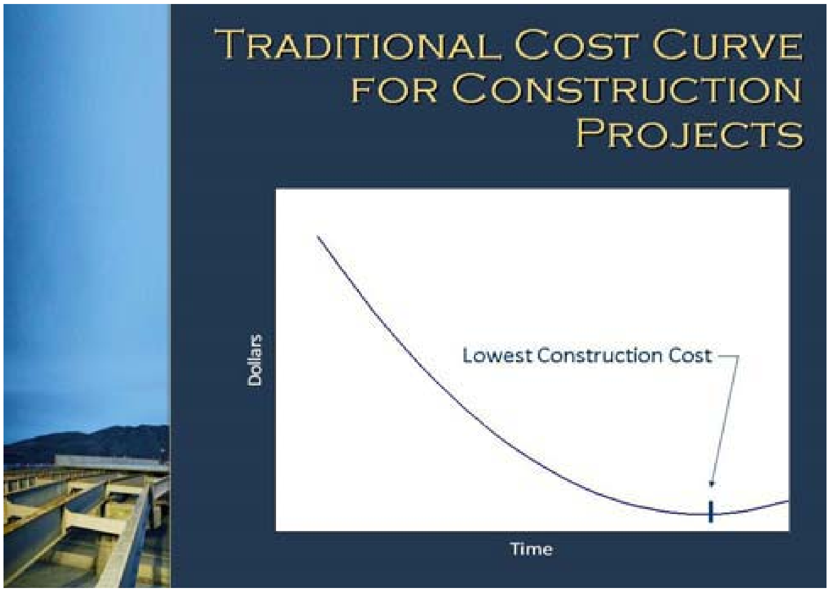
The goal of all of these innovations is the same—to find a way to replace or fabricate a bridge in an extremely short time. The purpose of these innovations is to shorten the time that bridges and construction projects are closed to the traveling public. The promise of ABC is that it creates construction closure times that are shorter than with traditional construction methods. The pinnacle of ABC methods involves using SPMTs and removing and replacing old or damaged structures in a matter of hours. This is a significant time savings compared to weeks, or even months, of closure called for by traditional methods.

The nature of construction has evolved. For the past 70 years, construction has involved building the Interstate and highway system where infrastructure did not exist. Construction included building roadways and bridges with new alignments and new sections in which limited impacts to the surrounding area and traveling public existed. Today, very little new or greenfield Interstate and roadway systems are being constructed. Projects involve roadway and bridge rehabilitations and the widening of existing systems. Unlike yesteryear, today's projects impact the surrounding areas and traveling public tremendously.

With the new environment of construction projects, conventional construction techniques have reached their limit for decreasing user delays. The need to accelerate construction activities is imminent and the use of ABC meets this need. In most urban reconstruction projects, bridge construction is on the critical path. Using ABC methods provides the ability to remove bridge construction from the critical path in projects.

Traditionally, construction contracts in our industry are awarded to low bid. This business model worked extremely well when new roadway systems were being constructed with limited impacts to the surrounding areas and traveling public. This business model was used to build the U.S. Interstate system and thousands of miles of the secondary system. It was an extremely powerful economic tool and saved the country millions, if not billions, of dollars. The idea is a simple one—allow a contractor to elect a time and a method for completing a project and submit a bid. The process encourages several contractors to submit competing bids, and through the process, the lowest cost bid is selected. Ironclad contracts and specifications that the low-bid contractor signs are written, and the contractor proceeds to build the project. The result was wildly successful, and the mobility the Interstate system provides has made the United States an economic giant. This business model has served our industry and this country well.

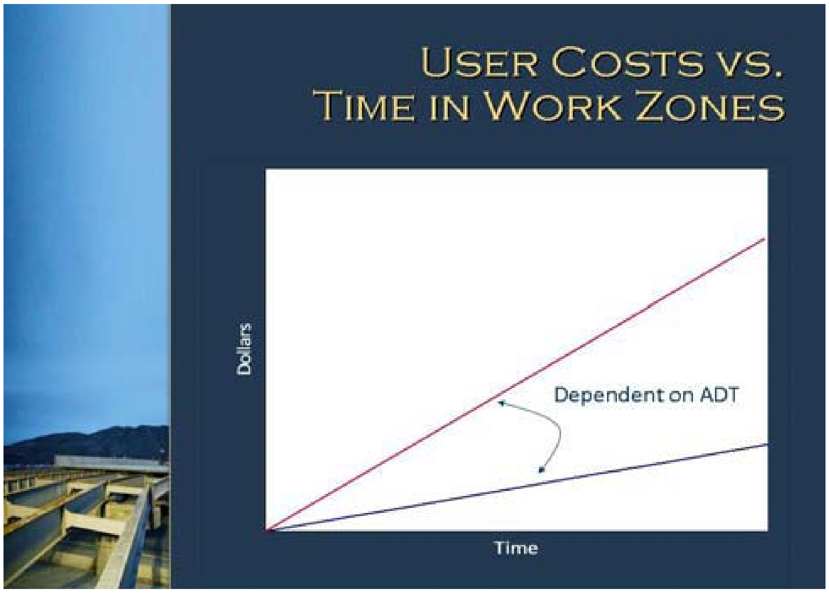
Figure 1 shows the cost of a typical construction project. As owners or contractors experience changes to the specified construction schedule due to increased resource needs or lost opportunity costs, the cost of the project usually increases. Many a battle has been had over who pays for these increased costs and who is responsible. The lesson was that if the construction project schedule changes, an increased cost is incurred by one or more parties involved.



TRADITIONAL COST CURVE FOR CONSTRUCTION PROJECTS

Figure 1. Traditional cost curve for construction projects

Today there are far fewer greenfield projects than in the past. There are very few transportation projects built that do not impact current users. The luxury of allowing a contractor to choose a schedule does not exist. Today's projects have enormous impacts, especially to the traveling public. Because of these costs, today's owners are under tremendous pressures to limit project durations. Figure 2 shows that user delays in work zones, which are a business cost to the public, increase linearly. These user delays also affect the economy of the area around the work zone—sometimes profoundly. As user costs mount, the pressures to decrease project durations increase. The steepness of the graph and the increased pressures on construction schedules depend on the volume of user traffic. The reality of today's projects is that user cost increases are directly related to an increase in construction duration.



USER COSTS VS. TIME IN WORK ZONES

Figure 2. User costs vs. time in work zones

As owners are experiencing pressures to limit or decrease construction durations, it is time to start thinking about a different business model that considers user costs as part of project costs. The first attempts to accelerate construction have led owners to provide incentives and disincentives to contractors around project delivery time. This new business model is just an extension of the logic that it is worth money to owners and to the public to decrease project impacts. The new paradigm requires owners to look at projects differently; user costs are now included when calculating project costs. Adding the two previous graphs together produces a total project cost curve that incorporates user costs, as shown in Figure 3. The low point on the project cost curve is the lowest project cost considering society costs and a compressed timeframe.

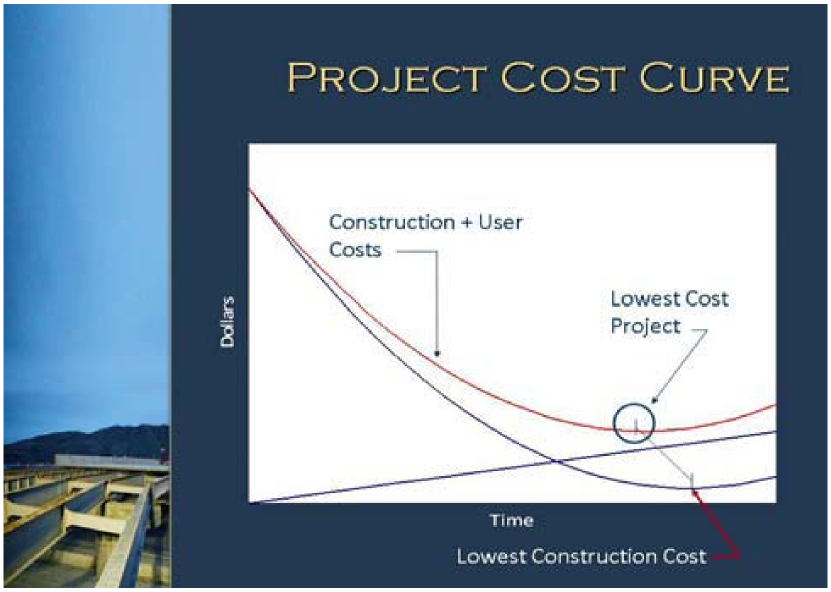


Figure 3. Project cost curve

As shown, the total cost includes more than just the lowest construction cost used in the traditional method of construction projects. As user delays decrease, positive public perception of the department skyrockets. Savvy owners are willing to pay a little more for construction costs to shorten the impacts to the traveling public and decrease construction delays. As transportation departments continue to embrace this model and deliver projects on time and within budget, they gain public praise and political capital. This new business model is the mind frame of the future.

As with the introduction of any new technology or method, it has taken time and increased project costs to initially implement these ideas. Figure 4 shows that any new method typically costs more than traditional methods at first. Over time, the costs become less as familiarity with the new method increases. Proven innovative technologies have the potential to be less costly than traditional methods in the long run.

While owners' first efforts to apply these techniques cost more than traditional methods, the time savings to road closures shows promise. For the initial projects that used PBS, UDOT paid a premium; however, UDOT was willing to absorb more upfront costs because there was the possibility of long-term tremendous economic benefit. UDOT's initial projects have shown that the time savings to users would yield positive cost-benefit ratios if applied to the correct set of projects. Subsequent PBS projects have shown decreases in unit costs for prefabricated elements as the industry has gained experience and confidence with these methods.

As more projects use innovative technologies and as experience is gained, the cost of the projects will decrease, as represented in Figure 4. In the case of PBS, the promise that manufacturing prefabricated elements can be less expensive than cast-in-place can be proven by looking at examples from our industry and other industries. Consider precast beams as an example, which have proven to be far more economical and of higher quality than cast-in-place beams for transportation projects. Other construction industries, such as high-rise parking garage structures, have been using prefabrication as a way to speed project delivery and decrease construction costs for years. As the costs of ABC gravitate towards a minimum, project costs will decrease and bridges will less likely be a critical path item in a construction schedule.

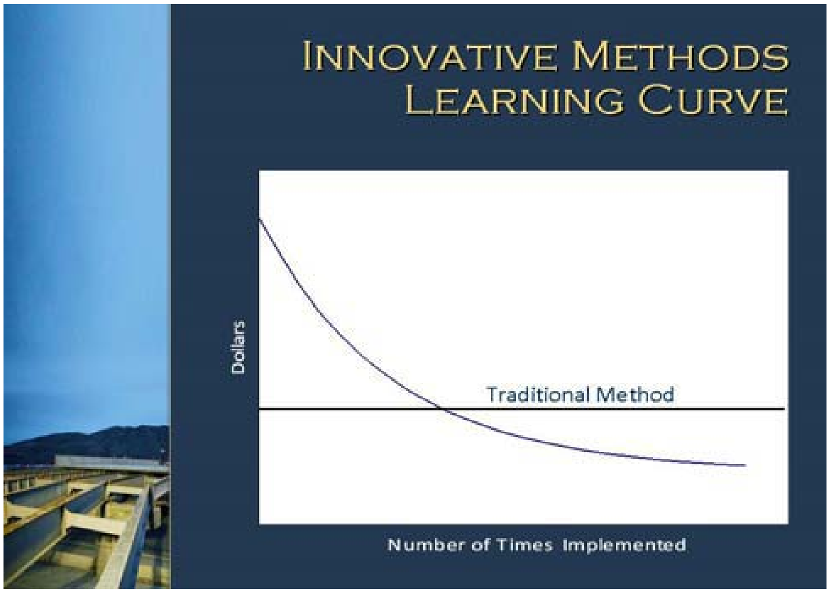


Figure 4. Innovative methods learning curve

BENEFITS AND CHALLENGES

In addition to the benefits that ABC can bring to schedule and to agency reputation, there are other benefits to implementing ABC, including increased safety and increased quality.

Safety benefits are realized in several areas of projects as owners implement ABC. Most of the benefits are associated with limiting work zone impacts to users. As user time spent in work zones decreases, the risks of accidents decrease. There are less user accidents because there are less user impacts on the projects. The second safety benefit relates to construction workers. Construction zones are generally already hazardous; the addition of constructing projects under live traffic has proven to be more deadly. By implementing ABC, the construction worker is exposed to less construction time in live traffic.

The quality ramifications of prefabrication are undeniable. The industry has known for years the effects of cure times and mix designs on the durability of concrete. Our industry has continually pushed those limits as a way to decrease construction duration and limit user delays. Additionally, site casting has many documented construction quality challenges, such as temperature changes, traffic vibrations, workmanship, and consistency issues. The use of prefabricated elements allows better cure times, more durable mix designs, and the advantages of a controlled manufacturing environment.

UDOT has faced several other challenges in the drive to implement ABC in projects. Some of the issues being addressed include

- Overcoming the reluctance of agency leadership to support ABC
- Overcoming the organizational barriers and building process to support ABC
- Gaining concurrence from the local industry—consultants, contractors, supplier, and internal staff

To implement ABC, UDOT needed the confidence and support of its senior leaders. UDOT was careful in the choice of projects and techniques to implement. UDOT leadership has been extremely supportive of finding ways to shorten project delivery times and implementing experimental projects that have been attempted and proven elsewhere. UDOT's Project Development mantra is that they look for ideas on the leading edge, not the bleeding edge. The reluctance of agency leadership to embrace ABC has diminished over time as successful projects have been completed.

Institutional hurdles also had to be overcome. UDOT middle management, the institutional process, policies, procedures, and attitudes had to be addressed. The strategies to sell experimental projects that are successful have worked with the balance of the organization. The Bridge Division has developed into champions of ABC, is working to become experts in its applications, and has adopted the idea that all projects are considered for implementing ABC methods first and lapses to traditional methods only if the project cannot benefit from decreased schedules.

UDOT also faces the challenge of leading industry, contractors, and consultants to ABC. All of the projects that UDOT produces are a collaboration of UDOT with the business industry. The industry needs to understand what ABC can bring to project delivery. This group also needs to understand the business reasons and the business opportunities represented by changing from traditional methods. The strategies represented by demonstration projects are great, but the effort to excite business that was most successful was to invite the industry on scanning tours. UDOT hosted the local industry on several tours to see innovative ABC projects in New York, New Jersey, Louisiana, and Florida. These opportunities allowed business to see, touch and feel projects, and more importantly, to talk to business people who had already implemented ABC. These were people who could provide information and lessons learned. UDOT will continue to educate, nurture understanding, and support ABC with our home state transportation industry. UDOT cannot implement ABC without the buy-in and support of contractors and consultants.

To address the issues being faced, UDOT took an active role in seeing the ABC initiative advanced. UDOT is a small department of transportation from a small state but has a national reputation for innovation and a willingness to try new things. The journey to embrace ABC began many years ago as Utah began experiencing the shift from building new roads on new alignments to rehabilitating and improving existing roads. UDOT began dabbling in A+B contracting and Incentive/Disincentive contracting as a way to decrease user delays and user costs. UDOT began by placing time constraints for project completion on projects to limit impacts to the surrounding areas and to the traveling public. UDOT initiated the use of A+B contracting in which the contractor specifies a project duration and cost. Incentives and disincentives were added to contracts. As the project schedules decreased, the quality of construction using traditional methods became a concern.

In the 90s, UDOT had another reason to accelerate project delivery as Utah prepared for the 2002 Olympics and reconstructed 17 miles of I-15 in Salt Lake City. The $1.6 billion dollar design-build contract brought to bear another tool to accelerate project delivery. This contract was the largest in the United States at that time and was the first time UDOT had used this innovative contracting method. One of the innovations in the project was ABC. The contractor chose the use of half-depth precast concrete deck panels as a way to shorten construction time, increase quality, and lesson the strain on the labor pool for skilled labor. The project was completed on time and under budget, and the innovation led UDOT to begin trying a series of PBS and ABC construction techniques. The most recent ABC project used SPMTs and replaced a bridge over I-215 in Salt Lake City during a weekend closure.

LESSONS LEARNED AND BEST PRACTICES FROM IMPLEMENTATION

- OBTAIN FUNDING FOR DEMONSTRATION PROJECTS—Innovative Bridge Research Development (IBRD) and Highways for Life (HfL) funding provided significant funding for UDOT to jumpstart demonstration projects.
- EDUCATE AND COMMUNICATE WITH THE INDUSTRY—The idea that departments of transportation engage industry with presentations, workshops, and even simple discussions of ABC as a program and intent drives the ABC technology forward.
- PERFORM SCANNING TOURS—Several scanning tours were organized for UDOT directors, designers, managers, consultants, and contractors to gain support for this initiative. UDOT developed a flowchart for implementing ABC and met with the Utah Associated General Contractors (AGC) to gather input and to gain support for the program.
- APPLY SYNERGY OF INNOVATIVE CONTRACTING WITH ABC—With the use of ABC in projects, UDOT has found that the use of innovative contracting methods like design-build (DB) and construction management general contractor (CMGC) complement these technologies. UDOT and the Utah AGC have successfully applied innovative design and construction methods to deliver projects with very short delivery times. The collaboration between the industry and UDOT that these methods foster has advanced ABC implementation.
- USE DECISION SUPPORT TOOLS FOR ABC METHODS—UDOT has started developing a program that will apply the right ABC tool and project delivery method to specific projects. An ABC Decision Chart was created to help project managers plan projects adequately at the concept level.
- IMPLEMENT STANDARDIZATION—ABC Standard Drawing and Specification development has started for full-depth precast concrete bridge decks and the use of SPMTs. Future standards will be created for additional bridge superstructure and substructure elements. Building contractor expertise and the industry infrastructure to support the use of ABC is a priority for UDOT.
- IDENTIFY A PROGRAM OF PROJECTS—Identify a program of projects that will help industry understand the size of the business opportunity represented by ABC projects.
- GET INVOLVED NATIONALLY—The American Association of State Highway and Transportation Officials (AASHTO), Technology Implementation Group (TIG), and Transportation Research Board (TRB) are working on improving ABC. Literature, design guides, best practices manuals, and technical support are available to help states engage in ABC.

SUMMARY

ABC is a partnership between the owner, design industry, and construction industry. To transition from traditional methods of contracting requires the buy-in and support of all parties. UDOT is promoting the use of ABC at all levels. Programmatically, UDOT will use the agency and industry momentum to get over the hurdles involved with the use of ABC. UDOT is changing the way they think about how they evaluate costs and benefits of projects to justify and move forward with these new concepts. The exciting promise of ABC is that as more projects are completed and the engineering and construction industry knowledge increases, ABC will cost the same, if not less, than traditional construction methods and user delays and costs will be minimized.

The Kansas Experience with Polymer Concrete Bridge Deck Overlays

Dave Meggers
Kansas Department of Transportation
2300 Van Buren
Topeka, KS 66611
Dave.meggers@ksdot.org

ABSTRACT

The Kansas Department of Transportation (KDOT) uses multi-coat polymer concrete overlays for bridge preservation. The first multi-coat polymer concrete overlay was placed in 1999 on Bridge No. 46 in Shawnee County, Kansas, as a research project.

Four suppliers were invited to place materials on the structure. Each material was allotted 100 linear ft of deck, 333 square yards per supplier. To this point in time, KDOT has placed over 90 polymer overlays all with the intent of minimizing chloride and water intrusion to preserve the structures.

Typically, candidate structures have had minimal spalling that has been repaired, and may have cracked delaminated silica fume or high density overlays with no failure of the surface itself. There are a number of structures under heavy traffic and no failures have been found due to delaminated concrete overlays. The intent of KDOT in using the polymer overlays is to place them on structures that are not seriously deteriorated and require minimal deck repair, thus preserving structures that are still very serviceable.

Three new structures in Kansas have had polymer overlays placed on them due to construction errors which resulted in cracked bridge deck concrete or a loss of concrete cover due to settings of the placement machine. One structure has had an overlay placed when new by design.

The average savings of using a polymer overlay over a silica fume overlay as a maintenance bridge preservation action is $16 per square yard. Significant savings are also found in user costs and traffic control costs. Most of the structures that the polymer overlays have been placed on in Kansas are relatively short, 200 to 400 ft. However, two structures in Sedgwick County 12,496 and 12,110 ft long, with approximately 42,500 vehicles per day in each direction and approximately 7% trucks, were overlaid in 2007 and 2008.

The Kansas experience in general has been very good; however, it has not been without some problems and a learning curve. Additional wording has been added to the Kansas Standard Specification to provide additional attention to eliminating problematic issues such as spillage of resins, moisture, and preventing contamination after shot blasting. Familiarity with the specification is important. There are now several contractors in Kansas with significant experience placing the polymer overlays.

KDOT is planning on the continued use of the multi-coat polymer concrete overlay for preservation purposes and is intends to begin using the overlays on new bridge structures.

Key words: bridge preservation—multi-coat polymer concrete

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INITIAL APPLICATION

The Kansas Department of Transportation (KDOT) began working with Epoxy Polymer Overlays in 1999 when a polymer overlay was placed on Bridge No. 46 in Shawnee County, Kansas. The structure was a concrete box girder structure, 398.5 ft long and 30 ft wide, on the ramp connecting westbound I-470 to westbound I-70 with an annual average daily traffic (AADT) of 6620 vehicles with 30%heavy trucks. The structure was built in 1959. The deck was patched and a high-density concrete overlay was placed in 1992; this overlay had significant cracking by 1999 when the polymer overlay was placed.

Bridge 46 was curved and had a significant slope so deicer was heavily used on the structure. Therefore, the intention in placing the polymer overlay was to seal the cracks in the high density overlay to prevent additional chlorides and water from reaching the uncoated reinforcing steel. The aggressive surface that was achieved with the flint rock broadcast aggregate was an added advantage.

As the application of the polymer overlay was a research project, four suppliers were invited to place materials on the structure. As stated earlier, the structure was nearly 400 ft long. Each material was allotted 100 linear ft of deck, 333 square yards per supplier. As expected, all four of the materials placed were very similar in physical properties and all materials were placed the same day by the same contractor. The bridge deck was shot blasted as per the specification (ICRI CSP 5-7) (see Figure 1). Costs included material, placing of the material, and shot blasting. The aggregate was a flint rock from Pitcher, Oklahoma, meeting the specified gradation. This material is still used on all polymer overlays in Kansas today. The KDOT specification was and still is fashioned around the suggested specification noted in the AASHTO-ARTBA Task Force 34 document and the Virginia Polymer Overlay specification. The placement of the materials went as expected with no problems.

In June of 2000, bond failure of the overlay materials was noted on the bridge. The failures were distributed longitudinally on the bridge and appeared to be large spills or splashes. All materials were failing similarly. The research unit evaluated the failures and took samples of the concrete for chemical evaluation. The evaluation indicated the presence of fusel oil; this material is a byproduct of alcohol production and is used in bond breaker for concrete. The overlay was sounded and all loose material was identified. Saw cuts were made well outside the sounded areas to ensure all material was removed from the contaminated concrete. The contaminated concrete was removed by aggressive sand blasting and some chipping. The concrete was again tested and no contamination was found. The overlay was replaced in the same sequence as it was originally constructed. After this failure, subsequent specifications for the polymer overlay called for an ICRI SPS of 6–7.

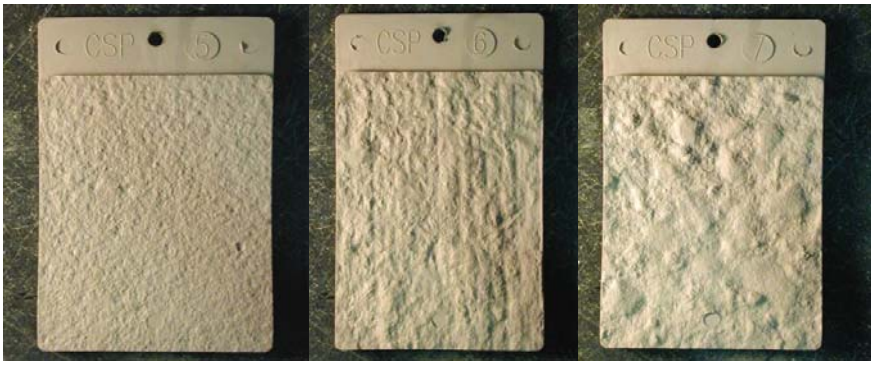

Figure 1. Concrete surface preparation samples

The overlay has been in place for 9 years and has no indication of any additional significant failures. It has had approximately 21,000,000 total vehicles of which 30% have been heavy trucks. In 2003, the skid coefficient was found to be 53 using the Pavement Surface Friction Tester (ASTM C 274) with a ribbed tire. Recently, the overlay was re-evaluated and was found to have some reflective cracking and several small areas that have spalled. This spalling may be due to some repairs performed on the structure that required cutting through the overlay and replacing it. There are plans to lightly shot blast the surface and place a single coat overlay to upgrade the surface.

PRESENT EXPERIENCE

As previously stated, KDOT placed the first polymer overlay to seal the cracked bridge deck, protect the uncoated reinforcing steel from additional chloride and water intrusion, and prevent damaging corrosion. To this point in time, KDOT has placed over 90 polymer overlays all with the intent of minimizing chloride and water intrusion to preserve the structures. Some structures have had minimal spalling that was repaired, and many have delaminated silica fume or high density overlays that have been delaminated for a number of years with no failure of the surface itself. On these structures, KDOT has chosen not to remove the overlay but simply shot blast the surface and place the polymer. There are a number of these structures under heavy traffic, and no failures have been found. If a deck is found to have shallow delaminations, the loose concrete is removed, the areas are repaired, the deck is shot blasted, and the polymer is placed. The intent of KDOT in using the polymer overlays is to place them on structures that are not seriously deteriorated and require minimal deck repair, thus preserving structures that are still very serviceable.

Three new structures in Kansas have had polymer overlays placed on them due to construction errors which resulted in cracked bridge deck concrete or a loss of concrete cover caused by errant settings of the placement machine. A polymer overlay was placed on Bridge No. 41 in Mitchell County due to concrete consistency problems encountered during placement of the deck, resulting in high permeability and low density concrete with some cracking and small voids. The bridge was determined to be structurally sound and the overlay was placed to protect the structure. On Bridge No. 24 in Sheridan County, the setting on the depth screed slipped, reducing the cover over the reinforcing steel to 2 in. rather than the standard 3 in. The overlay was placed to provide the lost protection. Both of the structures listed above received single-coat polymer overlays; however, the polymer was placed at a rate of 1.5 times that of a typical first layer of an overlay.

The third new structure to have a polymer overlay placed on it was Bridge 48 in Lyon County. A corrosion-inhibiting admixture was used in the bridge deck concrete and extensive cracking of the deck occurred. The polymer overlay was placed on the deck to seal the cracks and achieve the expected structure life. This overlay was a standard two-coat system.

Bridge 206 in Sedgwick County is the only new structure on which a polymer overlay was placed before opening to traffic by design. This structure is located on a roadway that is directly off of KDOT property and all of the salt/sand trucks used for snow and ice control cross the structure coming and going to the KDOT shop. The previous structure had an excess of 11 lbs of chlorides per cubic yard before removal. The purpose of the overlay was to give additional protection from chloride intrusion.

KDOT placed one polymer overlay in 1999, 2000, 2001, and 2002; placed five overlays in 2003; four in 2004; 14 in 2005; 19 in 2006;15 in 2007; 26 in 2008; and 12 will be let in 2009. The success of the polymer overlays in the early years of application was tracked by the bridge management unit of KDOT Bridge Design, and by district managers. The use of the overlays has grown rapidly. At this point in time, five counties in Kansas have placed a total of 14 polymer overlays with one county placing 11 overlays.

Shawnee County has just finished placing a polymer overlay on a structure that is 1296 ft long and 28 ft wide, a total of 4035 square yards. The reason for the popularity of the polymer overlay is not just the material-cost differential between a silica fume overlay and a polymer overlay, but also the significant difference in the cost of traffic control and user costs due to lane closers.

COST COMPARISON

What follows is a cost comparison summary between a typical polymer overlay and typical silica fume overlay.

The typical rehabilitative silica fume overlay involves milling of the bridge deck, and removal and replacement of deteriorated concrete, which usually costs more than expected due to the aggressive action of the milling machine and placement of the overlay. The average cost for placement of a silica fume overlay between 2001 and 2008 was $19.80 per square yard for milling, and $51.40 per square yard for placement of the silica fume overlay. This is a total cost of $71.20 per square yard. The average cost of a polymer overlay for the same period was $55.05 per square yard, a savings of $16.00 per square yard. As previously stated, significant savings are also found in user costs and traffic control costs.

Costs listed above for the polymer overlays include deck prep and placement of the Polymer Overlay. They do not include patching, if any, or traffic control, as traffic control is a very fluid cost on projects. What follows is a cost comparison of a “typical” project traffic control.

Typically an existing structure with a distressed deck would be milled, patched, and a silica fume overlay would be placed on the deck to reduce chloride intrusion and slow the corrosion process of the reinforcing steel. Most decks that would receive this “maintenance overlay” would be old enough to have black reinforcing steel rather than epoxy-coated reinforcing steel. KDOT also places silica fume overlays on high-traffic volume new structures that have epoxy-coated steel. Kansas began using epoxy coated steel in the mid 1980s.

The following is a summary of traffic control costs for a two-lane structure silica fume overlay and polymer overlay on both a two lane and four lane highway:

- Two lane highway silica fume
  $700 per day with signals, and lane closed overnight for 20 working days
- Four lane highway silica fume
  $800 per day and lane closed overnight for 20 working days
- Two lane highway polymer
  $1200 per day using live flaggers, and lane is open overnight for five working days
- Four lane highway polymer
  $335 per day, lane is open overnight for five working days

Placing of a silica fume overlay on two lanes of an average Kansas structure (350 ± feet) involves milling of the bridge deck, patching of the deck, and placing and curing of the silica fume overlay. These costs will not be addressed due to bidding procedures and variability. Placing of the polymer overlay on the same structure would involve patching, shot blasting to the proper surface texture, and placing of the overlay. The significant difference in time required to complete these two projects is mostly due to cure time required for the silica fume concrete. It would typically be seven days before traffic is allowed on the structure. The time required to place the silica fume overlay on the deck indicated above would require approximately 20 days and overnight lane drops. The time required to place a polymer overlay on the same structure would require approximately five days with no overnight lane drops.

On a two lane road the traffic control for placement of a silica fume overlay would require signals at a cost of approximately $700 per day for approximately $14,000. Applying a polymer overlay to the structure would require five days and human flagging at a cost of $1,200 per day for approximately $6,000. This indicates a savings of $8,000 for traffic control using a polymer overlay.

On a four lane road the traffic control for placement of a silica fume overlay would cost approximately $800 per day for approximately $16,000. Applying a polymer overlay to the structure would require five days and minimal flagging at a cost of $335 per day for approximately $1,675. This indicates a savings of $14,325 for traffic control using a polymer overlay and 15 days of user costs.

SPECIAL APPLICATIONS AND STRUCTURES

Most of the structures that the polymer overlays have been placed on in Kansas are relatively short (200 – 400 ft). There have been a few larger structures, but two structures in particular are worth some detailed discussion: the structures are Bridges 290 and 291 in Sedgwick County. These two structures are on I-135 with three lanes each and access ramps. The structures are basically twin viaducts 12,496 and 12,110 ft long and approximately 42,500 vehicles per day in each direction with approximately 7% trucks.

Due to the high traffic volume on these structures, the construction process was changed considerably to increase the production and minimize lane closures. The contractor was allowed to work from 7:00 p.m. until 6:00 a.m. Sunday through Thursday nights with a large penalty for late lane opening in the morning. The bridge deck was shot blasted to the required surface relief as usual, but the process was started well ahead (two weeks) of polymer placement to prevent the preparation of the deck from slowing the placing of the polymer overlay. The contractor placed one coat of polymer on an area approximately 1,500 ft long and two lanes plus the shoulder width (5,000 square yards) each night. Before the placement of the polymer, the area was quickly re-shot blasted to clean off contamination that may have occurred after the initial preparation, and one coat of polymer with aggregate was placed per night. Traffic was allowed on the first coat the next day. The following night the first coat was quickly and lightly shot blasted to remove contamination that may have occurred during the day, and the second coat of polymer with aggregate was placed, thus completing a significant section of the bridge each of the two days, but allowing the opening of the bridge to traffic each morning.

The southbound structure and approximately half of the northbound structure was completed in the summer of 2007. The remainder of the northbound structure was completed in the spring of 2008 using the same procedures and epoxy materials.

SPECIFICATION CHANGES

The Kansas experience in general has been very good as can be seen from the increase in the use of the polymer overlays. However, it has not been without some problems and a learning curve as previously stated. The experiences have prompted KDOT to change the specifications to improve the quality of the completed overlays. After the de-bonding occurred on the first polymer, the specification was changed so that the surface preparation requirement was changed to ICRI CSP 6 or 7 to ensure removal of contaminated concrete.

Additional wording has been added to the specification to provide additional attention to eliminating problematic issues such as spilling of the polymer components on the deck surface, evaluating a deck for moisture after a rain, cleaning of the surface after a rain, and preventing contamination of the prepared deck by tracking materials on the shot-blasted surface. The specification now requires that the shot blasters be emptied at least 50 ft from the end of the bridge deck or area being prepared. This focus on contamination is due to finding small delaminated areas on several structures after completion of the project, see Figure 2. These areas are small, irregular and scattered. This indicates small areas of contamination. The contamination is probably due to dust being tracked onto the prepared deck or liquid spots on the deck that were not seen before placement of the polymer.

To improve the quality of the overall project, the polymer is now placed to a minimum height of 6 in.on the surface of the curb or barrier rail to ensure sealing of the joint at the deck surface and eliminate intrusion under the overlay of water and chlorides. Also, distribution equipment must be capable of verification of the mix ratio of the components and verification of the amount of material placed on the bridge deck. Kansas also now requires that a material supplier have a three year application history in Kansas before the material will be placed on the pre-qualified materials list.

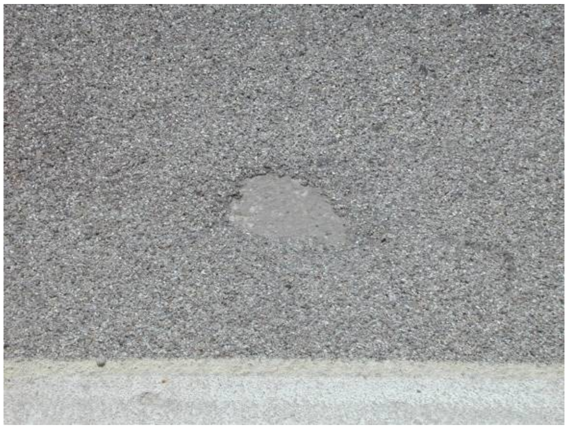

Figure 2. Small overlay spall

An additional change in procedures, but not in the specification, is that if a known experienced contractor uses material that has sufficient application history in Kansas, the overlay may be placed on the prepared deck previous to performing a test pull-off of the overlay material to expedite the project and minimize traffic delays. The pull-off test is performed after the overlay is in place and cured. All pull-off tests are performed by the contractor.

The KDOT Standard Specification has also been changed slightly to accommodate the use of the polymer overlays on new structures. Changes include concrete cure time for full-depth deck applications.

GENERAL INFORMATION

Kansas has had significant success with removing only shallow delaminations and repairing only those areas that are spalling. Delaminated concrete overlays that have not begun to break up are left in place. A number of the bridge decks that have had the polymer overlays placed on them have had high density overlays with a considerable amount of delamination. The high density overlays on these bridges were cracked but not coming apart and spalling. As the shot blasting is not an aggressive treatment, it is felt that if the overlays are withstanding the impacts of traffic without the polymer overlay, they will be reinforced by the polymer overlay. All but one of the overlays in place are epoxy, one structure has had a methacrylate base polymer overlay applied.

CONCLUSIONS

The polymer concrete bridge deck overlays have become very valuable tools for preserving bridge structures in Kansas. The KDOT has more than 90 polymer overlays in place, and Kansas Counties are beginning to see the value of the polymer overlay with additional county bridges being protected on a regular basis.

The KDOT will continue to use this preservation tool in the future, with applications for new bridge decks in an effort to reduce the use of silica fume overlays on new decks.

Infrastructure Asset Management in Hillsborough County, Florida

Michael N. Maltezos
Hillsborough County Public Works Department
601 E. Kennedy Blvd
P.O. Box 1110—22nd Floor
Tampa, FL 33601-1110
maltezosm@hillsboroughcounty.org

ABSTRACT

The American Society of Civil Engineers (ASCE) 2009 Report Card for America's Infrastructure offers an assessment of the state of our nation's infrastructure, with grades based on the most up-to-date information available. ASCE estimates the nation still stands at a D average. With each passing day, the inability of the country's aging infrastructure to meet the needs of our growing population further threatens our economy and environmental quality of life. Establishing a comprehensive, long-term infrastructure development and maintenance plan must become a priority for our policy leaders.

Public commitment is critical to any infrastructure improvement plan. Federal funding alone won't fix the nation's crumbling infrastructure. The public needs to understand that when they vote down a half-cent tax increase that would be set aside for transportation improvements, they are effectively sentencing themselves to car repair bills caused by potholes and more time stuck in rush hour traffic. State and local governments must also be part of the solutions.

Florida's already overburdened public services will be further threatened without proper investment in the additional capacity needed to support our increasing population. In addition to capacity planning, modernization of aging systems in Florida is fundamental to providing a safe and operational infrastructure while improving the quality of life for the state's residents. Furthermore, planning and allocating funds for the continued forecasted growth in the state is essential as existing facilities and services become obsolete, overburdened, or fall into disrepair. Fiscally responsible planning for sustainable and safe infrastructure 5, 10, 20, or 50 years from now must start today.

The desired outcome from the development and publication of this Infrastructure Report Card is for the state legislature and U.S. Congress to allocate funding for state infrastructure at levels that meet the need identified by this Report Card, to support infrastructure funding that promotes economic growth and a high quality of life in local governments throughout the state, and to obtain Florida voter support for infrastructure funding initiatives and fees. In Hillsborough County, Florida, the Public Works Department has performed analyses and developed recommendations to "Raise the Grade" in each applicable infrastructure category. As such, we have developed and initiated an Asset Management Program in accordance with GASB 34. This initiative encompasses the areas of operations and maintenance (O&M) and overall asset management planning and will be measured to improve the "whole life" cycle of assets.

Key words: America's infrastructure—ASCE Report Card—asset management

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The Iowa Travel Analysis Model iTRAM

Phil Mescher
Office of Systems Planning
Iowa Department of Transportation
800 Lincoln Way
Ames, Iowa 50010
phil.mescher@dot.iowa.gov

ABSTRACT

This presentation discusses the evolution of the Iowa Department of Transportation (Iowa DOT) statewide model, the key purposes for using a statewide model, and future uses of the model.

Key words: Iowa Department of Transportation—statewide model—travel

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Post-Grouted Drilled Shaft Load Test Program Council Bluffs, Iowa

Curtis Monk
Iowa Division
Federal Highway Administration
105 6th Street
Ames, Iowa 50010
Curtis.Monk@fhwa.dot.gov

ABSTRACT

The Iowa Department of Transportation and the Federal Highway Administration Iowa Division have been working to expand the use of drilled shafts for bridge construction for several years. However, drilled shafts have typically been limited to bridge sites that allowed the shaft to be socketed into rock or intermediate geomaterials such as shale. Because of site restrictions (including deep sand deposits, vibration concerns, noise, and stage construction) at the Broadway Viaduct bridge site, drilled shafts founded in medium dense sand were evaluated. The concept of post grouting or pumping grout under pressure to the base of a completed shaft to mobilize end bearing had been used by other state departments of transportation and federal agencies at similar sites. Before using the concept in Iowa, a load test program was conducted. The program consisted of load testing three shafts using the STATNAMIC method. Two shafts were post-grouted and one was un-grouted, which allowed an objective determination of the benefits of post-grouting. The test program identified issues that need to be resolved when using this method of drilled shaft construction. When those issues are addressed, the program confirmed that post-grouting is a viable concept and can be a cost-effective technique when constructing drilled shafts in sand.

Key words: base grouting—deep foundation—drilled shafts—STATNAMIC load test

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Comparison of On-Road Biodiesel Emissions in Transit Buses

Abhisek Mudgal
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
abhisek@iastate.edu

Massiel Orellana
Department of Agronomy
Iowa State University
102 Snedecor
Ames, IA 50011-1210
massiel@iastate.edu

Shauna Hallmark
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
shallmar@iastate.edu

ABSTRACT

Depleting fossil fuel resources and environmental protection concerns have triggered research on biodiesel in terms of its emissions reduction potential. However, the emissions impacts of biofuels are not well understood.

The objective of this research was to conduct on-road and laboratory tests to compare the emission impacts of different blends of biodiesel to regular diesel fuel under different operating conditions. The team conducted on-road tests that utilized a portable emissions monitoring system that was used to instrument transit buses. Regular diesel and different blends of biodiesel were evaluated during on-road engine operation by instrumenting three in-use transit buses from the CyRide system of Ames, Iowa, along existing transit routes. Evaluation of transit buses was selected for this study rather than heavy-duty trucks since transit buses have a regular route. This way, emissions for each of the biodiesel blends could be compared across the same operating conditions.

With B20, HC and PM emission rates decreased for all the buses. The decrease in PM emissions is significant, particularly for heavy-duty vehicles that use the most diesel. The decrease in HC is not significant for diesel engines. CO emissions showed inconsistency, which is also immaterial for diesel engines. NO_x emissions showed contradictions in the results.

Key words: biodiesel—emissions—transit bus

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Comparative Study of Costs Incurred by Transportation Users and Charges Compensated

Abhisek Mudgal
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
abhisek@iastate.edu

ABSTRACT

Studies show that motor vehicle users (both the automobile and truck users) in the United States seem to pay only about 77.9% of the costs they occasion. In this paper, it is shown through a series of studies how total external cost (primarily accident, emissions, and noise) is significant in comparison to other private costs of using transportation facilities. The globe is facing a challenge from the increasing number of cars leading to greenhouse gas (GHG) emissions and traffic congestion, causing a loss of millions of dollars. Internalizing the external costs can support shifting of ridership from auto to public transportation, evaluation (decision making) of the true cost and benefits of consuming a particular transportation facility, and increasing the awareness of the responsibility of minimizing accidents, emissions, and noise. While it is essential to assimilate these into the user cost, very limited research has been published that assigns actual dollar values to the costs of vehicle-generated externalities. There are challenges associated with (1) quantifying actual adverse affects and (2) aggregating a variety of road users who contribute differently to the transportation externalities. Even without considering the external cost, the present government expenditures fall short of the tax and fee payments. Considering the external cost would further supports the thesis that there is a significant discrepancy between the charges paid by the users and the actual cost incurred by them.

Key words: external cost—freight—private cost—transportation externalities—transportation tax

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Neural Networks Modeling of Biodiesel Emissions from Transit Buses

Abhisek Mudgal
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
abhisek@iastate.edu

Kasthurirangan Gopalakrishnan
Department of Civil, Construction, and Environmental Engineering
Iowa State University
353 Town Engineering
Ames, IA 50011
rangan@iastate.edu

Shauna Hallmark
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
shallmar@iastate.edu

ABSTRACT

Owing to growing environmental, economic, and geopolitical concerns of sustainability, there is growing perceived economic and political need for the development of alternative fuel sources. Biodiesel is an alternative fuel produced from domestic, renewable resources, such as vegetable oils and waste oil products. Roughly 90% of the biodiesel produced in the United States today is made from soybean oil. Many federal and state fleet vehicles now use biodiesel blends in their diesel engines. Biodiesel can be blended with diesel fuel in any proportion, since it is physically similar to petroleum-based fuel. There is a growing body of emissions data for biodiesel. Research studies have shown that compared to conventional diesel, the use of biodiesel significantly reduces particulate emissions (PM), carbon monoxide/dioxide (CO and CO_2), and hydrocarbons (HC). More research is required about the properties of biodiesel and its effects on the engine's performance and emissions.

In this research study, qualitative and quantitative biodiesel fueling performance and operational data were collected from urban mass transit buses in Ames, Iowa. A state-of-the-art portable emission measurement system (PEMS) unit was used to measure the pollutant emissions, along with ambient weather conditions, global positioning system (GPS) readings, and vehicle engine data. A low-cost method to estimate emissions is to develop predictive emissions models. On-road emissions data are characterized by high variability owing to a number of variables pertaining to vehicle operation. Therefore, non-linear models using speed, acceleration, and other vehicle dynamics parameters are required. Further, descriptive statistics of research results showed that emissions were not normally distributed, which narrowed down the scope of using statistical models, many of which require the data to be normally distributed. These conditions seem to be ideal for the application of Neural Networks (NNs), which are increasingly being used to solve resource-intensive complex problems as an alternative to using more traditional techniques such as regression methods.

In this study, the applicability of backpropagation NNs for determining the performance and exhaust emissions of a diesel engine fueled with biodiesel blends was investigated. Backpropagation NNs are very powerful and versatile networks that can be taught a mapping from one data space to another using a stipulated set of patterns/examples to be learned. Results showed that the engine-performance emissions

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can be predicted for different biodiesel blends with reasonable accuracy using the best-performance NN models developed in this study.

Key words: backpropagation—biodiesel—emissions—neural network

Identifying Thresholds for Run-Off-Road Events

Abhisek Mudgal
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
abhisek@iastate.edu

Shauna Hallmark
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
shallmar@iastate.edu

ABSTRACT

Run-off-road crashes cause one-third of all traffic fatalities, and two-thirds of those crashes occur in rural areas. In order to better address run-off-road and other crashes, the Strategic Highway Research Program 2 (SHRP2) program is planning a large naturalistic driving study that will instrument around 2,500 private vehicles. The advantage to the study is that it allows a snapshot into all driving behavior. The data collected will allow researchers to observe actual crashes as well as situations, called crash surrogates, where a crash may have occurred had the situation been slightly different. However, since a large amount of data will result, it will be necessary to identify vehicle parameters that indicate that a vehicle has left the roadway or its lane of travel. The Center for Transportation Research and Education at Iowa State University is working on a preliminary project to identify critical thresholds that indicate a run-off-road event has occurred. The study is attempting to identify vehicle kinematics, such as a lateral acceleration of a certain magnitude, which indicates this has occurred.

Key words: critical threshold—natural driving study—run-off-road crashes

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Nanotechnology to Manipulate the Aggregate-Cement Paste Bond: Impacts on Concrete Performance

Jessica M. Sanfilippo
Department of Materials Science and Engineering
University of Wisconsin, Madison
1415 Engineering Dr.
Madison, WI 53706
jmsanfilippo@wisc.edu

Marc A. Anderson
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1415 Engineering Dr.
Madison, WI 53706
nanopor@wisc.edu

Jose F. Muñoz
Department of Materials Science and Engineering
University of Wisconsin, Madison
1415 Engineering Dr.
Madison, WI 53706
jfmunoz@wisc.edu

Steven M. Cramer
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1415 Engineering Dr.
Madison, WI 53706
cramer@engr.wisc.edu

M. Isabel Tejedor
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1415 Engineering Dr.
Madison, WI 53706
mitejedo@facstaff.wisc.edu

EXTENDED ABSTRACT

It is well recognized that the area of contact between the cement paste and aggregates, known as the interfacial transition zone (ITZ), is one of the most vulnerable areas of concrete. The microstructure of this interface, the nature of its chemistry, and the high porosity coupled with the aggregate's surface and mineralogy affect the adhesion between the aggregate and cement paste and therefore, dictate the performance of concrete. Problems arise at this adhesion point because there is no current method in place to account for the differing aggregate assortments used in concrete. Currently, aggregate is tested to be sure it behaves as predicted and is useable, leaving out marginal sources. As concrete is the number one building material in the world and we continually consume this material, it is vital that we allow ourselves to use these less desirable sources, which leaves us with the question of how. How can we improve the material at the surface as to not decrease performance or jeopardize the durability? Can we engineer the properties of the ITZ using nanoporous films?

The traditional strategy for improving the nature of the ITZ has been the successive addition of different pozzolanic materials having a high surface area/particle size relation to the cement-paste fraction. However, the beneficial effects of these additions to the ITZ are diluted, since these additives largely end up in the bulk of the cement paste, where they attempt to manifest their improvements. The aim of this research is to show the benefit and practicality of depositing pozzolanic materials as thin films on the surface of aggregates used for concrete production to specifically improve the ITZ. Our preliminary results have shown that a small dose of these additives can significantly modify the adhesion between aggregate and cement paste and porosity in the ITZ.

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The consequence of this approach of adding nanoporous thin films to aggregate surfaces is an overall improvement of the principal mechanical properties of concrete. In particular, mortar made with a 0.03 silica-oxide/cement ratio and with the silica oxide deposited as a surface coating of just 1/3 of the total fine aggregates showed a 40% improvement in compression, flexural, and tensile strength at early ages along with significant decrease in the porosity of the ITZ. These results clearly indicate that the addition of silica oxides into concrete as thin films on aggregate surfaces has a high potential for improving overall performance of this highly consumed construction material. The direct application of these additives into the ITZ will increase their efficacy with respect to traditional supplementary cementitious materials. This new technology could be easily implemented to produce a stronger and more durable concrete.

As we look further into the mechanism for this improvement, we remember the question “Can we engineer the ITZ?” Although these mechanical improvements are positive, what is truly the potential of the material? We took a closer look at the ITZ, and through microscopy, we noticed that our materials reduce the porosity in the area of concern. We believe that these materials are undergoing a pozzolanic reaction and not simply increasing the surface area of the aggregate, allowing for adhesion simply due to roughness. As such, this material opens the door to solve further problems.

Currently, some aggregate sources are deemed unusable because deleterious microfines (particles <75 µm) found attached to the aggregate and/or degradation mechanisms, such as alkali-silica or alkali-carbonate reactions (ASR and ACR), threaten the integrity of the concrete. It is well known that clay minerals strongly absorb water and possess pozzolanic characteristics; they have extremely high potential to cause shrinkage and cracking. By placing our pozzolanic, nanoporous films over the clay, we can enable the transformation of the clay minerals into traditional hydration products when in the presence of high pH conditions supplied during concrete hydration.

We believe that ASR and ACR distress can also be ameliorated by using these nanoporous coatings. Depending on the nature of the nanoparticles needed to counteract the aggregate and microfine reactivity, two main mechanisms of amelioration are envisioned:

1. Isolating the source of reactive minerals. Some of our nanoporous films are inert and insoluble in the basic environment created by the cement paste. These films have negatively charged pores and will consequently prevent the diffusion of -OH groups to the surface of the aggregate, in effect, preventing reactions.
2. Creating an inhospitable ITZ for the development of ASR and ACR. Here the presence of highly reactive nanoporous films on aggregates will create an ITZ having little $Ca(OH)_2$ and thus, be an undesirable zone for the development of ASR.

The use of this new method for applying additives in concrete is expected to be much more efficient than the traditional dispersions of supplementary cementitious materials in cement paste. Furthermore, it will help to create a material with improved characteristics and less energy demand for maintenance. As an example, the higher strength developed at early ages will be not only useful for accelerating construction and rehabilitation operations but also in other areas of construction, such as joining compounds for use in the rapid construction of precast railway and highway bridge structures. Additionally, a concrete with higher values of tensile and flexural strength will likely show a reduced tendency for crack development, while a lower porosity in the ITZ should grant higher resistance to D-cracking. The overall results should be more durable concrete infrastructures by improving the highly vulnerable ITZ.

Key words: aggregate-cement paste bond—concrete durability—nanoporous thin films—porosity-mechanical properties

Sedimentation of Multi-Barrel Culverts

H-C. Ho
Department of Civil and Environmental Engineering
The University of Iowa
IIHR-Hydroscience & Engineering 100 SHL
Iowa City, IA 52246-5942
hao-che-ho@uiowa.edu

M. Muste
Department of Civil and Environmental Engineering
The University of Iowa
IIHR-Hydroscience & Engineering 307 HL
Iowa City, IA 52246-5942
marian-muste@uiowa.edu

ABSTRACT

Box culverts are generally designed to handle events with a 50 year return period and, therefore, most of the time they convey considerably lower flows. In many situations, water flow through a typical multi-barrel box culvert is relatively low throughout most of the year and usually concentrates in one barrel. A common adverse consequence for multi-barrel culverts when one barrel ends up carrying most of the flow during low-flow periods is that over several years of relatively low flow, some of the barrels may silt-in so as to become partially filled with sediment. Such sedimentation can reduce the capacity of culverts to handle the larger flow events and pose high-water problems upstream of culverts. This problem and the costs it incurs are compounded because many culverts are small enough in area, yet also rather long, that cleaning sediment from a partially filled culvert can be very difficult and costly. The problem is particularly severe for culverts draining small rural watersheds.

There is a need for methods of prevention or reduction of the in-filling of culverts, both for existing culverts and new culverts. Existing manuals, books, and guides do not provide adequate information on sediment control at box culverts, or for multi-barrel culverts generally. The present paper focuses on ways to ensure that multi-barrel culverts do not become silted in. The paper focuses especially on methods whereby culverts may self-cleanse themselves of sediment.

Key words: culverts—sedimentation—self-cleaning structures

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INTRODUCTION

Culverts are the common means to convey flow through the roadway system for smaller streams. Typically, the culvert is designed to handle storm events (e.g., 50 year return period discharge). Culverts may comprise multiple culvert pipes (a multi-barrel culvert) or a single culvert pipe. In general, larger flows and road embankment heights entail the use of multi-barrel culverts. The advantage of a multi-barrel culvert is that the requirement of upstream headwater is smaller than for a single-barrel culvert. However, the adverse documented effect is sedimentation (Vassilios 1995; Andrzej el. 2001; Charbeneau 2002). Multi-barrel culverts are prone to have sedimentation problems because of the channel transition connecting dissimilar cross-sections between the stream and the culvert. In the case where the design procedure suggests that required size and geometry of culverts extend beyond the width of the natural channel, channel transitions are required to convey drainage flow to and from the culvert. Expansion is needed upstream of the culvert, and contraction is needed downstream of the culvert. The transitions disturb the natural channel regime and have undesirable consequences, such as sedimentation through the culvert.

The present paper addresses sediment deposition pattern around the culvert and self-cleaning system designed to flush sediment out by using the power of drainage flows. The issues were investigated by a brief field survey and a series of laboratory and numerical experiments.

Field Survey of Multi-Barrel Culverts

Over many years of lower flow, some of the barrels of multi-box culverts can silt-in and become partially filled with sediment. A field survey was conducted in 2006, Iowa City, Iowa, USA. Figure1 shows a three-box culvert viewed from the road and upstream channel. The natural channel width was narrower than the culvert; the expansion was observed and sediment was allowed to deposit in this zone and barrels. This can reduce the capacity and result in decreased safety because the culvert may not perform according to design. The full field survey is reported by Muste et. al. (2009). This paper presented the site at the center line of the channel, which was approximately through the center of the culvert.

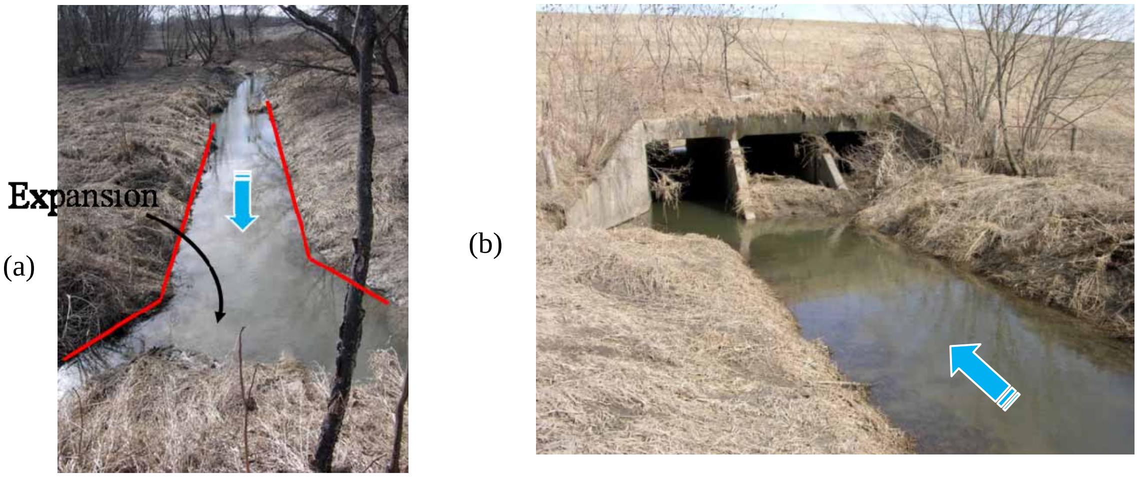

Figure 1. Culvert site at Iowa City, Iowa, USA—blue arrow indicates flow direction (a) look upstream from the culvert approximately along center line, (b) look downstream from the channel left bank

Investigate Tools

A triple set of tests was used to find a working, self-cleaning, multi-box design. A 1:20 scale three-box culvert model was used to replicate the baseline tests and screen the self-cleaning culvert configurations for their effectiveness to mitigate sedimentation problems for a range of flow conditions (Figure 2a). The tests were run using clear-water scour and continuous sediment feeding. Numerical simulations were used to refine the self-cleaning culvert geometry and test it for a range of flow conditions complementary to those tested in the laboratory experiments. Passive scalar visualizations were used to simulate the sediment transport. A 1:5 scale three-box culvert model was used to assess the performance of the designed self-cleaning culvert configuration (Figure 2b). The tests were run using live-bed scour and sediment recirculation. Extensive running times were used to achieve equilibrium for both flow and sediment transport.

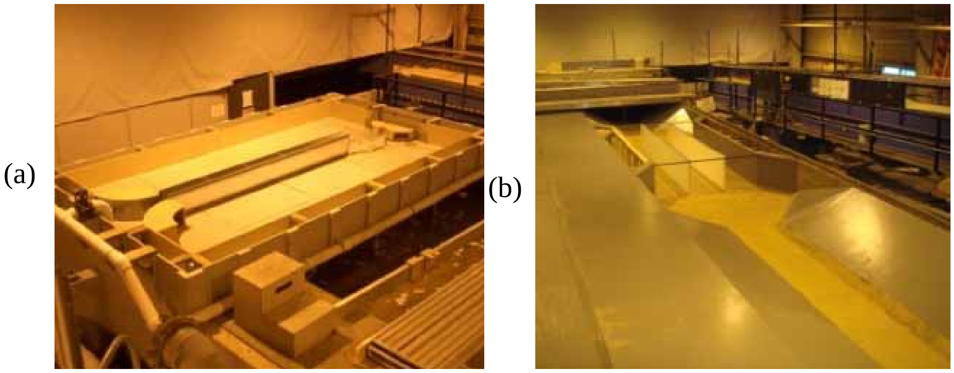

Figure 2. Overview of laboratory models—(a) a 1:20 scale three-box culvert model, (b) a 1:5 scale three-box culvert model

Design Concept of Self-Cleaning System

The basic concept of a self-cleaning system for sediment control was to increase the flow velocities and concentrate the flow to the main channel. The driving criterion for designing the self-cleaning culvert geometry was to make modifications in the upstream area of the culvert that would restore the shape and functionality of the original (undisturbed) stream. For this purpose, the lateral expansion areas were filled in with sloping volumes of material to both reduce the depth and to direct the flow and sediment toward the central barrel, where the original stream was located prior to the culvert construction. The fillet-based self-cleaning design is presented in Figure 3.

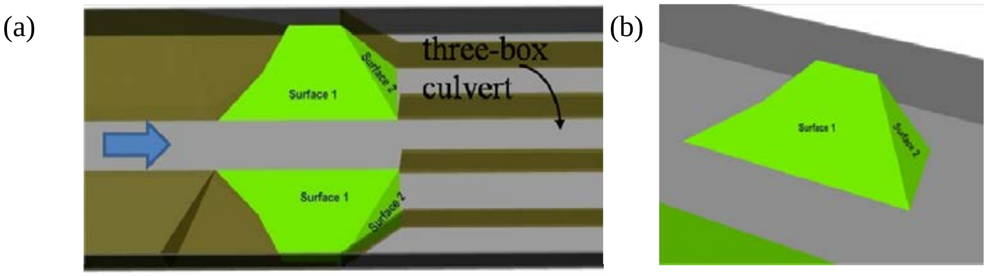

Figure 3. The fillet-based self-cleaning design geometry (a) fillets constructed upstream from the three-box culvert, (b) close-view of the fillet

Self-Cleaning System Outcomes

The fillet-based self-cleaning design developed through this study proved its reliability and efficiency through a variety of tests. Figure 4a shows baseline tests in the 1:20 model and the numerical model. A strong non-uniform velocity distribution was observed in the experiments. Sediment was prone to deposit and accumulate in the side of the expansion upstream the culvert. Figure 4b shows screening tests in the 1:20 model and the numerical model. The conditioned culverts (with fillets set in) displayed favorable flow behavior compared with the original ones. Among the fillets' main effects are the following:

1. Direct the sediment through the central barrel of the multi-box culvert
2. Maintain the effectiveness over a range of flows (even for the highest flows where small deposits are created, they do not obstruct the active area of the lateral culvert boxes)
3. Maintain the overall sediment transport rates within the boxes of the conditioned culverts at levels comparable with those in the original culverts

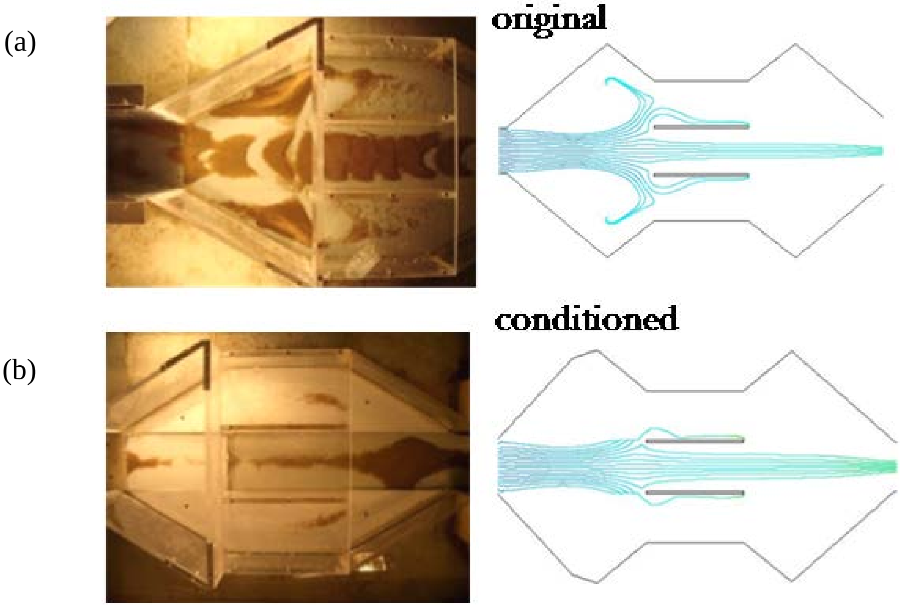

Figure 4. Comparison between the three-box culvert with and without self-cleaning system (a) no self-cleaning system in physical and numerical models, (b) self-cleaning system constructed in physical and numerical models

The efficiency of self-cleaning system was also conducted in the 1:5 model. Photographs (see Figure 5) of sediment deposition were taken from the same distance at an oblique angle using a reference in the images (the horizontal pole). The images allow us to observe that the sedimentation that occurs in the critical area of the upstream culvert expansion where deposition occurs at the highest rates and with the most detrimental impacts. Visual inspection of the images in Figure 5b shows that the self-cleaning fillets set in the expansion have the aforementioned effects.

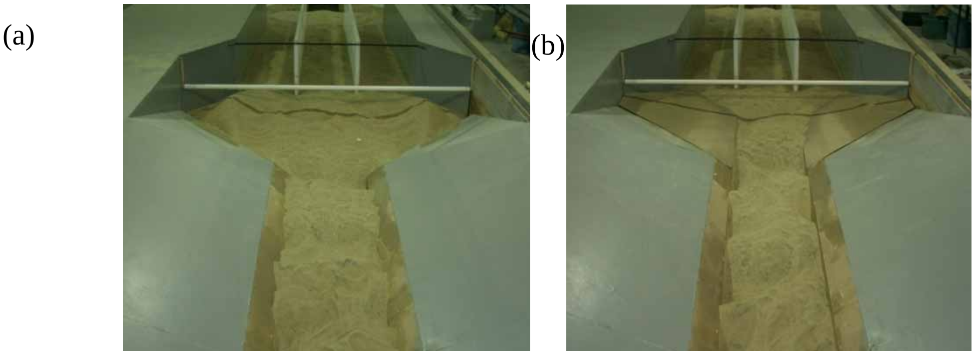

Figure 5. Sediment deposition patterns without and within fillet-based self-cleaning system (a) no self-cleaning system, (b) self-cleaning system constructed

Large-Scale Particle Image Velocimetry (LSPIV) was used to measure velocity distribution for the reference culvert model and the fillet-based self-cleaning culvert design. The isovelocity contours plotted in Figure 6 illustrate that the velocity magnitude was considerably increased throughout the center area of the expansion leading to an increased flow power that enhances the transport of sediment incoming toward the culvert. The LSPIV measurements undoubtedly demonstrate that water and sediment are forced to the central culvert box when the self-cleaning fillets are set in the expansion.

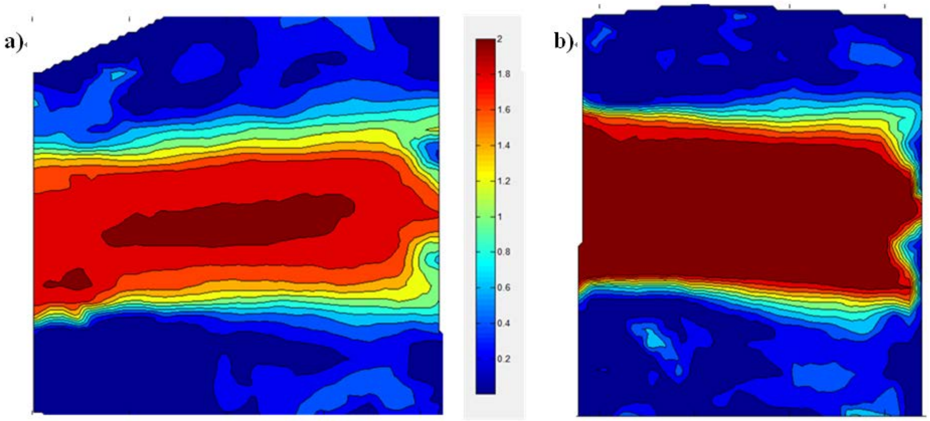

Figure 6. Velocity distribution upstream from the 1:5 culvert model (a) no self-cleaning system, (b) self-cleaning system constructed

CONCLUSION

Site visits of multi-barrel culverts in Iowa showed a common feature: sediment deposits developed in the upstream vicinity of the culvert. Severe sedimentation situations were encountered at several culverts. The deposits were partially blocking the culvert active area and usually were covered by vegetation. Cleanup operations are costly and for some of the visited culverts, were needed just two years after a previous cleanup. The main objective of this research is to understand and conceptualize the mechanics of sedimentation process at multi-box culverts and develop self-cleaning systems that flush out sediment deposits using the power of drainage flows.

Observations in the laboratory conducted in a 1:20 scale three-box culvert model, guided by companion numerical simulations, enabled the researchers to understand the mechanics of the sedimentation processes developing in three-box culverts, a typical culvert design for Iowa small streams. The first finding of the study was that the culvert design assumption of flow uniformity in expansion leading to the culvert is not correct. A strong non-uniform flow distribution was documented in the experiments through the culvert vicinity.

The fillet-based self-cleaning culvert design developed through the present study proved its reliability and efficiency through a triple set of tests (hydraulic model runs in the 1:20 and 1:5 scale models, and numerical simulations). The design is simple to implement in any stage of the culvert lifetime, i.e., at the time of construction or later on by retrofitting the area in the vicinity of the structure at the time of a cleanup. In the latter situation, the fillets can be mostly constructed with local material, i.e., the sediment deposited at the culvert is relocated in the area of fillets during the cleaning. The retrofitting using the actual sediment deposits are obviously the most efficient from cost perspective. The fillets such obtained can be “rip-rapped” and, possibly, grouted to roughen their surface for enhanced resistance to flow action. The grouting is also recommended for creating a vegetation barrier.

Due to the number and complexity of the factors involved in the sedimentation process and the limited amount of resources available for the study, only one culvert’s geometry was investigated. The modeled geometry replicates the triple reinforced box culvert (TRRCBG1- 01), which is typical for Iowa small streams. The flow approaching the structure was assumed to be perpendicular, despite the fact that many culvert situations depart from this layout. Finally, complex flow aspects related to modeling of sediment transport could have not been captured in the study, both because of existing knowledge gaps (e.g., triggering events, sedimentation-prone flow regimes) and modeling complexity (e.g., simultaneous suspended and bed load transport). An ongoing study will address many of these aspects and consequently further the results of the present investigation.

ACKNOWLEDGEMENTS

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Mass Concrete Thermal Control Case Study: I-80 over the Missouri River Bridge Construction

James S. Nelson
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, Iowa 50010
James.S.Nelson@dot.iowa.gov

John Gajda
CTL Group
5400 Old Orchard Road
Skokie, Illinois 60077
JGajda@CTLGroup.com

Ahmad Abu-Hawash
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, Iowa 50010
Ahmad.Abu-Hawash@dot.iowa.gov

ABSTRACT

Bridge construction projects with large concrete placements (typically in substructure components) are at risk for problems due to the heat of hydration generated after concrete placement. Concrete subject to heat of hydration problems is generally defined as mass concrete. There are two primary concerns with mass concrete heat of hydration. One concern is delayed ettringite formation (DEF), where excessive heat leads to unstable hydration, a long-term problem with DEF cracking that may not show up for years after construction. The second concern is thermal cracking due to temperature differential in the mass of concrete. Tensile strain in the concrete caused by temperature gradients between hot interior portions and the cooler exterior portions of the concrete can cause cracking if the strain exceeds the capacity of the curing concrete.

This presentation will be a general overview of mass concrete with a case study presented. The case study is I-80 over the Missouri River Bridge. The bridge, a $56 million construction project, has significant mass concrete placements due to the large substructure elements required in a border river bridge crossing with a long center span of 425 ft for navigation clearance. The Iowa Department of Transportation applied a special provision to the project with generally accepted practices of limiting maximum temperature and temperature differential among other requirements to reduce the risk of concrete cracking problems associated with mass concrete. The contractor chose to submit a Value Engineering (VE) proposal for a performance-based thermal control plan. The contractor hired a specialty consultant to develop the performance-based thermal control plan that took into account the specific mix design and materials used on the project. The mix was tested for adiabatic temperature rise and a thermal control model was created for the various mass concrete placements. Placement restrictions and methods, including internal cooling, were defined based on the thermal control model. The presentation will summarize the performance-based thermal control plan and results.

Key words: bridge construction—heat of hydration—mass concrete—substructure

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Characteristics of Fatal Truck Crashes in the United States

Nishitha Bezwada
Department of Civil Engineering
Kansas State University
2118 Fiedlar Hall
Manhattan, KS 66506
nbezwad@ksu.edu

Sunanda Dissanayake
Department of Civil Engineering
Kansas State University
2118 Fiedlar Hall
Manhattan, KS 66506
sunanda@ksu.edu

ABSTRACT

In 2007, one out of nine traffic fatalities resulted from a collision involving a large truck, of which 84% are occupants of other vehicles, even though the trucks accounted for only 3% of all registered vehicles and 7% of total vehicles miles traveled. This contrasting proportion indicates that truck crashes in general tend to be more severe than other crashes and particularly devastating for the occupants of other vehicles. To study this issue, fatal crash data procured from the Fatality Analysis Reporting System (FARS) were used and various conditions that prevail at the time of fatal truck crashes were evaluated. Findings indicate that around 73% of fatal truck crashes occur on rural roadways when compared to urban roadways. About 82% of all the fatal truck crashes occur on two-lane highways, and about 58% occur on two-way trafficways that are not physically divided. Also, almost 68% of the fatal truck collisions involve impacts on the front end of the vehicle, which weakens the argument on poor rear-side visibility being the main reason for truck crashes. Driving under influence was another critical factor, as around 700 drunken drivers are involved in fatal truck crashes every year. Another important observation was that of all the drunken drivers in fatal truck crashes, the non-truck drivers are observed to have greater alcohol involvement (around 62 %) than the truck drivers.

Several other factors have been observed for the better understanding of the fatal truck crash characteristics. By addressing these factors through the implementation of appropriate remedial measures, the overall truck crash rate can be reduced, which can help in improving the overall safety of the transportation system.

Key words: fatal crashes—large trucks—truck crashes—truck safety

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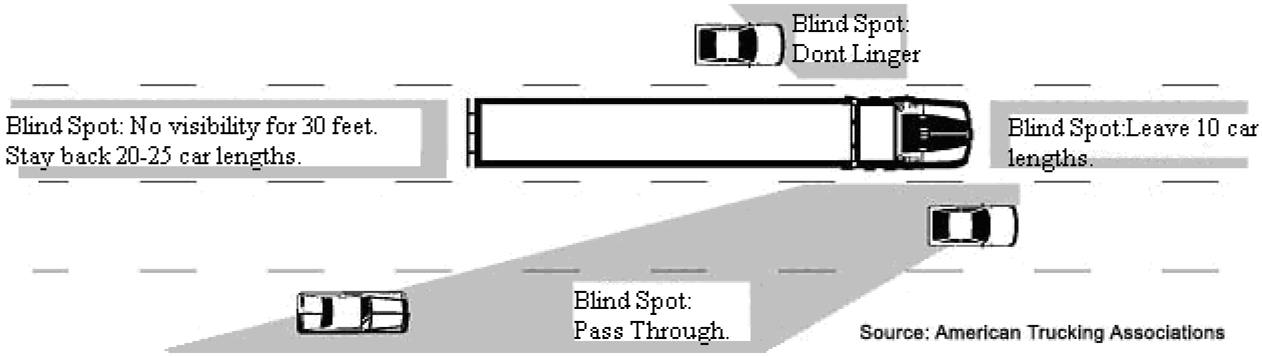
INTRODUCTION

Large-truck-related crashes contribute to a significant percentage of motor vehicle crashes in the United States, which involve fatalities and injuries. Of the 41,059 fatalities in motor vehicle crashes in 2007, 12% (4,808) died in crashes that involved a large truck, and 17% of those fatalities in large-truck crashes were occupants of large trucks. Though the large trucks contribute to only 8% of the vehicles involved in fatal crashes for the last 5 years, their impact in terms of severity proves to be a major concern.

Large trucks have different performance characteristics than other smaller vehicles. The physical dimension of the vehicle makes it difficult for drivers to maneuver large trucks smoothly on roadways. They can be 40 or more times heavier than the other vehicles in the traffic stream and have a slower initial pick up and a longer deceleration time. Truck drivers might face many challenges while traversing on Interstate or state highways at high speeds, at intersections, or while taking turns to have control over the vehicle. Also, the element of blind spots, as shown in Figure 1, makes it even more challenging for the truck driver and the surrounding vehicle drivers to avoid the heavy crash risk.


Blind Spot:
Dont Linger
Blind Spot: No visibility for 30 feet.
Stay back 20-25 car lengths.
Blind Spot:Leave 10 car lengths.
Blind Spot:
Pass Through.

Source: American Trucking Associations

Figure 1. No zones, or blind spots, around a large truck

The crash statistics observed from the previous years, as seen in Table 1, show significant consistency in the frequencies of the different categories of large-truck-involved crashes. These trends reflect the need for a more effective analysis, which would provide characteristic facts pertaining to these crashes and help generate productive remedial measures. Achieving effective safety goals to downsize the intensity of the issue will require approaching truck safety aspects from a variety of parameters.

Table 1. Large-truck-involved crash statistics

Year	Injury Crashes	Property Damage Only (PDO) Crashes	Fatal Crashes	Single Vehicle Fatalities	Multi Vehicle Crash Fatalities	Total Fatalities
2002	90,000	322,000	4,224	449	4,490	4,939
2003	85,000	347,000	4,335	457	4,579	5,036
2004	83,000	312,000	4,478	469	4,766	5,235
2005	78,000	341,000	4,551	478	4,762	5,240
2006	77,000	287,000	4,321	499	4,496	4,995

Source: Large Truck Safety Facts 2006

The amount of truck travel is dramatically increasing with the growing rate of factors like freight transport, which in turn requires continued attention in order to find ways of reducing truck crash risk.

The Federal Motor Carrier Safety Administration (FMCSA) has set as a goal of “50 by 2010,” a 50% reduction in commercial-truck-related fatalities by the year 2010. Accordingly, it is important for the safety community to identify the characteristics related to large truck involved fatal crashes.

PROBLEM STATEMENT

To attenuate the fatal truck crash frequency in the country and achieve the sustainability of this trend seems difficult with the growing rate of movement of people and goods throughout the country. Hence, it is essential to analyze the situations under which fatal truck crashes are occurring. These factors that prevail at the time of a fatal truck crash and their frequencies/rates can give a picture of the conditions under which a larger proportion of such crashes occur.

This study deals with the identification of these characteristics for all fatal crashes in the country for the period of 2003–2007. Also from these observed characteristics to make reasonable suggestions for the mitigation of the fatal truck crash risk.

LITERATURE REVIEW

Numerous researchers had investigated and analyzed truck crashes using various techniques and sources. Krishnaswami, Blower, Schneider, and Putcha (2005) conducted an elaborate study for nearly a decade in establishing a unique truck-crash characteristic database and analyzed several parameters related to truck crashes. Data for this project was acquired from a number of sources, including Fatality Analysis Reporting System (FARS), Trucks Involved in Fatal Accidents (TIFA), and General Estimates System (GES). This paper analyzed the causes of heavy truck-driver aggressiveness impact in two-vehicle truck/light-vehicle crashes and also derived detailed models that have helped propose countermeasures to mitigate collision severity.

Another analysis conducted using the same data sources on the rear-end fatal truck crashes (Craft 2002) had observed that though trucks initiate a collision by striking the other vehicle, in fatal crashes, trucks are struck by other vehicles more often. Also, the overlapping effect of light condition and the alcohol-involvement level of the drivers were observed, and it was seen that the other-vehicle drivers were more often involved in alcohol consumption under all light conditions.

In a study about (Williams, Allan, and Shabanova 2003) motor vehicle crash rate comparisons made with respect to truck or non-truck drivers, their at-fault status was observed as the chief criterion. Data from FARS were used for the period of 1996–2000. Drivers in fatal, single-vehicle crashes were assumed to have responsibility for the crash. In fatal two-vehicle crashes, driver-operator errors reported by police were used to assign crash responsibility. Tables based on the deaths in crashes involving one or more passenger vehicles for which drivers of various ages were likely to be responsible per 100,000 licensed drivers by occupant type and many other categories were calculated.

Many other projects based on the analysis of driver parameters like age and gender were used to generate models using the driver-behavior factors to have a precise understanding of the driver issues in crashes. Crum and Morrow (2002) investigated the influence of carrier scheduling practices on truck driver fatigue by developing and empirically testing a truck driver fatigue model. Earlier than this, Massie, Green, and Campbell (1997) had developed another model with the four predictor variables of driver age, gender, time of the day, and average annual mileage. The effect of these four variables on crash involvement rate was studied, and their level of significance was obtained.

In order to identify the unsafe driver actions that lead to fatal car-truck crashes, a study analyzed two-vehicle crashes in the 1995–98 FARS database to compare car-car crashes with car-truck crashes (Lidia, Kostyniuk, Fredrick, Streff, and Zakrajsek 2002*)*. In this, the 94 at-fault cases categorized as per the FARS were used to see the predominant faults in both types of crash situations. A key finding of this study is that most of the 94 unsafe driver acts were about as likely in fatal car-truck crashes as in fatal car-car crashes. Therefore, general safe-driving practices are also relevant around large trucks.

United States General Accounting Office (2003*)* made a report on the "Share the Road Safely" program, whose goal is to educate the public about driving safely around large trucks. This report analyzed the crash risk factors that predominantly arise while driving around large trucks. The program elaborated the necessity of having specific roadway educative measures for the public to mitigate this issue and lower the truck crash rate in general.

OBJECTIVES

The most primary objectives of this study include

- To analyze and evaluate various crash characteristics that prevail at the occurrence of fatal truck crashes
- To observe the trend of the crash occurrence under the overlapping effect of two-crash parameters

DATA AND METHODOLOGY

Data for the study was procured from the National Highway Traffic Safety Administration's FARS for a period of 2003-2007. The database documents descriptive data on vehicles, drivers, roadways, and environmental conditions collected from police reports, emergency medical service reports, hospital records, and coroner's reports of all fatal crashes in the country. The data are categorized into accident, person, and vehicle files. The accident file consists of all the general characteristics of the crashes; the person file has the details of every person involved in the fatal crashes, and the vehicle file explains the vehicular details. Every crash is given a unique identification code by which the files were merged using the statistical analysis computing software.

The accident file was primarily merged with the vehicle file to extract all the crashes involving a large truck (body weight >10,000 pounds). The various crash characteristics are recorded by using the filtering techniques in Microsoft Excel and Access. To obtain information about all the people in that crash, this file is in turn merged with the person file.

After suitably merging and filtering these files, the fatal truck crash data for the 5-year time period of 2003–2007 were combined to obtain more consolidated results with respect to several parameters and their frequencies. Further, the values obtained were compared at various levels to analyze the trends and patterns of a specific crash parameter with respect to time or type of crash or the extent of fault of the drivers involved. Eventually, certain pairs of parameters were overlapped to observe the contrasts in the combination of conditions prevailing during higher crash occurrence level. These trends are used to make critical inferences by interpreting them in the most pragmatic conventions.

RESULTS AND FINDINGS

The present study has shown that large trucks contribute to more fatalities in other (non-truck) vehicles than in trucks themselves. On an average, 84% of the fatalities occurring in large-truck crashes in the country are not the occupants of trucks. This reinforces the threat large trucks impose on other motor vehicles, pedestrians, and pedal cyclists.

Initial Point of Impact

One of the primary observations made on the data was to obtain the direction of impact, which is the initial point on the truck where the other vehicle collides. As already shown in Figure1, trucks have blind spots in all directions; this parameter helps show which zone is more crucial for a higher crash risk. By observing the initial point of impact on the truck, the position of the colliding vehicle with respect to the truck was estimated. From that, the blind spot, which results in a higher crash rate, was interpreted. From the Figure 2 is seen that the almost 62.5% of the cases resulted with the trucks having the initial impact on their front side. This might weaken the argument that poor visibility range for trucks on their rear side leads to majority of rear-end crashes in trucks. From this, it was also inferred that other vehicle drivers should be more vigilant when driving in front of a truck rather than the rear. The figure also shows around 15.5% of the crashes on the left-hand side of the driver. This can be considered significant because from Figure 1, it is observed that the left-hand side of the truck driver has the smallest blind spot zone when compared to all the other directions.

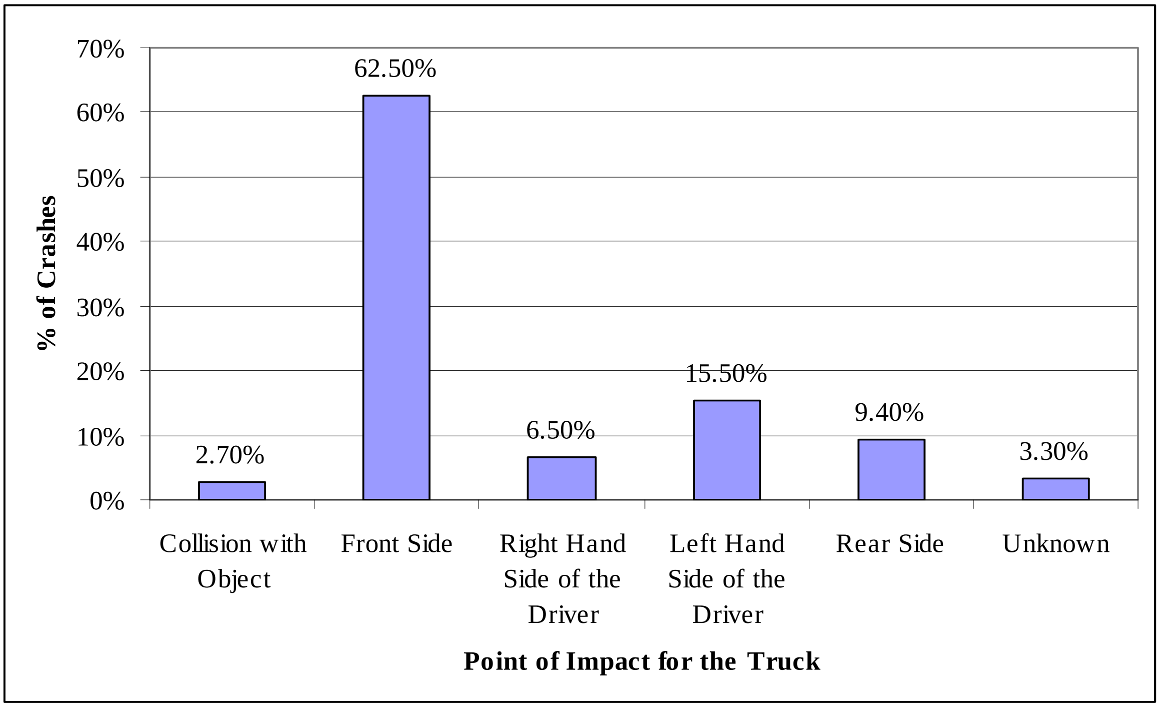


Figure 2. Point of impact for trucks in fatal crashes for the period of 2003–2007

Alcohol Involvement

Alcohol involvement of drivers has the potential to be one of the most significant contributing factors to result in crashes, which could also be the case in truck crashes. Analysis of this factor shows that of all the drunken drivers involved in fatal truck crashes, only 12.7% are truck drivers, and the rest of the 87.3% are non-truck drivers with blood alcohol levels higher than the permissible 0.8 mg/ml. This clears the misconception that a larger percentage of truck drivers are under influence of alcohol/drugs. Hence, it can be deduced that in fatal truck crashes with alcohol involvement, the non-truck drivers are largely at fault.

Manner of Collision

The manner of collision of the trucks in fatal collisions was observed for the all crashes from the combined dataset for the period of 2003–2007, as shown in Figure 3. Angle crashes have the highest proportion of 34.2%, followed by 23.7% of cases where the vehicles collide with a fixed object like a tree or a guardrail, etc. Head-on and rear-end crashes also form a significant portion of crashes, resulting in more fatalities.

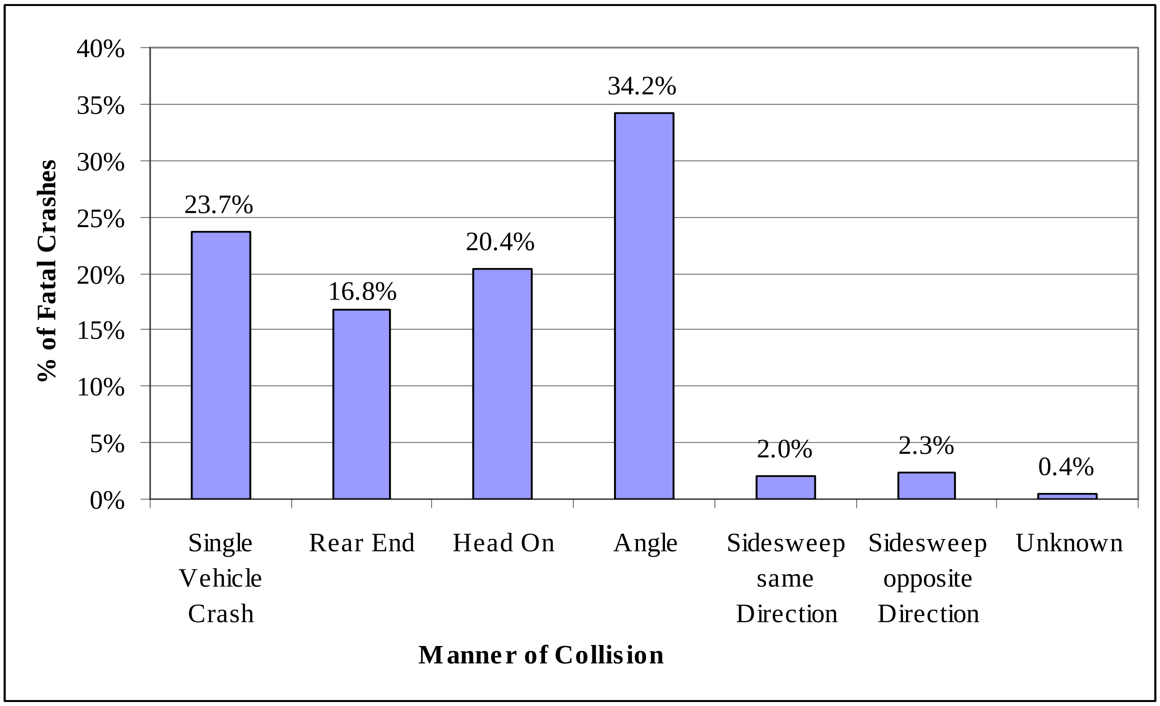


Figure 3. Manner of collision of fatal truck crashes

Speed Limit

Trucks are difficult to maneuver smoothly, and when at higher speeds, they have a risk of losing control. This can also be one of the primary factors contributing to higher crash risk. The speed limit of the roadway where the truck is traversing before succumbing to fatal crash can approximately show the speed of the truck. As seen in Figure 4, the percentage of fatal crashes increases with increase in speed limit up to 60mph. The range of 51–60 mph has the highest number (an average of 5,280 crashes per year) of fatal truck crashes in the past five years. The sudden drop in the number of crashes from 51–60 mph to 61–70 mph can be because of the smaller number of roadways with the later speed range.

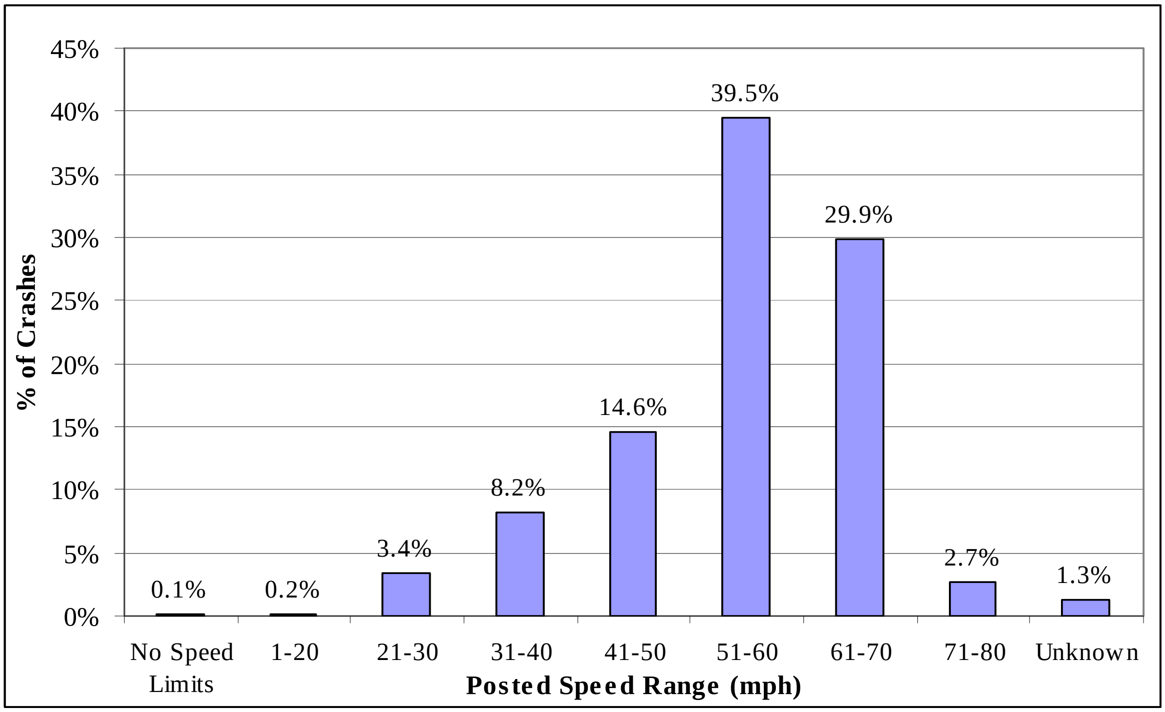


Figure 4. Fatal truck crashes in different speed limit ranges

Truck Driver Age Group

A number of driver-related parameters can be responsible for influencing the crash risk factor, especially for trucks that travel on a commercial basis for longer and more strenuous hours. In a study made by Crum and Morrow (2005), they explain that truck driver fatigue plays a major role in the occurrence of a crash. They have investigated and established a driver fatigue model to test various carrier scheduling practices with other driver parameters. Another important study was done by Williams, Allan, and Shabanova (2003) to scale the amount of responsibility in drivers by age and gender for all motor vehicle crashes. Here, they compared the number of drivers at fault in different age groups and gender. From their analysis, they proved that the element of “responsibility” declined with age until about age 63 then increased as a function of age.

From Figure 5, it is seen that the number of drivers involved in fatal truck crashes is higher in the age range of 41–50 yrs than other groups. The age group of 41–50 yrs has the highest percentage (29%) of fatal truck crashes, which may be the effect of driver fatigue factor having a larger impact on this age group.

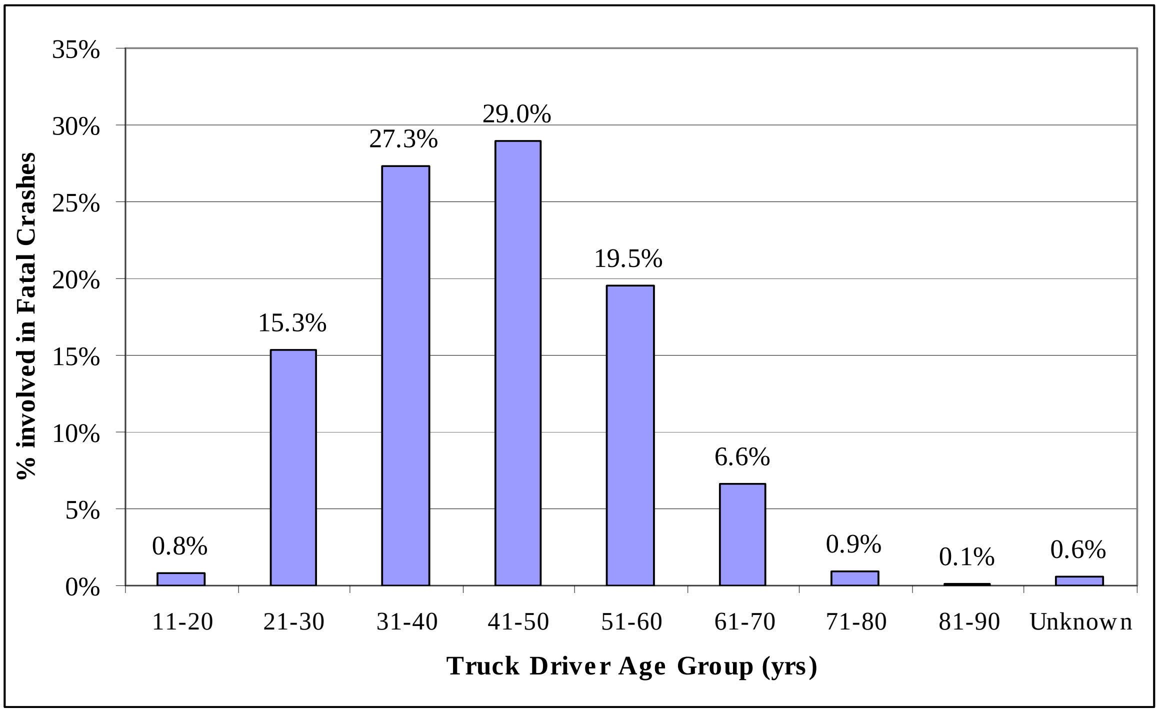


Figure 5. Age range groups of truck drivers involved in fatal truck crashes

Types of Traffic Ways

Truck maneuverability becomes more challenging with different kind of roadways, and even actions like lane changing and lane merging can sometimes become critical factors in leading to a crash. Also, the presence of dividers also affects the number of fatal crashes because they have the potential to reduce the severity of an occurred crash and sometimes even prevent fatality.

From Figure 6, it is seen that a majority of almost 23,968 crashes have occurred on two-way trafficways with no physical division in the past ten years. This shows that this kind of roadway has a greater tendency in promoting the occurrence of fatal crashes. Traffic flowing in opposite directions with no physical division in between can be one of the most suitable situations where the smallest of human errors can result in highly severe crash scenarios. Roadways of this type should be improved by providing the necessary divisions so as to minimize the frequency of fatal truck crashes.

The number of lanes on the roadways with these 23,968 crashes was analyzed, and it has been observed that almost 20,848 of those crashes occurred on two-lane, two-way roadways. The difficulty in controlling the large size of the vehicle in narrow or small roadways can be the reason for this high frequency. Two-lane roadways are often congested and cannot be easily traversed. This situation, in conjunction with the two-way trafficway without any physical division, can be the scene causing the occurrence of a fatal truck crash.

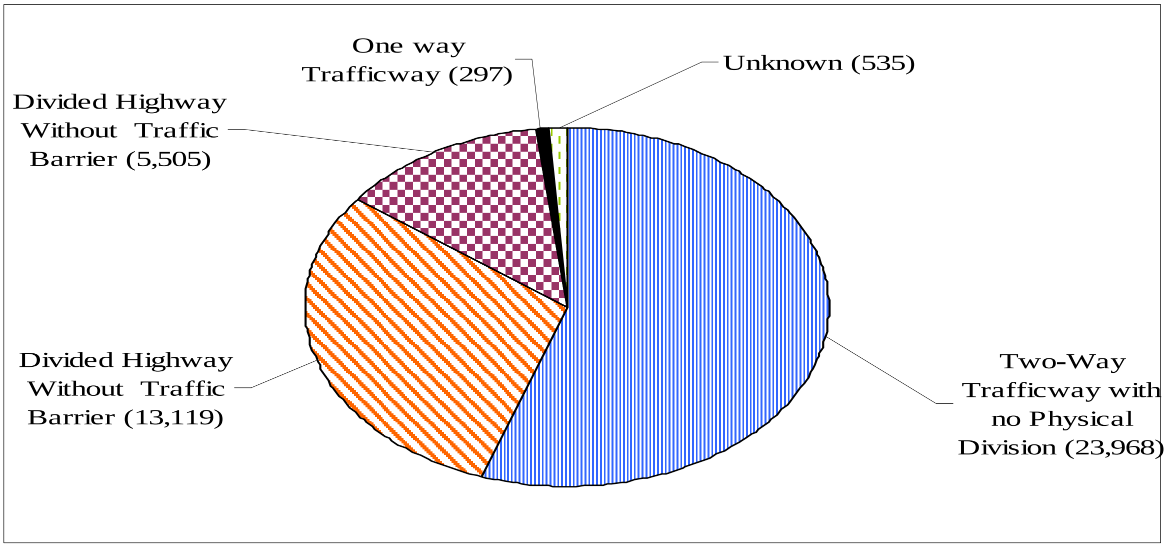


Figure 6. Proportion of fatal truck crashes on different traffic flow ways

Level of Deformation on Urban and Rural Roadways

As seen in Figure 7, the level of deformation of the vehicles involved in fatal truck crashes is severely disabling in most cases. This trend remains consistent in both urban and rural roadways. As large trucks are heavy in weight and volume and also as it was observed in Figure 4 that majority of fatal truck crashes occur at high-speed levels, it is therefore evident that the consequences of such conditions result in severe impact on the collided vehicles. However, the percent of severely disabled vehicles is proportionally smaller in the urban sector when compared to the rural sector. The availability of more space for maneuvering on urban roads could probably be the reason for this observation.

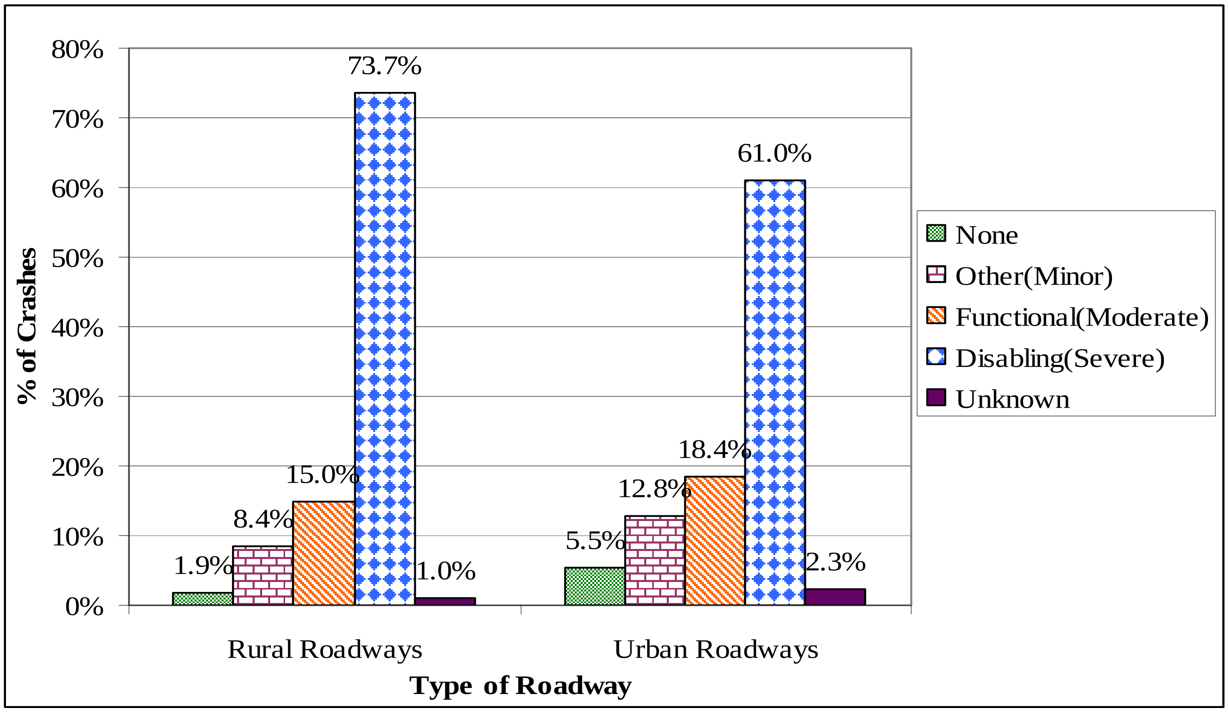


Figure 7. Level of deformation of all vehicles involved in fatal truck crashes

Truck Driver At-fault Factors

Figure 8 explains the various types of truck driver-related factors that may have contributed to the fatal crash. Around 28.1% of the truck drivers are observed to have contributed to fatal truck crashes due to non-compliance to traffic regulations. Improper driving is another factor, which in 24.6% of cases, has contributed to fatal truck crashes.These categories will include factors like running off the road, erratic lane change, following improperly, failure to keep in lane properly, etc. Also, the figure shows that 15.8% of truck drivers involved in fatal truck crashes have an improper mental condition, such as fatigue, drowsiness, inattentiveness, drugs, etc. Such factors can contibute heavily to the occurance of a crash.

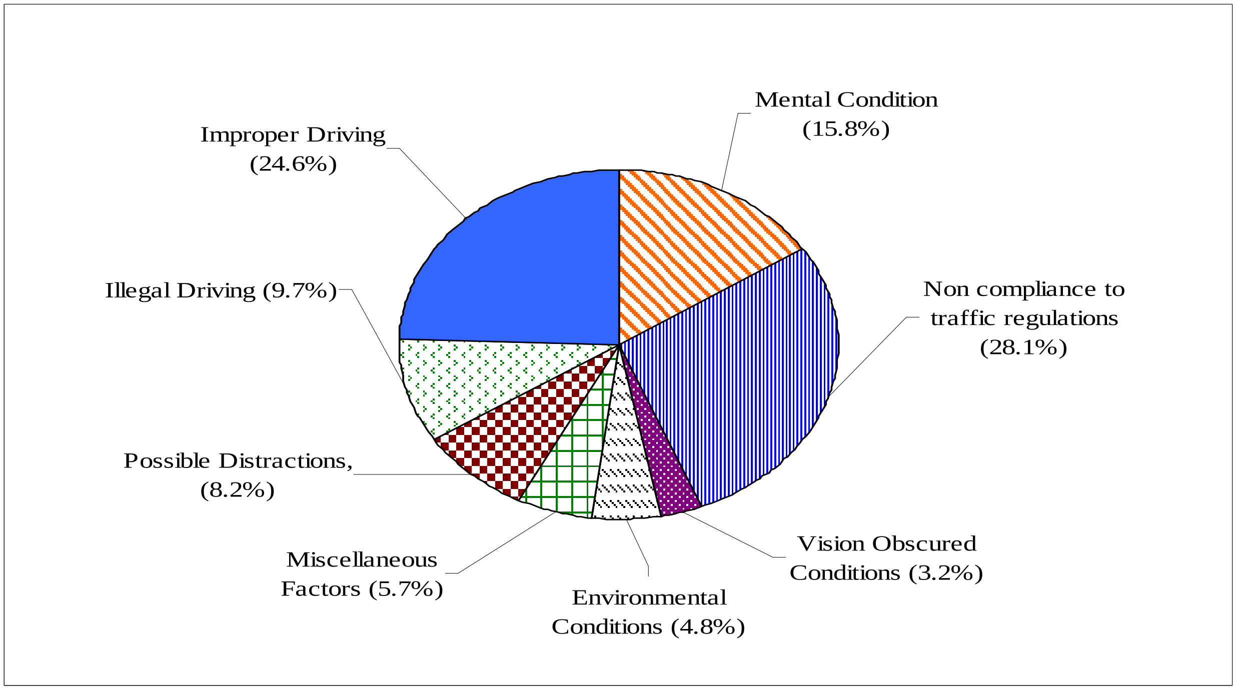


Figure 8. Truck driver-related contributing factors in a fatal crash

Truck Striking/Struck on Different Roadways

Ralph Craft (2002) studied the rear-end truck crashes by comparing those where the truck was the striking vehicle and those where the truck was the struck vehicle. A similar framework was adapted to the current dataset, as shown in Figure 9, to observe the crashes on different types of roadways in the past five years.

It was observed from this that the "truck striking" and "truck struck" categories have a high number of crashes on state highways, contrasting the "other crashes" category, which have a high number of crashes on Interstates rather than other types of roadways. Truck striking another vehicle results in higher number of crashes than being struck on both Interstates and state highways, but this comparison has equal proportion in case of U.S. highways.

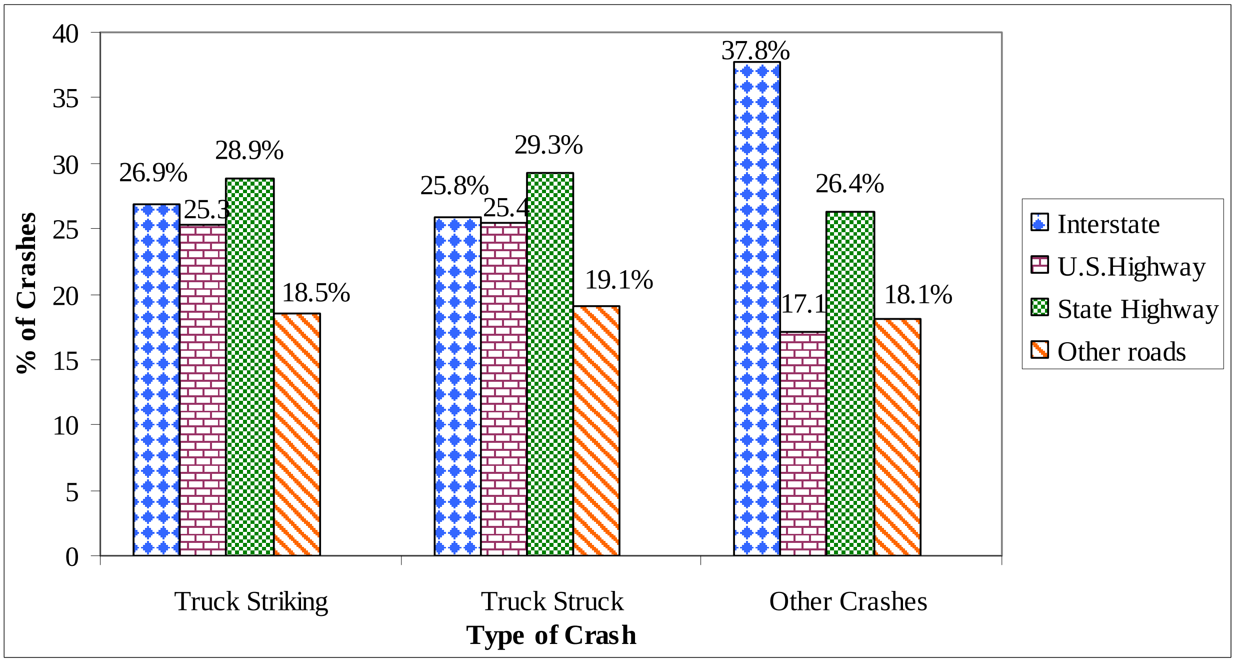


Figure 9. Fatal truck crashes by roadway type

Truck Striking/Struck in Different Light Conditions

When a similar criterion is used for different types of light conditions as shown in Figure 10, it was observed that the proportion of cases where trucks are struck has a lesser value than cases where the truck strikes other vehicles. In constrast, the percentage of trucks struck is higher in "dark/dark but lightened conditions" when compared to cases where the trucks are striking other vehicles.

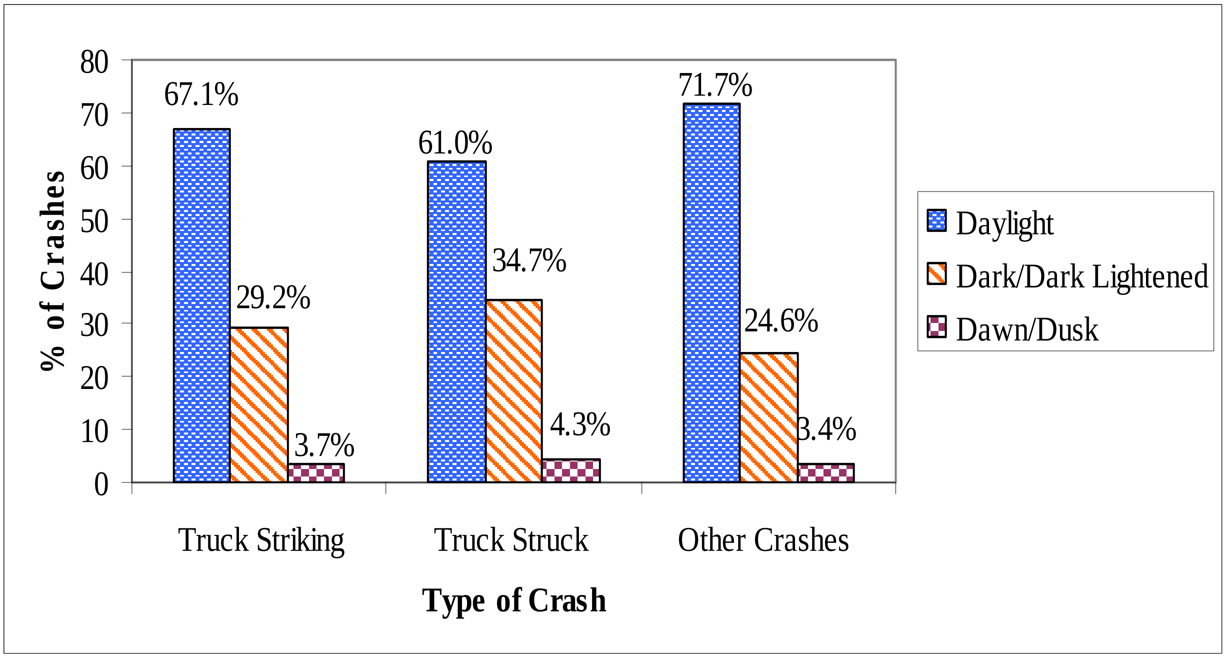


Figure 10. Fatal truck crashes in different light conditions

CONCLUSIONS

Certain significant characteristics of fatal truck crashes have been observed from this analysis. The fatal crash frequency was observed to be greater with vehicle in front of the truck rather than anywhere else. In case of alcohol involvement, non-truck drivers seem to have indulged themselves in almost 87% of cases, proven by the high rate of blood alcohol level. Trucks seem to suffer majority of the fatal crashes at higher speed levels like 51–60 mph. Fatigue factor can be a leading characteristic that increases the number of fatal truck crashes with increase in age of the truck driver. Two-way, two-lane traffic flow ways with no physical division are leading to higher crash risk and fatalities. Such roadways should be altered by providing the necessary equipment. Improper driving and non-compliance to traffic regulations have also been observed to be the chief driver-related contributing factor in the case of fatal truck crashes.

By comparing the simultaneous effect of two-truck fatal crash characteristics, “truck striking” and “truck being struck” seemed to have similar proportions on all roadway types. Also, this proportion remained consistent, even under different light conditions. In general, several other factors can be critically observed later in a comparative study to see the at-fault criteria of truck and non-truck drivers can be analyzed to obtain a more detailed picture of this present analysis.

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The *Actual* Cost of Food Systems on Roadway Infrastructure

Inya Nlenanya
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
inya@iastate.edu

Omar Smadi
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
smadi@iastate.edu

ABSTRACT

This project is designed to provide more insight into the infrastructural challenges of agricultural enterprises in the state and to also facilitate the understanding needed to implement broader energy-related policy and planning. Food makes up a significant portion of roadway freight. On average, most food travels over 1,500 miles from farm to table in trucks that each causes an amount of damage equivalent to 10,000 passenger cars. Also, the increase in truck freight compounds structural damage, congestion, and carbon emissions and compromises road safety— just to mention a few of the important issues with our transport system. Using Iowa Department of Transportation (Iowa DOT) data on the highway system, this endeavor aims to capitalize on current research efforts to develop a systematic methodology for estimating the actual cost of moving food produce from farm to market, including the following: environment (carbon emissions and air quality), infrastructure, energy (fuel), congestion, safety, and user (taxpayer) costs.

Key words: air quality—carbon emissions—congestion—cost per mile—food system—freight—roadway cost

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A Framework for Performance-Based Permeability and Density Acceptance Criteria for HMA Pavements in Wisconsin

Sam Owusu-Ababio
Department of Civil and Environmental Engineering
University of Wisconsin, Platteville
136 Ottensman Hall
Platteville, WI 53818
owusu@uwplatt.edu

Robert L. Schmitt
Department of Civil and Environmental Engineering
University of Wisconsin, Platteville
132 Ottensman Hall
Platteville, WI 53818
schmitro@uwplatt.edu

ABSTRACT

Asphalt-surfaced pavements constructed with the appropriate cross slopes are intended to provide a watertight surface to promote safety and preserve pavement structural integrity from moisture damage. The asphalt mixture must be compacted to the desired density to limit air and water permeability and resist permanent deformation (rutting). Although it is widely recognized that excessive water permeability can significantly increase the potential for poor pavement performance, the relationships between permeability, density, and in-service performance of asphalt pavements have not been clearly defined. A prior study has suggested a minimum compacted density of 92.3% of Gmm is needed to minimize excessive permeability of in-place Hot Mix Asphalt (HMA) pavements having 9.5 and 12.5 mm nominal maximum size (NMAS) mixtures. This air-void content corresponds to a permeability value of $100\text{x}10^{-5}$ cm/sec, with higher values for larger NMAS mixtures. These critical values for in-place air voids and permeability are based solely on their relationship and are only empirically derived. This paper presents a framework for defining permeability and density criteria based on performance of in-service pavements.

Key words: density—HMA—pavement—performance—permeability

INTRODUCTION

The placement of impermeable Hot Mix Asphalt (HMA) pavements has been recognized as an effective means to protect against rapid oxidation of the binder materials and excessive moisture damage in HMA pavement layers. In many open-graded friction course (OGFC) mixtures the water is allowed to flow through the compacted pavement. Density and permeability relationship studies performed by Cooley et al. (2001) indicated that an in-place air-void content below 7.7% is needed to minimize excessive permeability of in-place HMA pavements having 9.5 and 12.5 mm nominal maximum size (NMAS) mixtures. This air-void content corresponded to a permeability value of 100×10^{-5} cm/sec. For mixtures having a 19.0 mm NMAS, the in-place air-void content was found to be 5.5% and yielded a permeability value of 120×10^{-5} cm/sec. These critical values for in-place air voids and permeability were based solely on their relationship and are only empirically derived.

Intuitively, an excessive amount of permeability significantly increases the potential for poor performing pavements; however, the relationships of both air voids and permeability with actual field performance have not been clearly defined. The principal objective of this study was to develop a framework for establishing target permeability and density values to yield acceptable performance of in-service pavements. The framework was developed and tested using in-service HMA pavement data on permeability, mix properties, and performance.

DATA COLLECTION

The collection of data involved an initial office review and selection of appropriate in-service pavement segments plus field measurements of permeability and density of in-service HMA pavements.

Office Review and Selection of HMA Projects

To aid in the project selection, databases involving design, new construction reports, traffic, and performance were obtained from the Pavement Management Unit of the Wisconsin Department of Transportation (WisDOT) and reviewed. The construction database indicated a total of 68 warranted HMA pavements were constructed in Wisconsin between 1995 and 2004. Since warranted pavements are believed to perform better than non-warranted pavements, the data was divided into warranty and non-warranty asphalt pavements. Three age groups were considered in this selection including 10-, 7-, and 5-year-old pavements. The 10-year selection was made to provide understanding of permeability impacts on field performance of pavements constructed during the beginning of warranty practices in Wisconsin (i.e., 1995). The 7-year-old pavements were selected to represent pavements constructed at the beginning of the implementation of Superpave mix design practice in Wisconsin, while the 5-year-old pavements were chosen to represent pavements that had just moved out of the warranty stage and no longer under the jurisdiction of the contractor. The majority of warranty projects were performed on roadways functionally classified as Class 10 (rural principal arterial) and Class 20 (rural minor arterial). Hence, the selection was made for only these two functional classifications.

It has been well documented in the literature that several factors affect permeability, such as aggregate gradation and density. However, a comprehensive literature review yielded no investigations relating field performance and permeability. Since the principal focus of this study was on the relationship of performance and permeability, various key performance indicators were evaluated at this stage of the study, including rutting, edge raveling, and Pavement Distress Index (PDI). The PDI measures overall condition of the pavement in terms of all distresses. It is measured on a scale of 0 (no distresses) to 100 (worst surface condition). Key distresses related to permeability and density are rutting and raveling.

Rutting distorts the pavement cross section and has the potential to trap water when it is of significant depth. The standing water can facilitate the initiation or progression of moisture-related distresses, thus causing water to permeate the pavement. Edge raveling may be an indicator of permeability-related distress, and was assessed during project selection.

Once the warranty projects were selected, non-warranty projects of similar age and functional class were identified. The non-warranty projects were chosen either along the same route in the immediate vicinity, or in the same geographical region. This was purposely done to block the effects of climate and variability in traffic patterns on pavement performance. In addition, selection of projects closer to warranty projects reduced travel distance from one project to another and helped to effectively manage field data collection resources.

Field Measurements of Density and Permeability

For each project, a 0.1-mile segment was selected, usually located 0.3 mile from the beginning of a selected WisDOT designated reference point (RP) along the roadway. Within the 0.1-mile segment, 20 test locations were randomly chosen; 10 test locations were selected within the wheel path to assess the relationship of permeability with rutting and densification from traffic, while the remaining 10 were selected between the wheel paths to evaluate permeability and densification similar to as-built conditions.

Eight of the 20 test locations were randomly selected for water permeability and core testing, where four locations were within the wheel path and the remaining four locations between the wheel paths. Figure 1 shows a schematic for test locations within a 0.1-mile test segment. The reason for sampling only eight locations was to balance the sample size with an allotted budget for a half-day testing on each segment. The purpose for the 20 air permeameter and nuclear density sites was to strengthen the data set, since the number of water permeability and core test sites was limited.

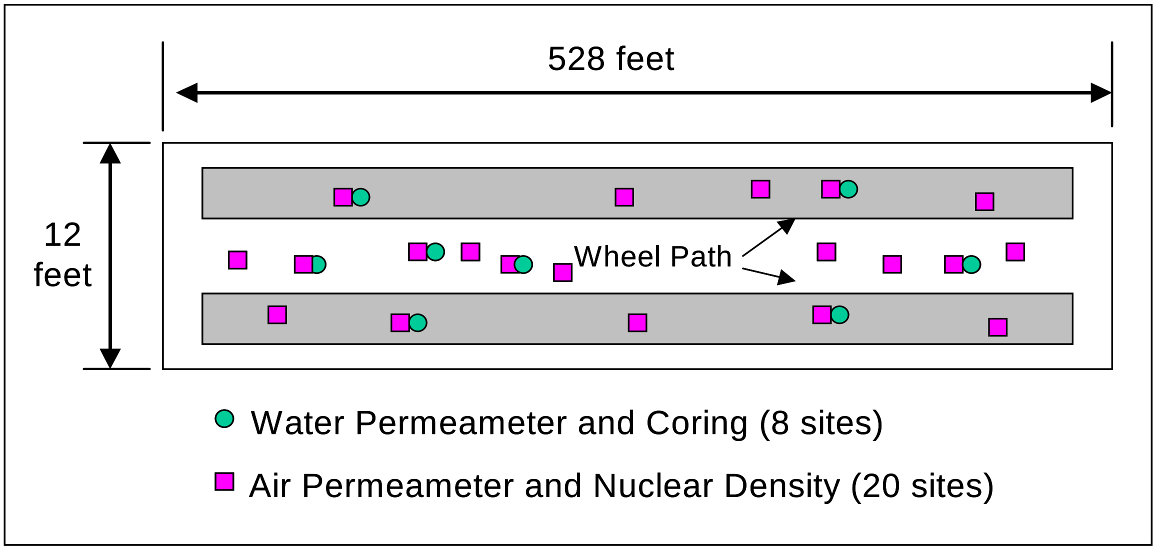

528 feet
12 feet
Wheel Path
● Water Permeameter and Coring (8 sites)
■ Air Permeameter and Nuclear Density (20 sites)

Figure 1. Test site schematic

During field testing on the first six projects in the study, minimal water permeability was observed, with values generally below $1x10^{-5}$ cm/sec. Water in the top standpipe of the permeameter dropped 1 cm or 2 cm in a period of two minutes, and in some cases, there was zero permeability. Density values were typically 95% or higher, and difference of density between wheel path and between path was about 0% to

3%. This type of data provided little resolution to clearly decipher if permeability had an effect on performance. Thus, it was decided to include some newer pavements built from 2000 to 2003 based on the following considerations:

- Less densification in and between wheel paths was used to understand density change from new construction to in-service period
- Less densified pavements would be more permeable
- If paved during implementation of Superpave, recommendations would be easier to implement (projects paved before 2000 used the Marshall mix design method)
- Projects from a previous WHRP permeability-density study (Russell et al. 2004) could be tested to understand the change in both permeability and density after traffic loading and aging, and aligned with the measured distresses.

Table 1 shows the final projects used for field evaluation, while Table 2 shows base type and surface thickness characteristics of the selected projects. The following section describes the field measurement procedures.

Table 1. Final projects used for field evaluation of permeability and density

Hwy	County	Termini	Functional Class	Surface Age (years)	Rut depth (in)	PDI	Warranty Project	
							Yes	No
11	ROCK	ORFORDVILLE-FOOTVILLE	10	10	0.08	14	√	
59	GREEN	ALBANY – STH 104	20	10	0.22	22		√
60	CRAWFORD	E JCT USH 61 – STH 80	20	7	0.09	8	√	
60	SAUK	CTH B - CTH O	20	7	0.21	10		√
14	RICHLAND	RICHLAND CENTER-EAST COUNTY LINE	10	5	0.09	9	√	
129	GRANT	LANCASTER BYPASS	10	5	0.28	18		√
131	MONROE	STH 71-TOMAH	20	5	0.08	15	√	
58	JUNEAU	CHAPEL RD-STEINER RD	20	5	0.10	50		√
64	CHIPPEWA	STH 124 – CTH E	20	5	0.10	8	√	
12	EAU CLAIRE	AUGUSTA (W JCT CTH G - ECPL)	20	5	0.14	38		√
13	DOUGLAS	TOWN RD – CTH F	20	5	0.11	8		√
77	WASHBURN	SOO LINE RR – N JCT STH 27	20	5	0.21	18		√
23	MARQUETTE	MONTELLO-PRINCETON	10	3	0.04	11		√
96	WAUPACA	STH 110 – TEWS DR.	20	3	0.04	25		√
22	WAUPACA	MANAWA-USH 45	20	3	0.07	4		√
96	BROWN	STH 32/57 – CTH G	20	5	0.06	26		√
8	ONEIDA	W. CO. LINE – STH 47	10	2	0.06	11		√
47	VILAS	CTH D – LAC DU FLAMBEAU	20	2	0.10	29		√
32	FOREST	S. CO. LINE – WABENO	20	2	0.07	4	√	
70	FOREST	W. CO. LINE – STH 55	20	2	0.13	4		√

Table 2. Base and thickness characteristics of selected projects

Project	Hwy	County	Surface Thick., in.	Base Type
1	59	GREEN	4.5	Old AC
2	11	ROCK	4.5	Old AC
3	60	SAUK	4.50	Old AC
4	14	RICHLAND	3	Rubblize
5	60	CRAWFORD	3.00	Pulverize
6	129	GRANT	4.5	Pulverize
7	58	JUNEAU	1.5	Old AC
8	131	MONROE	5	DGBC
9	12	EAUCLAIRE	4.5	Full Depth Mill
10	64	CHIPPEWA	4.5	Pulverize
11	77	WASHBURN	4.5	Old AC
12	13	DOUGLAS	4	Pulverize
13	23	MARQUETTE	4.25	Existing AC
14	96	WAUPACA	4	Old AC
15	22	WAUPACA	4	Pulverize
16	96	BROWN	3.5	Old AC
17	47	VILAS	3.5	Pulverize
18	8	ONEIDA	6	CABC
19	32	FOREST	4.5	Pulverize
20	70	FOREST	3.5	Old AC

Nuclear and Core Density Testing

The nuclear density gauge (CPN Model MC-3, Serial #M391105379) was used for density measurements at each of the randomly selected 20 locations within the 0.1-mile segment. Initially, eight cores were sampled on the first set of projects; however, the abrasive and highly-densified in-service pavements caused significant wear to the core bits and unexpected costs, and thus the number of cores was reduced to two each on the remaining projects. The reason for this smaller number was to ensure the layer thickness for permeability calculations, and to provide an offset value to adjust the nuclear density readings. WisDOT Method 1559 (modified AASHTO T-166) was used to determine bulk density of core samples.

Water Permeability Testing

The National Center for Asphalt Technology (NCAT) water permeameter was used for permeability testing. It was centered within the rectangular base used for nuclear density testing, sealant was applied to a rubber gasket between the pavement and permeameter base, two 50 lb weights were added to prevent uplift force from the water head, then the pavement was saturated. Several trials were conducted at each test site for repeatability information and to incorporate testing variability into the analysis.

Prior to field testing, there was a concern about the seal between the water permeameter and in-service pavement, particularly on rutted or rough-textured surfaces, so an evaluation of rubber gaskets was conducted at the Reichel-Korfmann Company in Milwaukee. These gaskets were able to successfully seal the water permeameter throughout field testing.

Air Permeability Testing

Air permeability testing was conducted using the ROMUS device at 20 test locations on each project, where 8 of 20 locations were comparative sites with the NCAT water permeameter. Air permeability testing was conducted after water permeability testing, with test locations offset 6 to 12 inches longitudinally to avoid the wet pavement surface.

To initiate testing, the bottom of the ROMUS device was first sealed to the pavement surface by way of a grease seal. The sealant grease was manually pumped through the device into a recessed base ring which was sized to replicate the opening of the NCAT water permeameter and designed to eliminate problems observed with the various sealing techniques used for the NCAT device.

Once the device was sealed to the pavement surface, pressing of the start button initiated a fully automated system that first creates a vacuum within the internal pressure chamber. For this research project, the ROMUS device was programmed to record four timing increments, each representing a change in vacuum pressure equivalent to four inches of water. This set-up simulates a falling head water permeability test with head drops from 24 to 20 inches, 20 to 16 inches, 16 to 12 inches, and 12 to 8 inches. Once the test is complete, the four timing increments are displayed on a digital display for manual recordation.

FRAMEWORK DEVELOPMENT

A conceptual framework for developing design criteria for permeability and density based on in-service pavement performance was developed. The framework was based on the following premises:

- Mix design properties dictate to some extent the final as-built density achieved at time of construction.
- The as-built density dictates the future performance of the pavement, i.e., higher as-built densities result in better pavement performance as measured by the PDI/year.
- Higher as-built densities result in less permeable pavements.
- Less permeable pavements exhibit better overall performance as measured by the PDI/year.

The framework is depicted in Figure 2. It involves the initial selection of a design or target performance (as measured by the PDI/year) to determine the expected design permeability, K_1. The permeability, K_1, is in turn used to specify the as-built density at construction, ρ_{01} to achieve K_1. Similarly, the design or target PDI/year is used to determine the as-built density, ρ_{02} to achieve the target value. The maximum of the two values is chosen as the controlling density to yield the desired PDI/year and corresponding design permeability. The controlling density is further used to select the critical mix design property as represented, for example, by the voids filled with binder (VFB).

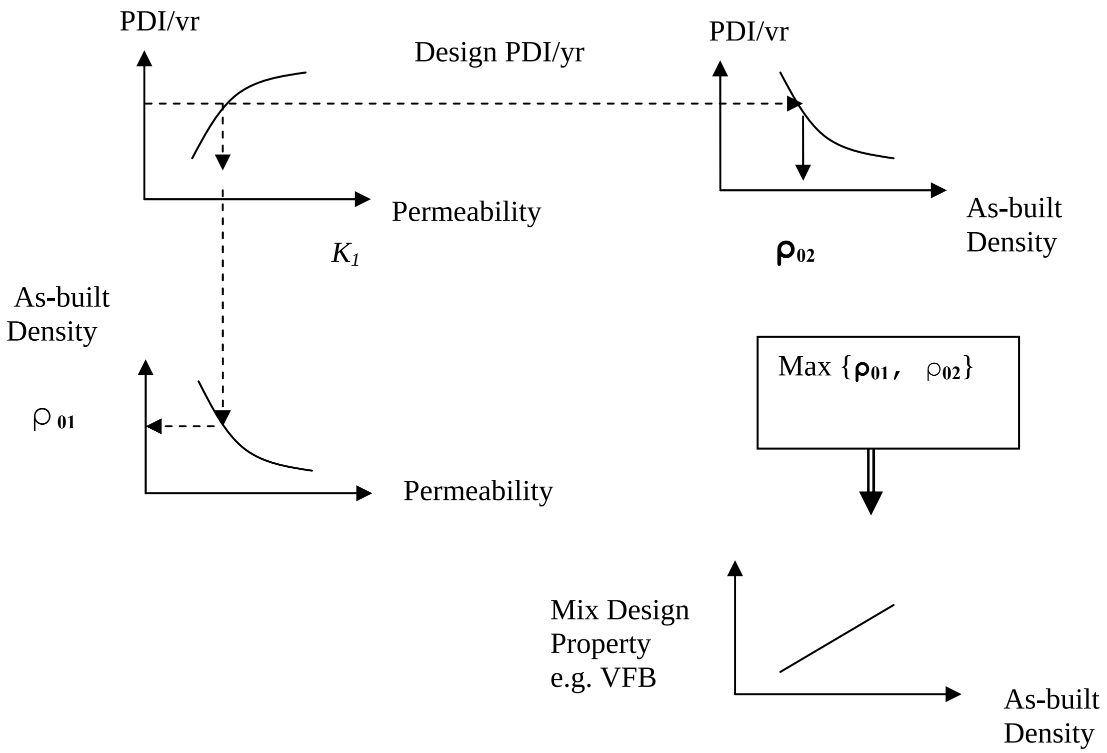


Figure 2. Framework for relating density, permeability and mix design properties to performance

The four premises that defined the framework were examined using data on 9 of the 20 projects selected for this study. The reason was that only those nine projects had records on the as-built densities. The nine projects had an average age of 4.8 years. The premises are examined in the following sections.

Mix Design Properties and As-built Density Relationships

A preliminary analysis involving data plots and simple regression was conducted to examine key mix design variables that influence as-built density. Two key variables including the design truck traffic (a surrogate for HMA mix type) and the VFB were identified to influence the as-built density. The relationships between truck traffic and as-built density, as well as VFB and as-built density are shown, respectively, in Figures 3 and 4. Figure 3 suggests that as-built density achieved appears to decrease non-linearly with increased truck traffic level. This observation may be due to the fact that higher traffic levels generate thicker pavements in design and may be difficult to compact compared to thinner pavements, which are associated with lower traffic levels. The model representing the relationship shown in Figure 3 is represented as Model #1 in Table 3. Figure 4 shows that higher VFB results in higher as-built density. The relationship is depicted as Model #2 in Table 3.

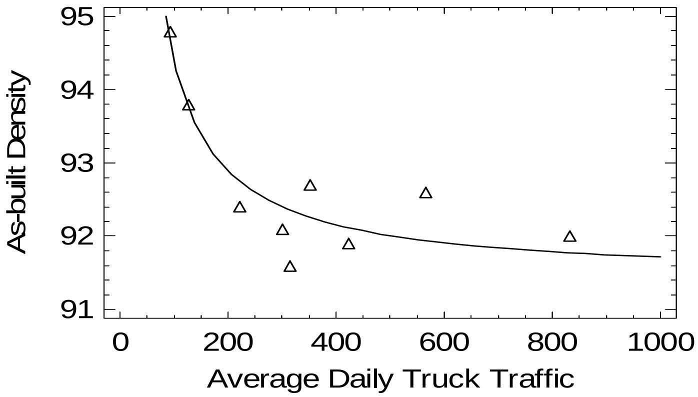


Figure 3. As-built construction density versus truck traffic

Table 3. Model characteristics

Model #	Model Form	Standard Error of Est.	F-Ratio	P-Value	R-squared
1	r_0=91.4199 + 293.524/ADTT	0.470482	30.96	0.0008	0.816
2	r_0=62.2908 + 0.408718*VFB	0.763187	7.83	0.0312	0.566
3	PDI/year_= 1/(-33.0673 + 0.36269* r_0)	0.275334	14.58	0.0066	0.675
4	K_w =1/(-252.41 + 2.78521* r_0)	2.81666	8.14	0.0290	0.576
5	PDI/year = -0.475181 + 4.63626* $(K_w)^{0.5}$	1.67685	6.51	0.0434	0.520

r_0 = As-built density
ADTT = Average Daily Truck Traffic
VFB = Voids filled with bitumen
K_w =Water permeability

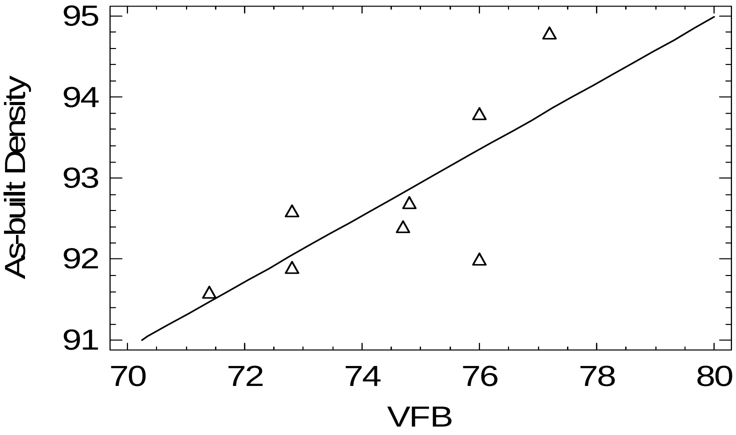


Figure 4. As-built density and VFB relationship

Performance and As-built Density Relationship

The relationship between as-built density and performance (indicated by the PDI/year) is shown in Figure 5 and represented as Model #3 in Table 3. Figure 5 suggests that pavements with high as-built densities tend to exhibit better performance.

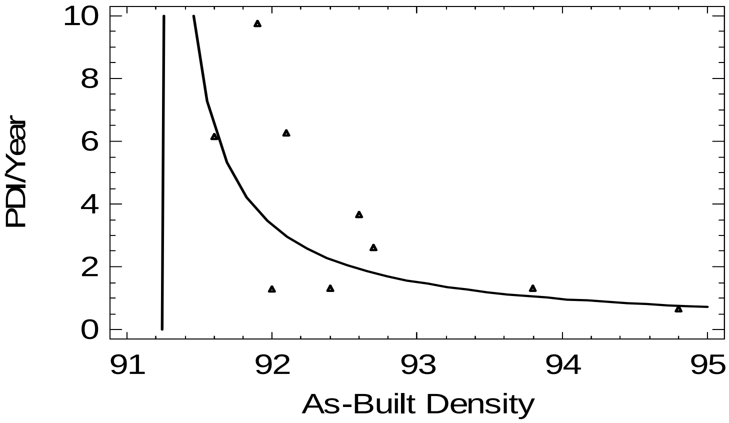


Figure 5. Pavement performance and as-built density relationship

Permeability and As-built Density Relationship

Figure 6 shows the relationship between as-built density and water permeability. This relationship is represented in Table 3 as Model #4 and supports the premise that higher as-built densities result in less permeable pavements.

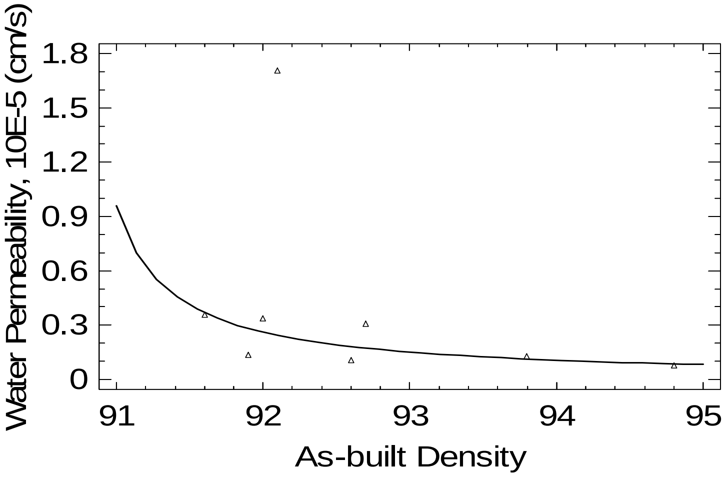


Figure 6. Water permeability and as-built density relationship

Performance and Permeability Relationship

Figure 7 shows the relationship between performance and water permeability. This relationship is represented as Model #5 in Table 3. Figure 7 suggests that pavement deterioration increases in a non-linear fashion with increasing water permeability.

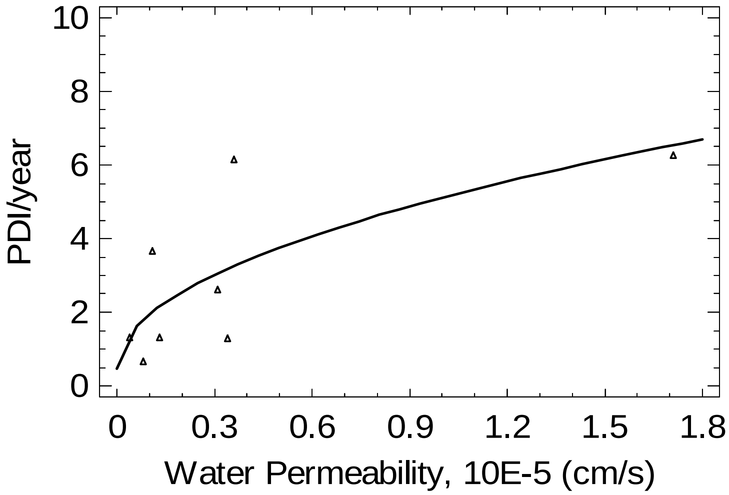


Figure 7. Pavement performance and water permeability relationship

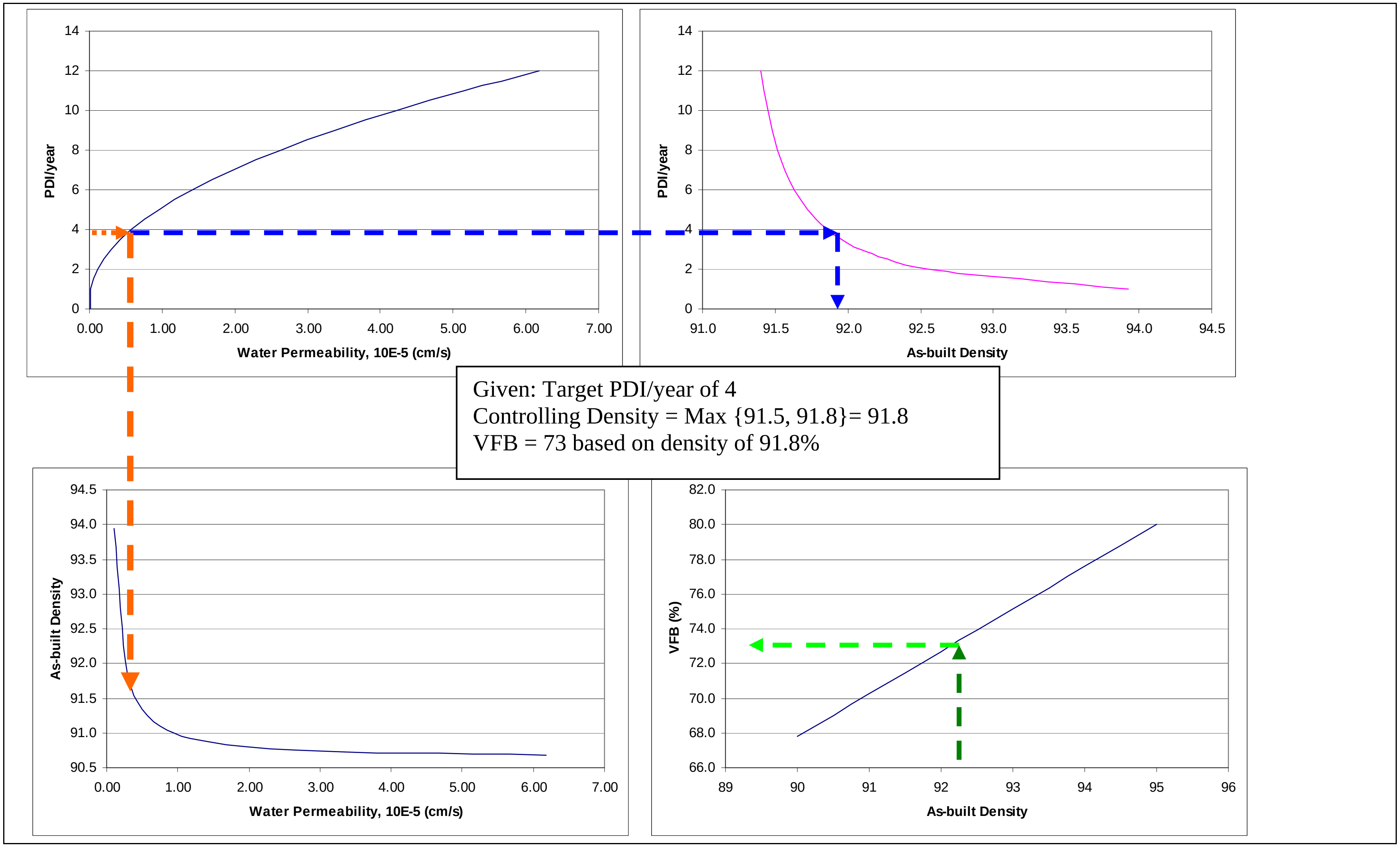
FRAMEWORK APPLICATION AND CRITERIA

The application of the framework described in the previous subsections and summarized in the models presented in Table 3 is illustrated in Figure 8. In this illustration, a desired PDI/year of 4.0 is assumed as the target PDI/year for preventive maintenance intervention. A different target value could be selected if the agency focus is not on preventive maintenance but on other intervention strategy, e.g., rehabilitation. Using Figure 8, an as-built density of 91.5% is triggered for the target PDI/year of 4.0 with a corresponding expected water permeability of 0.60 x 10^{-5} cm/sec. A density of 91.8% is also required to achieve the desired PDI/year of 4.0. Hence, the controlling density to satisfy permeability and PDI/year requirements is the latter value (91.8%), which is the greater of the two. The corresponding VFB based on the density is approximately 73%.

SUMMARY AND CONCLUSIONS

This paper presented a framework for developing design criteria for permeability and density based on in-service performance of HMA pavements. Within the framework, a number of statistical relationships were developed to facilitate the application of the framework. These relationships included PDI/year (i.e., the performance indicator) versus permeability, PDI/year versus as-built density, permeability versus as-built density, and as-built density versus mix parameters, e.g., VFB. The model was applied by selecting a target or design PDI/year value for preventive maintenance and using it to determine the expected permeability. The expected permeability was in turn used to determine a density value to yield that permeability. The PDI/year value was again used to determine another density value to produce that desired PDI/year. The maximum of the two density values was considered the controlling density required to produce the desired in-service pavement performance with corresponding design permeability. In addition, the controlling density was used to show how mix properties (e.g., VFB, as illustrated in this paper) could be back-estimated. The back-estimation provided guidance, for example, on the VFB design target in a mix to produce the desired in-service pavement performance.

The main weakness of the developed framework was the small number of samples used in developing the various statistical relationships. To develop models that are robust to unique project conditions, and capable of being broadly stated in construction or design specifications, it was recommended that a larger data set be assembled and models be developed to establish specific thresholds, particularly, target PDI/year values to yield density, permeability, and mixture components during Phase II of this study.



Given: Target PDI/year of 4
Controlling Density = Max {91.5, 91.8}= 91.8
VFB = 73 based on density of 91.8%

Figure 8. Framework application model for relating permeability and mix design properties to performance

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Developing Low-Power Systems for Automated Traffic Monitoring

Amit Pande
Department of Electrical and Computer Engineering
Iowa State University
2215 Coover Hall
Ames, IA 50011
amit@iastate.edu

Abhisek Mudgal
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
abhisek@iastate.edu

ABSTRACT

Video surveillance plays a key role in traffic safety. Surveillance systems are deployed at traffic lights, toll booths, and bridges for vigilance and detection of violations of traffic rules. They are also employed for measurements such as traffic volume. Traditional cameras have high power consumption, although *violations* are only recorded for a small fraction of time. We propose novel techniques to use reconfigurable devices that can dynamically adapt their power requirements, video compression performance, and visual quality based on real-time traffic. Such devices will run in low-power mode during normal traffic but would automatically switch to high-power mode during specific tasks. Recent research in embedded systems has addressed the design of such power-aware hardware that can be adapted for efficient traffic management. Traffic surveillance systems are integrated with lasers or radar technology for identifying speeding vehicles. Upon detection of such violation, the radar or laser signal triggers the camera to record the vehicle's license plate, and thus, a citation is issued. Such techniques can be extended to red-light cameras, tollbooth cameras, and other domains. We evaluated the proposed framework (low-power consumption systems) through prototype experiments and simulations using MATLAB, Xilinx ISE, and ModelSim.

Key words: low-power systems—simulation—traffic safety—video surveillance

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Survey-Based Approach to Identify Highway Safety Characteristics and Issues of Older Drivers

Loshaka Perera
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66502
loshaka@ksu.edu

Sunanda Dissanayake
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66502
sunanda@ksu.edu

ABSTRACT

In the past, many research studies have analyzed crash data to identify various factors contributing to older driver over-involvement in crashes. However, it is also necessary to identify other types of safety-related information which cannot be extracted from crash data, such as exposure to different weather and road conditions, difficulties associated with vehicle maneuvering, modifications made to driving patterns, etc., which might be playing a significant role in older-driver safety. Therefore, a questionnaire was prepared and a survey was conducted to identify those issues and difficulties highlighted in crash data. Based on the responses, a detailed examination was done to understand different behavioral changes in older drivers with respect to various conditions.

Based on the survey results, it was found that most of the older drivers have more than fifty years of driving experience and their seat belt usage was also found to be high. Left turns appear to be the most challenging maneuvering task. Analysis based on age revealed that level of difficulty associated with older drivers increases with age, and similarly, preference to avoiding demanding conditions, such as snowy weather, nighttime driving, and use of freeways also increases with aging. Drivers older than 70 years were highly involved in crashes and those with elevated income levels and education had higher involvement in crashes. Older male drivers indicated higher levels of difficulties in stopping, stopped waiting to turn, or slowing down situations. On the other hand, females showed higher levels of difficulty associated with identifying speeds and distance of oncoming traffic compared to males. Findings of this study could be used to develop more focused programs towards improving older driver safety.

Key words: older drivers—safety—survey

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INTRODUCTION

In the United States, the most common method of travel for elderly people is driving an automobile (Lyman, McGwin, and Sims 2001), especially in places where public transportation is not well-established. Personal mobility plays an important role in day-to-day activities and social functions for people irrespective of their age (Li, Braver, and Chen 2003). For the elderly, the ability to drive is a significant predictor indicating quality of life, functional independence, and physical and mental health conditions (Li, Braver, and Chen 2003). However, with time, they experience a decline in visual, physical, and cognitive functions, which are directly associated with ability to operate an automobile safely (McGwin, Chapman, and Owsley 2000). Conversely, older drivers appear to make modifications to their driving behavior and patterns over time in order to compensate for physical and cognitive changes associated with aging (D'Ambrosio, Coughlin, Mohyde, Gilbert, and Reimer 2007). However, older drivers still have one of the highest automobile crash rates per mile traveled as compared with other age groups (Lyman, McGwin, and Sims 2001; McGwin and Brown 1999).

Older-driver–involved (age 65 and above) crashes were analyzed and different characteristics were identified in prior studies. But it is not advisable to arrive at conclusions about older drivers solely depending on crash data, since those characteristics are linked only with a special segment of older drivers who were involved in crashes. In other words, many older drivers haven't been involved in crashes during the last few years and their highway safety experience is completely unobserved in such analysis. However, their characteristics should also be taken into consideration to make fair conclusions about older driver safety. After considering all factors, it was necessary to seek other sorts of information which could not be extracted from crash data, such as exposure to different weather and road conditions, modifications made to driving patterns, and so forth. As a result, a questionnaire was prepared and a survey was administrated addressing those issues and difficulties highlighted in crash data for a more detailed examination, and to understand different behavioral changes in older drivers with respect to driving under various circumstances. The survey form consisted of five main areas: general, demographic, exposure related, challenging situation, and difficulty level. The objective of this survey was to obtain information from older drivers irrespective of their involvement in crashes in order to get a general idea about the behavior, exposure, and different types of difficulties associated with them.

METHODOLOGY

Odds Ratio

Logistic regression was used to calculate odds ratios (OR) and 95% confidence intervals (CI) to assess the strength of the association between independent variables and dependent variables. The dependent variable considered here has two possible outcomes, 0 and 1, corresponding to "yes" if the event occurred and "no" if the event did not occurred. Therefore, binary logistic regression is considered in this analysis. The odds in favor of an event occurring is defined as the probability that the event will occur divided by the probability that the event will not occur. In logistic regression, the event of interest is always y =1. Given a particular set of values for the independent variables, the odds in favor of y =1 can be calculated as follows (Anderson, Sweeney, and Williams 2005):

$$Odds = \frac{P\left(y = 1 \middle| x_1, x_2, ..., x_p\right)}{P\left(y = 0 \middle| x_1, x_2, ..., x_p\right)}, \tag{1}$$

$P(y = 0 \mid x_1, x_2, ..., x_n)$ where $P(y = 1 \mid x_1, x_2, ..., x_n)$ = probability of event occurring and = probability of event not occurring.

The odds ratio measures the impact on the odds of a one-unit increase in only one of the independent variables. The odds ratio looks at the odds that y =1 given that one of the independent variables is increased by one unit (odds1), divided by the odds that y =1 given no change in the value of the independent variables (odds0).

$$odds\ ratio = \frac{odds_1}{odds_0} \tag{2}$$

This statistical method was used to analyze survey data mainly in relation to respondents who mentioned that they met with crashes during the last 10 years. Odds ratios and relevant confidence intervals at 95% were calculated for various conditions and are presented under results and discussion.

Questions were selected from demographic, general, exposure, and difficult sections where there could be a possibility of a relationship in connection with crash involvement. Even though answers for the difficulty level and exposure-related questions were in ordinal format, it can be considered that either respondents had no difficulty/exposure or had difficulty/exposure in some degree and therefore were re-classified as a binary (“yes” or “no”) variable. In the marital status situation, it was considered as married vs. single (including divorced, separated, and widowed). For questions with ordinal responses, the first option was selected as the reference group and odds were calculated for others relative to the first.

RESULTS AND DISCUSSION

As the first step, simple percentages were calculated for every question to get an idea about the overall situation. Frequencies and percentages for general questions are shown in Table 1. When looking at the simple percentages, 97% of respondents were currently driving and 92% had more than 50 years of driving experience. Forty-one percent of older drivers drove every day, whereas a majority of the others drove at least two or three days a week. A majority drove less than 500 miles per month, but 2% drove more than 2,000 miles per month. Out of the 284 respondents, 51 had been involved in crashes during the last 10 years. A majority of the older drivers hadn’t been involved in any traffic violations after turning 65 years, whereas 12% had received tickets for speeding.

Frequencies and relevant percentages pertaining to demographic, socio-economic, and educational background-related questions are presented in Table 2. When looking at the distribution of the sample based on age, a fair distribution can be seen among all age group categories included in the survey form. Thirty-five percent of the respondents had participated in driver education courses after turning 65 years of age. Almost all respondents had at least attended high school, and only 2% hadn’t had any formal schooling. Most of the respondents would stop driving either when their doctor advises or when their vision gets poor.

Table 1. Responses to general survey questions by older drivers in Kansas

Question	Freq.	%	Question	Freq.	%
Do you currently drive			***Miles driven per month***		
Yes	275	97%	0 -100 miles	115	40%
No	9	3%	101 -200 miles	53	19%
Driving experience			201 -500 miles	67	24%
0 -10 years	0	0%	501 -1000 miles	27	10%
11-20 years	1	0%	1001- 2000 miles	12	4%
21-30 years	0	0%	More than 2000 miles	6	2%
31-40 years	6	2%	***Seat belt usage over the years***		
41-50 years	15	5%	Increased	144	51%
More than 50 years	260	92%	Decreased	2	1%
Vehicle type			Almost the same	126	44%
Car	222	78%	Don't know	7	2%
SUV	19	7%	***Involved in a crash (last 10yrs)***		
Van	37	13%	Yes	51	18%
Pick up Truck	21	7%	No	229	81%
Other	7	2%	***Traffic violation(s) receive after 65 yrs***		
Vehicle age			Never received	205	72%
0 -5 years	103	36%	Speeding	33	12%
6- 10 years	110	39%	Parking	6	2%
11 -15 years	51	18%	DUI	0	0%
16-20 years	19	7%	Reckless driving	1	0%
21-25 years	4	1%	Expired tags/ license	6	2%
More than 25 years	1	0%	Vehicle deficiencies	0	0%
Driving frequency			Other (specify)…………………	11	4%
Everyday	116	41%	***When you would stop driving***		
4-6 days per week	58	20%	When my doctor advises	146	51%
2-3 days per week	76	27%	When my adult children interfere	43	15%
Once a week	19	7%	When my vision gets poor	136	48%
Once a month	2	1%	When my spouse advises	28	10%
Once in a while	11	4%	None of the above	22	8%

Note: Freq. represents frequency

Table 3 shows exposure-related frequencies and percentages. When looking at seat belt usage among older drivers, it can be noted that 85%responded that they always wear seat belts while driving and 80% do the same as a passenger. In addition, 51% believe their seat belt usage has gone up over past years, while 44% said it is almost the same. According to a past study, seat belt usage among older occupants hospitalized as a result of highway crashes was found to be 61% in Kansas (Ratnayake and Dissanayake 2007), which is well below the usage rates mentioned by respondents in the survey. In general, past studies have found that among belted drivers, an older driver was nearly seven times more likely to be killed or hospitalized than a younger driver (Cook, Knight, Olson, Nechodom, and Dean 2000).

Unlike quantitative-type questions, qualitative questions are more difficult to be evaluated. Thus, a common methodology which has been extensively used in the past was used here to evaluate the answers. This method assigned different weights to each answer; selected weights in this case range from 0 to 100. Following that, an average weighted value was calculated for each question, which represented the standpoint of respondents in a quantitative manner. Further, this number will describe the likelihood of occurrence as a probability. Calculated values for each question are presented in the last columns of Table 3 and Table 4, headed as "Likelihood of Occurrence." For example, likelihood of occurrence indicates the chance of a randomly selected person being in compliance with a particular event.

Table 2. Responses to demographic, socio-economic, and educational background-related questions by older drivers in Kansas

Question	Freq.	%	Question	Freq.	%
Age group			***Educational qualification***		
65 - 70 years	42	15%	No formal schooling	5	2%
71- 75 years	48	17%	Some high school	66	23%
76 - 80 years	66	23%	Some college	81	29%
81- 85 years	66	23%	Four year college	43	15%
More than 85 years	61	21%	Graduate degree	65	23%
Driver education courses after 65 yrs			Other (specify)……	15	5%
Yes	98	35%	***Annual household income***		
No	184	65%	Less than $ 9,999	12	4%
Gender			$ 10,000 - $ 14,999	24	8%
Male	114	40%	$ 15,000 - $ 19,999	30	11%
Female	170	60%	$ 20,000 - $ 29,999	59	21%
Marital status			$ 30,000 - $ 49,999	61	21%
Single	13	5%	$ 50,000 or above	52	18%
Married	132	46%	***Current residence status***		
Divorced	25	9%	Own house	122	43%
Widowed	108	38%	Rental	145	51%
Separated	6	2%			

Note: Freq. represents frequency and L.O. represents likelihood of occurrence

The assigned weights are as follows:

- Never—0
- Very rarely—25
- Sometimes—50
- Most of the time—75
- Always—100

Accordingly, 95% said they wear seat belts while driving and 93% wear them as a passenger. In other words, if an older driver was randomly selected, there was a 95% chance of that driver indicating that he/she wears a seat belt while driving. Similarly, if an older passenger was selected, there was a 93% chance of that particular passenger wearing a seat belt. Eighty-five percent of respondents stated they do not drive after consuming alcohol, but one percent responded that they always drink and drive. According to the survey, there was nearly a 64% chance of an older driver driving alone. This is a critical situation when they meet with accidents, because most of the time there is no one to call for help and this situation could be worse in rural areas.

Table 4 presents response-to-difficulty-type survey questions and respective likelihood of occurrence values have been calculated. When looking at difficulty-type questions and relevant likelihood of occurrence values, it is evident that difficulty of identifying speeds and distance of oncoming traffic was more common among older drivers. Further, difficulty associated with merging, diverging, judging gaps, and lane changing also appeared to be common among them. When carefully looking at the higher-ranked difficulties, it can be observed that all these parameters are related to one particular reason—vision and cognitive function of human beings that change with increased age.

Table 3. Frequencies, percentages, and likelihood of occurrence based on exposure

Question	Freq.	%	L.O.	Question	Freq.	%	L.O.
Seat belt usage (driver)			95	***Driving in snowy conditions***			39
Never	1	1%		Never	45	16%	
Very rarely	2	1%		Very rarely	80	28%	
Sometimes	9	3%		Sometimes	102	36%	
Most of the time	29	10%		Most of the time	26	9%	
Always	240	85%		Always	14	5%	
Seat belt usage (passenger)			93	***Driving in windy conditions***			56
Never	2	1%		Never	5	2%	
Very rarely	1	1%		Very rarely	35	12%	
Sometimes	8	3%		Sometimes	141	50%	
Most of the time	39	16%		Most of the time	56	20%	
Always	200	80%		Always	29	10%	
Driving at night			38	***Make sudden stops***			18
Never	38	13%		Never	119	42%	
Very rarely	86	30%		Very rarely	127	45%	
Sometimes	133	47%		Sometimes	24	8%	
Most of the time	11	4%		Most of the time	3	1%	
Always	11	4%		Always	4	1%	
Street is not lit well enough			38	***Drive after taking medicine***			39
Never	44	15%		Never	77	27%	
Very rarely	73	26%		Very rarely	71	25%	
Sometimes	106	37%		Sometimes	63	22%	
Most of the time	29	10%		Most of the time	39	14%	
Always	6	2%		Always	31	11%	
Driving on freeways			39	***Drive after taking alcohol***			5
Never	41	14%		Never	242	85%	
Very rarely	74	26%		Very rarely	24	8%	
Sometimes	134	47%		Sometimes	11	4%	
Most of the time	29	10%		Most of the time	1	1%	
Always	4	1%		Always	2	1%	
Driving in rainy conditions			50	***Drive alone***			64
Never	13	5%		Never	4	1%	
Very rarely	55	19%		Very rarely	23	8%	
Sometimes	147	52%		Sometimes	92	32%	
Most of the time	42	15%		Most of the time	133	47%	
Always	22	8%		Always	29	10%	

Note: Freq. represents frequency and L.O. represents likelihood of occurrence

Table 5 presents frequencies and percentages for questions focused on challenging situations. There is evidence from prior research that some drivers modify or self-regulate their driving habits in certain driving situations like high-traffic roads (Ball, Owsley, Stalvey, Roenker, Sloane, and Graves 1998). According to the survey data, 50% of the respondents would like to avoid high-traffic roads when driving, whereas preference for local roads and urban minor roads are high among older drivers. Frequency of use of different types of roads avoided by older drivers is depicted in Figure 1. Roundabouts seemed to be the major type of intersection where older drivers are in obscurity. Left turns appeared to be the most challenging maneuvering situation for older drivers at intersections, especially where there were no signal lights or green arrows. However, almost all older drivers seemed to be confident about right turns and protected left turns. Similar results were found in prior research stating that older drivers were no more likely to be involved in right-turn–related crashes compared to younger drivers, but they were over represented twice as often as younger drivers in left-turn–related crashes (Cook, Knight, Olson, Nechodom, and Dean 2000).

Table 4. Responses to difficulty-type survey questions by older drivers in Kansas

Question	Freq.	%	L.O.	Question	Freq.	%	L.O.
Difficulty with stopping, stopped waiting to turn, or slowing down			14	***Difficulty with merging***			22
				Never	90	32%	
Never	150	54%		Very rarely	141	50%	
Very rarely	109	39%		Sometimes	48	17%	
Sometimes	16	6%		Most of the time	3	1%	
Most of the time	2	1%		Always	0	0%	
Always	2	1%		***Difficulty in judging gaps***			19
Difficulty with following the road straight			8	Never	105	38%	
				Very rarely	140	50%	
Never	203	72%		Sometimes	32	11%	
Very rarely	74	26%		Most of the time	3	1%	
Sometimes	4	1%		Always	0	0%	
Most of the time	0	0%		***Difficulty with diverging***			19
Always	1	1%		Never	100	36%	
Difficulty with identifying speeds and distance of oncoming traffic			24	Very rarely	148	53%	
				Sometimes	28	10%	
Not at all	87	31%		Most of the time	2	1%	
Very rarely	126	45%		Always	0	0%	
Sometimes	59	21%		***Difficulty with negotiating curves***			12
Most of the time	6	2%					
Always	1	1%		Never	165	58%	
Difficulty in lane changing			19	Very rarely	102	36%	
Never	107	38%		Sometimes	14	5%	
Very rarely	142	50%		Most of the time	1	1%	
Sometimes	31	11%		Always	1	1%	
Most of the time	2	1%					
Always	1	1%					

Note: Freq. represents frequency and L.O. represents likelihood of occurrence

Table 5. Responses to challenging situation survey questions by older drivers in Kansas

Question	Freq.	%	Question	Freq.	%
Difficulties at intersections			***Intersection with difficulties***		
Yes	21	7%	Stop light/ traffic lights	2	1%
No	255	90%	STOP sign controlled	3	1%
Challenging driving tasks			YIELD sign controlled	12	4%
L.T. with no signal lights	53	19%	Roundabouts	32	11%
L.T. without a green arrow	35	12%	No control	15	5%
L.T. with a green arrow	1	1%	***Roads you would like to avoid***		
L.T. at un-signalized intersections	44	15%	Freeways	77	27%
Making R.T.	1	1%	Urban major roads	43	15%
Yielding or Stopping	12	4%	Urban minor roads	16	6%
Passing through	3	1%	High traffic roads	141	50%
None of the above	178	63%	Two lane undivided highways	54	19%
			Rural roads	52	18%
			Local roads	6	2%
			None of the above	62	22%

Note: Freq. represents frequency

Differences Based on Gender

Table 6 shows the cross-relationships between the gender of older-driver respondents and different types of difficulties mentioned in the survey form. In the table, the likelihood percentage is also calculated and presented for each case. This cross-classification would help to identify high-difficulty levels associated with gender if present. To be more prudent, chi-square values were also calculated for each case.

Table 6. Gender vs. response to difficulty-type survey questions

Level of Difficulty	Difficulty with stopping, stopped waiting to turn, or slowing down		Difficulty with following the road straight		Difficulty in lane changing		Difficulty with merging	
	Male	**Female**	**Male**	**Female**	**Male**	**Female**	**Male**	**Female**
Never	50%	57%	75%	70%	38%	38%	34%	30%
Very rarely	39%	39%	21%	30%	52%	49%	54%	48%
Sometimes	9%	4%	3%	1%	9%	12%	11%	21%
Most of the time	1%	1%	0%	0%	1%	1%	2%	1%
Always	2%	0%	1%	0%	1%	0%	0%	0%
Weighted value	**17**	**12**	**7**	**8**	**19**	**19**	**20**	**23**

Level of Difficulty	Difficulty with identifying speeds and distance of oncoming traffic		Difficulty with diverging		Difficulty with negotiating curves		Difficulty in judging gaps	
	Male	**Female**	**Male**	**Female**	**Male**	**Female**	**Male**	**Female**
Never	45%	22%	33%	38%	63%	55%	43%	34%
Very rarely	38%	50%	55%	52%	33%	38%	46%	52%
Sometimes	17%	24%	12%	9%	4%	6%	10%	13%
Most of the time	0%	4%	0%	1%	0%	1%	1%	1%
Always	1%	0%	0%	0%	0%	1%	0%	0%
Weighted value	**19**	**27**	**20**	**18**	**10**	**13**	**17**	**20**

Note: Response rates are shown in percentages.

According to the percentage and likelihood, it can be mentioned that a higher level of difficulty was associated with males compared to females with respect to stopping, stopped waiting to turn, or slowing down situations. This relationship was proven by the chi-square test at a 95% confidence level (χ^2=5.922, p=0.0518). Similarly, with respect to diverging, males indicated a higher level of difficulty compared to females; on the other hand, females indicated a higher level of difficulty than males when negotiating curves. The relationship with diverging cannot be proven by a chi-square test (χ^2=0.605, p>0.5). With negotiating curves, there was a relationship at the 74% confidence level (χ^2=2.714, p=0.257). When merging and judging gaps to merge or turn, females showed higher levels of difficulty than males. But there was no evidence for a strong co-relationship between these two situations according to confidence level calculations. It was about 89% and 73% for these two cases, respectively (χ^2=4.352, p=0.1135 and χ^2=2.614, p=0.271). However, a significant difference can be observed with the difficulty associated with identifying speeds and distance of oncoming traffic conditions. In those situations, females showed a much higher level of difficulty compared to males, and it was statistically proven with the chi-square test at the 99.9% confidence level (χ^2=16.765, p<0.001). There was only a slight difference shown in the difficulty associated with following a straight road with respect to gender (χ^2=1.131, p>0.5), and with respect to lane changing, there was no difference shown at all (χ^2=0.447, p>0.5).

In Table 7, driving frequency and miles driven were tabulated based on gender. Accordingly, 20% more males drive every day than females and this is counterbalanced in other options. Furthermore, the percentage of females who drive once in a while is high, which supports the idea that older females drive less frequently compared to older males. In general, prior researchers have found that older drivers with functional impairment were more likely to drive less than four days per week, while older drivers with a history of cataracts or high blood pressure were more likely to report a low number of days driven per week (Lyman, McGwin, and Sims 2001). On average, more than 20% of females drive less than 100 miles per month compared to males, and this was nearly 12% in the next mileage category. When number of miles driven per month increases, the male driver percentage is higher than the female driver percentage.

Table 7. Gender vs. driving frequency and miles driven

Driving Frequency	Male	Female	Miles Driven	Male	Female
Everyday	53%	33%	0 -100 miles	28%	50%
4-6 days per week	18%	23%	101 -200 miles	12%	24%
2-3 days per week	25%	29%	201 -500 miles	27%	22%
Once a week	4%	9%	501 -1000 miles	19%	3%
Once a month	1%	1%	1001- 2000 miles	10%	1%
Once in a while	1%	6%	More than 2000 miles	4%	1%

A higher percentage of males were involved in crashes among respondents compared to females. However, prior research has found that older females have higher accident involvement rates than older males (Stamatiadis 1996). When a similar calculation was carried out based on the number of miles traveled, males showed a crash rate of 3.08 for million vehicle miles driven, whereas females showed a much higher crash rate of 7.83 for million vehicle miles driven. This illustrates a higher crash involvement risk with respect to females compared to males.

According to past studies in Kansas, seat belt usage among older crash victims was high compared to other age groups. But, irrespective of age, a majority of the crash victims were males and their seat belt usage was lower compared to females (Ratnayake and Dissanayake 2007). Similar results were found from the survey as well. Seat belt usage was high both as a driver and as a passenger (average likelihood of occurrence of 94.9 and 93.4, respectively, for drivers and passengers), and more male drivers wore their seat belts as compared to females (likelihood of occurrence of 97 vs. 91.9). However, fewer males wore seat belts as passengers as compared to females (likelihood of occurrence of 91.8 vs. 94.4).

Differences Based on Age

Similar to gender, it is important to identify different older driver behaviors associated with their age. When looking at the mileage driven based on age, it can be observed that in general, number of miles driven reduces as age increases ($\chi 2=47.714$, $p<0.001$). Further, there is a high co-relationship between driving frequency and age of the older driver ($\chi 2=29.190$, $p<0.001$). Considering the information revealed from these two situations, it is possible to state that older male drivers drive more frequently and more miles compared to older female drivers, confirming previous findings (Li, Braver, and Chen 2003).

Table 8 shows the percentages for likelihood of occurrence with respect to difficulty-level questions based on age. The percentages above the average are highlighted.

At a glance, it can be seen that age groups 76 to 80 years and 81 to 85 years have more difficulties than other age groups, or in other words, their difficulty levels are above the average. Further, it can be observed that the 81 to 85 years age group shows a higher probability of having difficulties compared to the 76 to 80 years age group in all cases. When considering the overall situation, numbers illustrate that likelihood of difficulty increases as age increases but have a slight decrease when it comes to the above-

85 years age group. Occasionally a few other age groups also indicate values above the average, but with no consistent pattern and thus can be disregarded as random variations.

Table 8. Age vs. response to difficulties-type survey questions

Age Group	Difficulty with stopping, stopped waiting to turn, or slowing down	Difficulty with following the road straight	Difficulty in lane changing	Difficulty with merging
65- 70 years	11.9	**9.5**	17.9	20.8
71- 75 years	11.7	6.3	15.1	18.2
76- 80 years	**14.0**	**8.0**	**20.8**	**21.9**
81- 85 years	**16.7**	**9.2**	**22.7**	**26.9**
Over 85 years	13.8	5.3	16.4	19.7
Average	*13.9*	*7.6*	*18.9*	*21.8*

Age Group	Difficulty with identifying speeds and distance of oncoming traffic	Difficulty with negotiating curves	Difficulty with diverging	Difficulty in judging gaps
65- 70 years	19.6	10.1	14.6	17.3
71- 75 years	20.2	12.0	18.2	**19.3**
76- 80 years	23.1	11.7	**19.8**	**19.6**
81- 85 years	**28.5**	**14.0**	**22.7**	**21.1**
Over 85 years	**25.4**	11.9	17.2	17.2
Average	*23.8*	*12.1*	*18.9*	*19.0*

Note: Values represent their likelihood of occurrence based on survey response.
Values greater than the average are bolded.

From past research studies, it is well-known that older drivers make modifications to their driving behavior over time in order to compensate for physical and cognitive changes associated with their aging. As a result, they either avoid driving in demanding situations or reduce the number of miles traveled under such conditions (D'Ambrosio, Coughlin, Mohyde, Gilbert, and Reimer 2007). According to the study, it was found that most of the drivers hesitated to drive in snowy weather conditions compared to windy and rainy weather conditions. Overall, it can be seen that as older drivers age, their willingness to drive under all three weather conditions decreases gradually. Preferences for driving at night and on the freeway also seemed to be as low as driving under snowy weather conditions. It can be noted that, more or less, willingness to drive at night and on the freeway also decreases with aging.

Miles driven by an older driver could be governed by various other factors such as income level, age, gender, etc. A chi-square test was carried out to identify the relationships statistically. For the income vs. miles driven, the calculated chi-square value (χ^2) was 23.010 and the tabular value at a 95% confidence level with 12 degrees of freedom was 21.026. Therefore, the calculated value was greater than the tabular value, and the relationship was statistically significant at the 5% level. Generally, higher individual income levels increase the number of miles driven (Traynor 2008). This is true with older drivers as well according to the survey data. Higher incomes increase time value for individuals and considering transportation, they wish to reduce travel time in various ways. They especially tend to drive at higher speeds and sometimes even try to follow less-safe driving actions, which can increase fatal risks (Traynor 2008). However, applicability of this situation to the older driver segment is questionable, since their time value is not that high compared to other age groups, and therefore, further investigation is needed before arriving at conclusions. Increased demand for transportation increases exposure to crashes (Traynor 2008), and according to the survey, average number of miles driven per month has gone up with increased household income levels. As mentioned before, for age vs. miles driven, the calculated χ^2 was 47.714 and the tabular value for 12 degrees of freedom was 21.026. This shows a correlation between age and miles driven as well. Similarly, gender and miles driven also showed a very high correlation (χ^2=50.147, DF=4, p<0.001), indicating a relationship between gender and miles driven by older drivers. When looking at seat belt-usage distribution with respect to different older driver age groups, seat belt usage was below the average level in age groups from 65 to 70 and 71 to 75, for drivers as well as passengers. This clearly indicates that seat belt usage increases as drivers age.

It was a commonly addressed issue in past studies that decisions about limiting or stopping driving was one of the most difficult tasks faced by older drivers. Therefore, a question was included in the survey form inquiring, "When do you think you would stop driving?" For this question, 270 older drivers responded and 14 who were asked did not. Since multiple answers were accepted for this question, the total number of responses was greater than 270. Accordingly, the majority would like to stop driving either when their doctors advise it or when their vision gets poor. When looking at the classification based on gender, females were more willing to listen to their doctors and adult children compared to males. Furthermore, female drivers would prefer to stop driving when their vision gets poor compared to older male drivers. On the other hand, more male drivers were willing to hear about the decision to stop driving from their spouses compared to female drivers. There have been several studies carried out in the past related to aging and the decision when to stop driving. D'Ambrosio et al. (D'Ambrosio, Coughlin, Mohyde, Gilbert, and Reimer 2007) had said that older drivers' decisions to stop driving were more influenced by their spouses if married and living with their spouse. Secondly, they would like to listen mostly to their doctors and adult children. But the results were slightly different in this study based on the survey conducted. Even though most of the respondents were married, they would still like to hear about the decision to stop driving from their doctors rather than from their spouses.

According to survey results, high-traffic roads, freeways, and two-lane undivided highways were among the less-preferred roads by older drivers. Their likelihood of avoidance of these roads increases as they get older, but there was a slight decrease indicated when drivers reached the age of 85 years.

In general, alcohol consumption by drivers increases with higher income levels (Traynor 2008). However, this issue was not truly visible among older drivers according to the survey data. Based on the percentage distribution, drivers driving after consuming alcohol remained almost the same for all income levels, showing no bias toward high-income earners.

From prior studies, it was found that older drivers with a history of at-fault crashes in the past five years reported more avoidance to challenging conditions than those who had crash-free records (Ball, Owsley, Stalvey, Roenker, Sloane, and Graves 1998). There was no such difference found in the survey data, but it is important to note that number of years considered in this survey was ten years, instead of the five years covered in the Ball et al. study. Furthermore, no detailed comparison was done with crash-free respondents' exposure since such data was not acquired.

Crashes and Contributing Factors

Crude odds ratios were calculated and presented in Table 9 for some selected variables. Odds values are based on respondents who had met with crashes during the last 10 years, and the word "respondents" will refer to the same definition hereafter in this discussion. Nighttime driving among respondents was 10% higher compared to others who don't drive at night and conversely driving on freeways was 12% less compared to respondents who don't frequently drive on freeways. When looking at different exposure conditions, exposure was high in rainy and snowy weather conditions, but less in windy weather conditions. This implies that more exposure to rainy and snowy weather conditions increases the chances of older drivers being involved in crashes. For all difficulty-type questions, respondents' representation was higher except for the stopping-related situation and straight following the road situation. It should be noted that the margins were less than five%, and therefore, it was not advisable to disregard it completely. Though most of the values were marginally higher, respondents showed 43% higher levels of difficulty with respect to merging and 96%higher levels of difficulty with diverging. Moreover, difficulties associated with identifying speeds and distance of oncoming traffic showed 2.4 times (140%) higher difficulty levels compared to respondents who didn't experience such difficulties.

Table 9. Crude odds ratios (ORs) and 95% confidence intervals (CIs) for crash involvement

Variable	ORs	95% CI	Variable		ORs	95% CI
Driving at night	1.10	0.43, 2.81	*Age*	65 – 70 years	*Reference*	
Driving on freeways	0.88	0.38, 2.05		71 – 75 years	1.26	0.37, 4.33
Driving in rain	2.27	0.28, 18.14		76 – 80 years	1.35	0.43, 4.26
Driving in snow	1.17	0.49, 2.82		81 – 85 years	3.12	1.06, 9.17
Driving in wind	0.68	0.07, 6.70		> 85 years	1.45	0.46, 4.60
Make sudden stop	1.06	0.57, 1.97				
Drive after taking medicine	1.35	0.65, 2.81	*Income*	< $20,000	*Reference*	
Drive after taking alcohol	1.06	0.44, 2.57		$20,000 - $30,000	1.37	0.52, 3.60
Drive alone	-	-		$30,000 - $50,000	1.32	0.50, 3.45
Difficulty with stopping, stopped waiting or slowing down	0.97	0.52, 1.82		> $50,000	1.61	0.61, 4.25
Difficulty with following the road straight	0.95	0.48, 1.87	*Education*	High School	*Reference*	
				College	1.27	0.60, 2.67
Difficulty in lane changing	1.25	0.66, 2.37		Graduate	1.50	0.65, 3.46
Difficulty with merging	1.43	0.72, 2.85				
Difficulty in judging gaps	1.32	0.69, 2.53	*Miles driven*	0 – 100 miles	*Reference*	
Difficulty with diverging	1.96	0.97, 3.96		101 – 200 miles	0.95	0.41, 2.18
Difficulty with negotiating curves	1.05	0.57, 1.94		201 – 500 miles	0.89	0.41, 1.95
Difficulty with identifying speeds and distance of oncoming traffic	2.40	1.11, 5.19		501 – 1000 miles	0.93	0.32, 2.73
				> 1001 miles	0.51	0.11, 2.39
Gender	1.15	0.62, 2.13				
Driver education courses after 65 yrs	1.88	1.01, 3.47	*Driving freq.*	Everyday	*Reference*	
				4 - 6 days/ week	0.93	0.41, 2.14
Marital status	1.03	0.56, 1.89		2 -3 days/ week	1.10	0.53, 2.30
Current residence status	0.49	0.25, 0.94		Once in a while	0.90	0.31, 2.61

Some odds ratios were calculated based on a few demographic questions in order to see how they are related to driving behavior of older drivers. Respondents who took driving education courses showed higher likeliness to be involved in crashes compared to others who haven't participated in such courses. This presumably could be due to the fact that, consequently, older drivers take a driving course after being involved in a crash. When considering older-driver groups based on age, the 65 to 70 years age group was considered the reference group and odds ratios have revealed that other drivers older than the 65 to 70 years group are overly involved in crashes compared to the reference group. Furthermore, it is important to highlight that the age group from 81 to 85 years had 3.12 times higher involvement rate compared to reference group. A similar pattern can be observed with respect to income levels and in relation to education. Higher annual income earners were more likely to be involved in crashes and the same could be seen with higher levels of education, where chances of being involved in a crash also increased. As number of miles driven increased, chances of being involved in a crash have decreased according to the ratios. This was probably due to the increased number of miles per week increasing their experience. On the other hand, it may be due to the fact that people with more difficulties try to minimize driving (Lyman, McGwin and Sims 2001; McGwin, Chapman, and Owsley 2000) and at the same time have high chances of being involved in crashes. Driving frequency shows that respondents who drive two to three days per week have slightly higher involvement rates compared to respondents who drive every day.

CONCLUSIONS

From the initial percentage calculations, it can be seen that most older drivers who responded to the survey have more than 50 years of driving experience, drive cars that are not older, and drive at least two days per week. Seat belt usage was found to be high among both older drivers and passengers. Roundabouts seemed to be unpopular among older drivers and left turns appeared to be the most challenging maneuvering task for them, especially at unsignalized intersections. However, older drivers showed higher confidence in right-turn maneuvering as well as left-turn maneuvering at locations where signals with protected left turns were present. Avoidance of high-traffic roads was more common among older drivers and conversely, preference for local roads and urban minor roads was high.

When looking at differences based on gender, males were overrepresented in the difficulties of stopping, stopped waiting to turn, or slowing down situations. On the other hand, females showed higher levels of difficulty associated with identifying speeds and distance of oncoming traffic compared to males. The average number of miles driven by female older drivers was less compared to male older drivers, and females had a higher propensity for involvement in crashes as indicated by the survey responses.

Analysis based on age revealed that the level of difficulty associated with older drivers increased with age and similarly, preference to avoid demanding conditions such as snowy weather, nighttime driving, and use of freeways also increased with aging. Co-relationships were found for miles driven with income, age, and gender. Number of miles driven was higher as income increased but with increasing age, number of miles driven decreased. A majority of older drivers would like to stop driving either when their doctors advise or when their vision gets poor.

Based on the respondents who met with crashes during the last 10 years, some interesting facts were found. Their exposure to rainy and snowy weather conditions were high, and they reported higher difficulties especially in association with merging, diverging, and identifying speeds and distance of oncoming traffic. Furthermore, statistics showed that drivers older than 70 years were highly involved in crashes, and those with elevated income levels and education had higher involvement in crashes; however, when number of miles driven increased, chances of being involved in crashes decreased. In future, these results can be incorporated with crash data analysis in order to suggest countermeasures to improve older driver safety.

ACKNOWLEDGMENTS

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Development, Evaluation, and Implementation of a Precast Paving Notch System

Brent M. Phares
Bridge Engineering Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
bphares@iastate.edu

Mike LaViolette
HNTB Corporation
Central Park Plaza North
222 S. 15th Street, Ste. 247-N
Omaha, NE 68102
mlaviolette@hntb.com

Dean Bierwagen
Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
Dean.Bierwagen@dot.iowa.gov

ABSTRACT

Bridge approach pavement settlement and the resulting formation of “bumps” at the end of bridges is a recurring problem on a number of Iowa bridges. One of the contributing factors in this settlement is failure of the bridge paving notch. A paving notch (also known as a corbel or a paving support) consists of a horizontal shelf constructed on the rear of a bridge abutment and is used to support the adjacent roadway pavement. Over time, these paving notches have been observed to deteriorate/fail due to a number of conditions, including horizontal abutment movement due to seasonal temperature changes, loss of backfill materials by erosion, inadequate construction practices, foundation soil settlement, heavy traffic loads, salt brine that leaks through the expansion joint, and an open expansion joint that tends to fill with dirt and debris and “push” the approach pavement off the paving notch.

The conventional repair procedure for this problem typically consists of removing the deteriorated paving notch concrete while preserving as much of the existing reinforcing steel as possible, constructing wood forms, and placing a cast-in-place (CIP) concrete paving notch followed by replacing the approach slab pavement. The conventional replacement method, however, requires that the bridge be taken out of service for an extended period of time, which disrupts the traveling public. The notable number of bridges that exhibit the failing paving notch problem and, more importantly, their location on highly traveled roadways necessitate the development of a standardized, much more quickly installed replacement method. With a standardized system, situations where the deterioration is unknown until approach pavement removal could be addressed with minimal traffic disruptions.

The developed precast paving notch system was intended for use in either new construction or as rapid replacement that can be installed in single-lane widths to allow for staged construction under traffic with a single overnight bridge closure. The system consists of a rectangular precast concrete element that is connected to the rear of the abutment using high-strength threaded steel rods and an epoxy adhesive that is similar to that used in segmental bridge construction.

The Iowa State University (ISU) Bridge Engineering Center (BEC) performed full-scale laboratory testing of the proposed paving notch replacement system. The objective of the testing program was to

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verify the structural capacity of the proposed precast paving notch system and to investigate the feasibility of the proposed solution. Following testing, the design of the paving notch system was finalized and utilized at a site near Knoxville, IA.

This paper will summarize the laboratory testing, including results and protocols, and will describe the first field application of the system.

Key words: bridge—paving notch—rapid—replacement

Horizontal Curves—A New Method for Identifying At-Risk Locations for Safety Investment

Howard Preston
CH2M HILL, Inc.
1295 Northland Drive
Mendota Heights, MN 55120
hpreston@ch2m.com

ABSTRACT

Minnesota's Strategic Highway Safety Plan identified addressing single-vehicle road departure crashes as one of the state's safety emphasis areas based on the fact that these types of crashes account for 32% of statewide fatal crashes, 36% of fatal crashes in Greater Minnesota, and 47% of fatal crashes on local systems in Greater Minnesota. These data clearly indicate that the large number of fatal crashes on State and local systems in rural areas associated with road departure crashes represent a pool of crashes susceptible to correction. This new focus on road departure crashes along rural highways resulted in a significant new challenge. Given that there are 52,000 miles of rural two-lane highway (8,000 miles on the State system and 44,000 miles on local systems) that average less than 0.5 crashes per mile per year, how do you identify at-risk locations for safety investments?

Initial analytical efforts to provide greater insight resulted in three key conclusions:

1. The low crash densities combined with the lack of any frequency of "black spots" suggests that the traditional reactive method—looking for locations with higher than expected frequencies of crashes—could not be an effective tool to identify at-risk locations along rural highways.
2. A new method for identifying at-risk locations using crash surrogates should be developed.
3. A specific part of the rural highway system was identified as being overrepresented relative to the occurrence of severe (fatal + "A" injury) road departure crashes—horizontal curves. An initial review of almost 1,500 curves on the local highway system in five counties and almost 300 curves on the State system in one Minnesota Department of Transportation District found that between 25% and 50% of the severe road departure crashes occurred in curves, even though curves only accounted for around 10% of the system mileage.

This last conclusion then lead directly to additional questions specifically related to horizontal curves. Are all curves equally at risk, and if not, can the characteristics be identified of the sub-set of curves that are most at-risk? A series of research projects found that all curves are NOT equally at-risk. Crash rates in curves were found to increase as the radius decreased below 2,000 feet, and approximately 90% of fatal crashes and 75% of injury crashes occurred in curves with radii less than 1,500 feet. Finally, the characteristics of curves with crashes were identified and these characteristics (radius, volume, presence of an intersection, visual trap, and proximity to other high priority curves) were incorporated into a methodology that can be used to identify and prioritize curves across a system for safety investment.

Key words: horizontal curves—Minnesota Strategic Highway Safety Plan—road departure crashes

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Integration Safety Assessment and Web-Based Visual Analytics

Xiao Qin
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
B239 Engineering Hall
1415 Engineering Drive
Madison, WI 53706-1691
xqin@engr.wisc.edu

Shuguang Hao
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
4161 Mechanical Engineering Building
1513 University Ave.
Madison, WI 53706-1691
hao2@wisc.edu

ABSTRACT

The keys to a successful safety program are to identify the locations exhibiting an abnormally high number of crashes with integrated information of highway safety and to disseminate the results to safety stakeholders. This paper reports the development and user interface of a web-based application for highway crash analysis and safety decision support.

Using web 2.0 technologies, this state-of-the-art application not only provides a comprehensive collection of query functionalities, which can help the users search for crashes using a variety of crash attributes, but also enables users to explore interactively between crashes and roadway network in the spatial context. The query results can be summarized online by injury severity. In addition, to aid the local agencies and practitioners to efficiently locate hazardous intersections in relation to other highway attributes, different layers of information available in Wisconsin Information Systems for Local Roads (WISLR) can be presented simultaneously with the crash information. The synthesis of crash information and highway inventory as well as traffic condition empowers the users to review crashes and recognize crash patterns from various perspectives. This application is developed based on the framework of ArcGIS Server, one of the most sophisticated geoprocessing platforms; yet, it is also very user-friendly to GIS application novices. The application through a combination of safety assessment with visual analysis is accessible through the Internet, thus providing maximum accessibility to a wide range of safety stakeholders.

Key words: safety program—web-based—WISLR

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Measurement and Evaluation of Tack Coat Application Rate for 4.75 mm NMAS Superpave Overlay Mix

Farhana Rahman
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
farhana@ksu.edu

Mustaque Hossain
Department of Civil Engineering
Kansas State University
2124 Fiedler Hall
Manhattan, KS 66506
mustak@ksu.edu

Stefan Romanoschi
Department of Civil and Environmental Engineering
University of Texas at Arlington
UTA Box 19308
Arlington, TX 76019-0308
romanoschi@uta.edu

ABSTRACT

Bond strength between two layers of hot mix asphalt (HMA) pavement is a major issue. Poor bond may be attributed to the premature fatigue failure, top down cracking, slippage cracks in the surface layer, and also difficulty in compaction due to insufficient shear strength. A thin and uniform application of emulsified asphalt or tack coat ensures a good bond between the existing surface and the overlying course. This paper discusses the evaluation of bond strength between the 4.75 mm Nominal Maximum Aggregate Size (NMAS) thin overlay and the Hot-in-Place Recycling (HIPR) layer for different tack coat application rates.

Tack coat measurements were taken on two rehabilitation projects on US 160 and K-25 in Kansas. Three different test sections were constructed on each project with variable rates of tack coat application. After one year in service, two in. diameter field cores were collected for laboratory bond tests. Laboratory pull-off tests show that most cores are fully bonded at the interface of the 4.75 mm NMAS overlay and the HIPR layer regardless of the tack application rate. The failure mode during pull-off tests at the HMA interface is highly dependent on the aggregate source and mix design of the adjacent layer material. This study also confirms that overlay construction with high tack coat application rate may attribute to the bond failure at HMA interface.

Key words: bond strength—interface layer—pull-off test—tack coat

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INTRODUCTION

Mixes with 4.75 mm Nominal Maximum Aggregate Size (NMAS) have the potential to provide smooth riding surface, improve ride quality and safety characteristics, extend pavement life, increase durability, and reduce permeability and road-tire noise. Many states, including Kansas, are forced to look at the pavement preservation techniques that are less costly due to budget constraints. Since past experiences with thin hot mix asphalt (HMA) overlays were good in a few states, the 4.75 mm NMAS mixes achieved positive attention by many state agencies. As the mixes are placed in thin lift application, they can be used for corrective maintenance, decrease construction time and cost, and may provide a very economical surface mix for low traffic volume facilities. However, since the layer is so thin, potential debonding of this mix was a real concern in Kansas. Thus, this study was initiated to research the bond strength of this layer incorporating this mix.

Recent Studies on Bond Strength

In 1999, the International Bitumen Emulsion Federation (IBEF) conducted a worldwide survey on the use of tack coat or bond coat materials. The survey collected information on tack material types, their application rates, curing time, test methods, and inspection methods. Responses from seven different countries confirmed that cationic emulsions are most commonly used with some use of anionic emulsion. The application rate generally ranged from 0.026 to 0.088 gal/yd^2 (0.12 to 0.4 l/m^2) (West, Zhang, and Moore 2005). No other countries expect Austria and Switzerland have bond strength testing methods and application criteria.

A bond strength study in Louisiana showed that tack coat type had significant influence on interface shear strength at lower temperature (Mohammad, Raqib, and Haung 2001). The optimum application rate for Cationic Rapid Setting Emulsion with Polymer (CRS-2P) was 0.02 gal/yd^2 (0.09 l/m^2). A Texas study on tack coat performance enhanced the idea that the nature of the interface surface type, which in turn was related to the aggregate structure of asphalt mix, had potential influence on tack coat performance (Yildirim, Smit, and Korkmaz 2005). The study also found that the bond strength of tack material was higher at higher application rate. A full-scale field study by the New Brunswick Department of Transportation (NBDOT) showed that one year and four year effectiveness of tack coat on pavement strength were statistically insignificant at the 95% confidence level (Mrawira and Yin 2006). The research conducted at Mississippi State University developed a tack coat evaluation device and performed laboratory testing on different tack coat application rates. The study concluded that tack coat type had significantly affected the bond strength of pavement layers. Tack coat setting time and evaporation rate of water from emulsified asphalt had potential impact on tack coat performance (Buchanan and Woods 2004). Recent National Center for Asphalt Technology (NCAT) study illustrated that the mixture type was an important factor that had affected bond strength at interface. Analysis concluded that fine-graded mixture with smaller NMAS had higher bond strength compared to the coarse-graded mixture with larger NMAS while temperature was another important factor to evaluate bond strength (West, Zhang, and Moore 2005). A Washington study showed that effect of tack coat on milled section was insignificant while bond strength on non-milled roadway section was highly dependent on tack coat performance (Tashman, Nam, and Papagiannakis 2006). This study also confirmed that tack coat curing time should be an important consideration during construction. A recent study at Kansas State University has confirmed that current specification of the Kansas Department of Transportation (KDOT) (approximately 21 gm/ft^2) for tack coat application rate would be sufficient to produce higher strength in all mixture types. Tack coat performance at fine to fine and fine to coarse interface layer was better compared to coarse to coarse mixture combination (Wheat 2007).

PROBLEM STATEMENT

Numerous variables control the tack coat performance to ensure sufficient bond between layers. The emulsified asphalt must break before it can be used as adhesive material. Inadequate emulsion curing may also introduce non-effective tack application (Buchanan and Woods 2004). Hence, the proper tack coat application rate is another vital issue to obtain optimum bond strength between HMA layers. Current specifications only introduce a range of application rates and the construction practice completely depends on field management to set the target. Some state agencies provide guidelines for use of heavier application rates for old HMA and concrete pavement, while lighter application rate or no tack coat is suggested for newly paved HMA mat. However, optimum application rate may differ from one state agency to another based on the total emulsion or the asphalt residue (West, Zhang, and Moore 2005). In 2005, NCAT investigated the bond strength performance of a new fine Superpave mixture with 4.75 mm Nominal Maximum Aggregate Size (NMAS). This study was conducted only on laboratory specimens to obtain optimum application rate. To date, no study has been done on the bond strength of the 4.75 mm NMAS mixture overlay achieved in the field.

OBJECTIVE

The primary objective of this research study was to evaluate the performance of tack coat for the 4.75 mm NMAS superpave mix overlay at different application rates. The tack coat performance was examined based on the interface bond strength test using field samples and a tensile strength measuring device.

TEST SECTIONS

Two rehabilitation projects were done in the summer of 2007 using 4.75 mm NMAS Superpave mixture. Three test sections with variable tack coat application rates were set up on each project. The strip maps for the test sections on US 160 and K-25 are shown in Figures 1a and 1b, respectively. Test section lengths on US 160 and K-25 were 120 ft (36.5 m) and 200 ft (61 m), respectively. During construction, high-performance slow-setting emulsified asphalt (SS-1HP) was applied at three different rates—low (0.02 gal/yd^2), medium (0.04 gal/yd^2), and high (0.08 gal/yd^2)—on the Hot-In-Place Recycled (HIPR) asphalt layer. After the tack coat sections were set up, normal pavement construction practices were followed which included HMA haul truck backing over the tack surfaces. A three-fourth inch (19 mm) thick overlay of 4.75 mm NMAS Superpave mixture was laid on the HIPR layer and compacted. Cores at every 20 ft (US 160) and 15 ft (K-25) intervals were collected along the right wheel path about one year after construction.

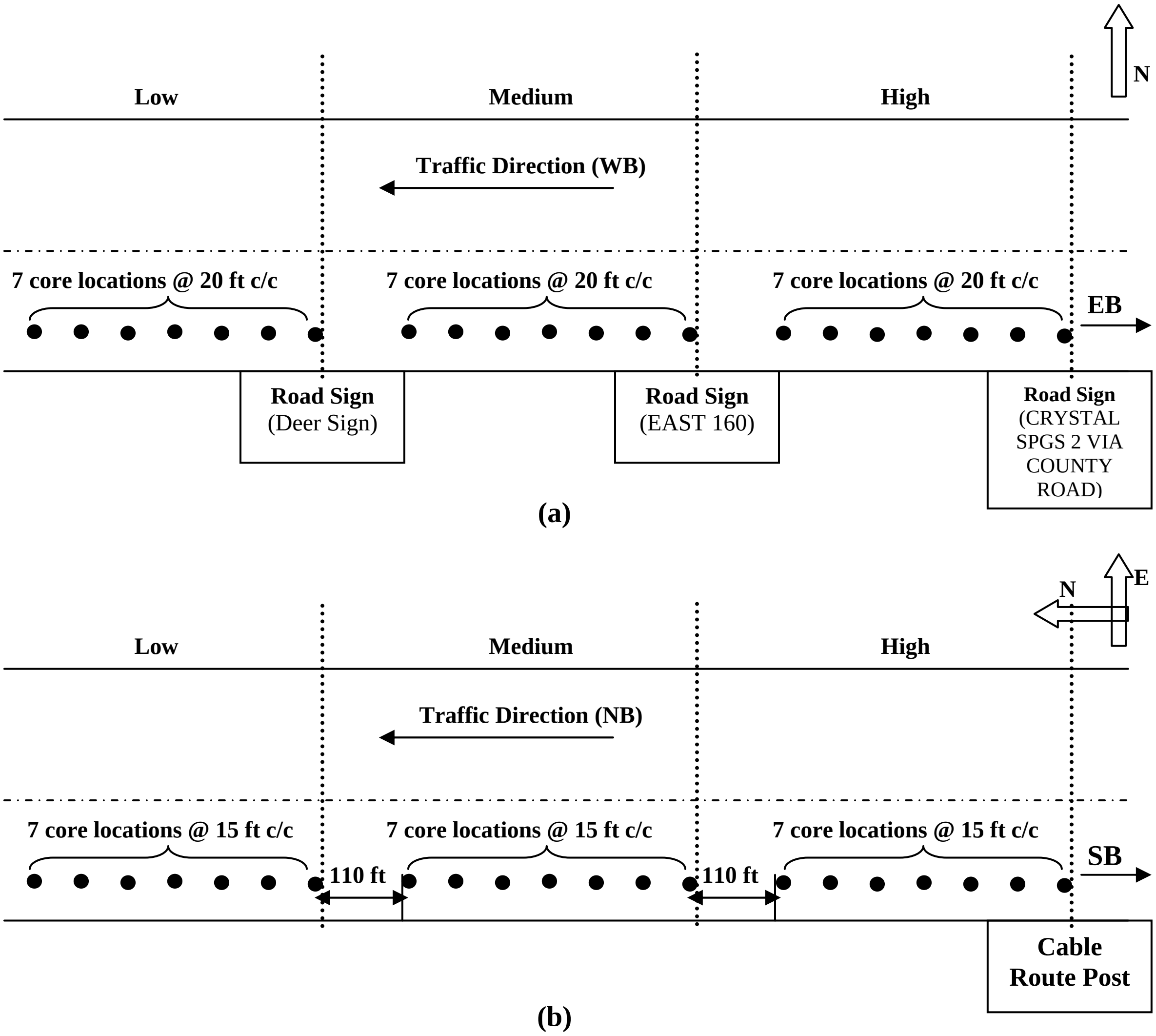
N
Low
Medium
High
Traffic Direction (WB)
7 core locations @ 20 ft c/c
7 core locations @ 20 ft c/c
7 core locations @ 20 ft c/c
EB
Road Sign (Deer Sign)
Road Sign (EAST 160)
Road Sign (CRYSTAL SPGS 2 VIA COUNTY ROAD)
(a)
N
E
Low
Medium
High
Traffic Direction (NB)
7 core locations @ 15 ft c/c
7 core locations @ 15 ft c/c
7 core locations @ 15 ft c/c
SB
110 ft
110 ft
Cable Route Post
(b)

Figure 1. Tack coat measurement and coring locations on (a) US 160 and (b) K-25 tack coat application rate measurements

The tack coat test sections were set up in June 2007. In situ tack application rate was measured at seven locations to check the actual application rates on each test section. The measurements were taken using pre-weighed, 1 × 1 ft dry wooden planks. A high-performance, slow-setting tack (SS-1HP) was used on both project locations. The tack coat properties are listed in Table 1.

Table 1. Tack coat properties used on US 160 and K-25 projects

Route	County	Tack Material	Shooting Temperature °F	Unit Weight (lbs/gal)	Specific Gravity	Residual Asphalt (%)
US 160 (EB)	Harper	SS-1HP	170	8.49	1.018	59.8
K-25 (SB)	Rawlins	SS-1HP	175	8.49	1.018	59.8

*1 lb/gal = 0.57 kg/l; 1^0 F = 17.78^0 C

The wooden planks were placed on the right wheel path of the roadway before the distributor truck applied the tack coat. After passage of the distributor truck, the planks were removed and weighed to determine the application rate. Figure 2 shows the distribution and measurement of tack coat on the US 160 project.

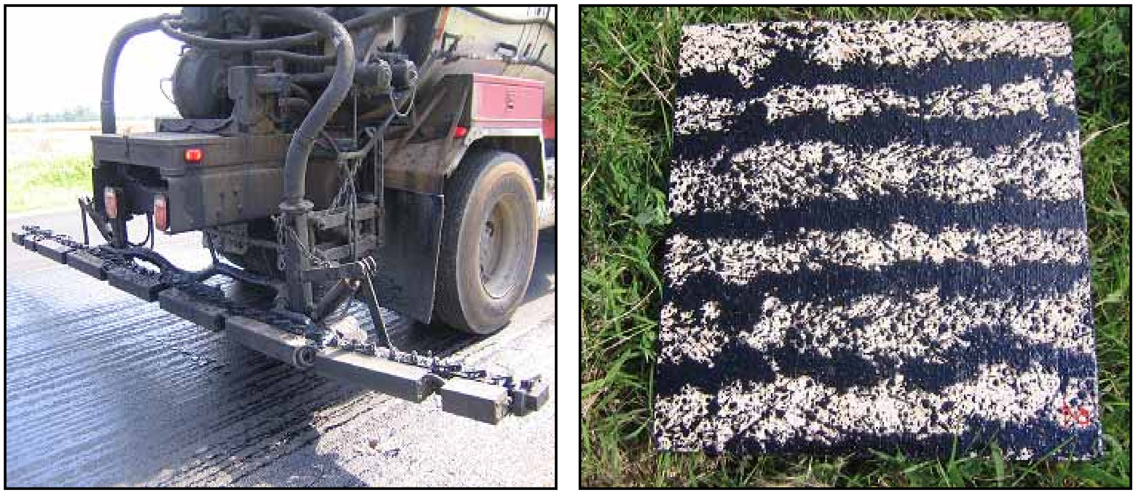

Figure 2. Tack coat application and measurement on US 160 in Harper County

Field Core Collections

The cores were collected in June 2008, one year after paving and traffic operation. Seven, 2 in. (approximately 51 mm) cores were collected along the right wheel path from each test section (Figure 3). No debonding occurred at the HIPR layer during core collection. The collected cores were cut to a height of 2 in. (51 mm) to perform pull-out tests. The test specimen contained only ¾ in. (19 mm) of 4.75 mm NMAS overlay and HIPR with tack coat at the interface.

Figure 3. Core collections on US-160 and K-25 route

PERFORMANCE EVALUATION TEST PROCEDURE

Pull-Off Test for Tensile Strength Measurement

The American Society of Testing and Materials (ASTM) has specified a standard test titled "Standard Test Method for Pull-Off Strength of Coating Using Potable Adhesion Tester." It measures the tensile force required to remove two bonded flat surfaces. The test result can be reported either in pass/fail system or by recording tensile force to split the bonded layers. No specifications are available regarding the initial normal force or pre-compression time required to perform the test. According to the ASTM standard, these initial conditions should be assigned by the testing apparatus manufacturer (ASTM 2003). KDOT has partially adopted this test procedure to evaluate in situ bond strength in the field. During this research study, the KDOT procedure was followed with a SATEC model T 5000 universal testing machine. Before testing, the cores were glued to a metal plate on both sides using epoxy (Pro-Poxy 300 fast A/B) as shown in Figure 4. The epoxy needs 16 to 24 hours to set and to make perfect bonding with the bituminous material. During testing, the core samples were conditioned under normal load of 0 to 10 lb (0 to 4.5 kg) for 5 seconds. The applied displacement was set to 1 in./min (25 mm/min). The test samples were then loaded to fail in direct tension (Figure 4).

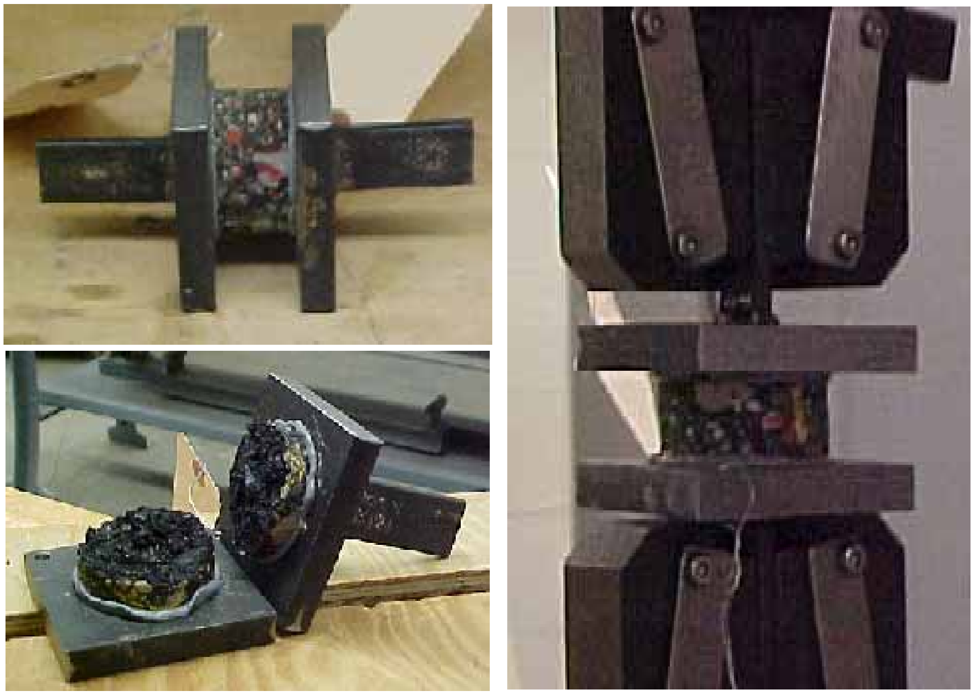

Figure 4. Pull-off strength test of tack coat material

RESULTS AND ANALYSIS

Tack Coat Application Rate Measurement

As mentioned earlier, three application rates were selected for each project location. Seven measurement points were set at 20 and 15 ft intervals on the US 160 and K-25 projects, respectively. Tables 2 and 3 show the measured application rates. The application rate measured during construction is fairly close to the target value on the K-25 project. However, on US 160, the measured application rates are way off the target rates. The high application rate was not achieved during construction. The statistical summary (mean and standard deviation) for the US 160 project shows less scatter compared to the K-25 project. These tables confirm that three distinct sections based on the tack coat application rate were not achieved on US 160. This implies that better equipment calibration is needed in the field.

Table 2. Measured tack coat application rate on US 160

Route	Section	Plank #	Plank Initial Weight (lbs)	Plank Weight with Tack (lbs)	Residue (lbs)	Application Rate (gal/yd^2)
US 160	High	1	0.80	0.82	0.0249	0.0264
		2	0.76	0.80	0.0344	0.0365
		3	0.77	0.81	0.0401	0.0425
		4	0.79	0.83	0.0362	0.0383
		5	0.73	0.76	0.0373	0.0395
		6	0.72	0.76	0.0392	0.0416
		7	0.75	0.79	0.0364	0.0386
		Avg.				0.038
		STDEV				0.0054
		Target				**0.08**
	Medium	8	0.77	0.81	0.0397	0.0421
		9	0.78	0.82	0.0370	0.0393
		10	0.75	0.78	0.0375	0.0397
		11	0.78	0.81	0.0353	0.0374
		12	0.72	0.75	0.0311	0.0330
		13	0.72	0.75	0.0238	0.0252
		14	0.79	0.82	0.0302	0.0320
		Avg.				0.036
		STDEV				0.0058
		Target				**0.04**
	Low	15	0.74	0.74	0.0053	0.0056
		16	0.80	0.82	0.0229	0.0243
		17	0.76	0.77	0.0104	0.0110
		18	0.76	0.78	0.0225	0.0238
		19	0.78	0.82	0.0355	0.0376
		20	0.78	0.81	0.0309	0.0327
		21	0.79	0.81	0.0194	0.0206
		Avg.				0.022
		STDEV				0.0113
		Target				**0.02**

*1 lb = 0.454 kg; ** 1 gal/yd^2 = 4.527 l/m^2

Table 3. Measured tack coat application rate on K-25

Route	Section	Plank #	Plank Initial Weight (lbs)	Plank Weight with Tack (lbs)	Residue (lbs)	Application Rate (gal/yd^2)
K-25	High	1	0.80	0.882	0.086	0.0907
		2	0.78	0.851	0.072	0.0763
		3	0.73	0.803	0.073	0.0774
		4	0.66	0.761	0.098	0.1038
		5	0.70	0.788	0.092	0.0977
		6	0.71	0.794	0.086	0.0907
		7	0.73	0.825	0.097	0.1031
		Avg.				0.09
		STDEV				0.0112
		Target				**0.08**
	Medium	8	0.77	0.827	0.054	0.0574
		9	0.80	0.845	0.049	0.0520
		10	0.74	0.783	0.045	0.0480
		11	0.67	0.713	0.049	0.0520
		12	0.69	0.741	0.051	0.0546
		13	0.75	0.807	0.058	0.0616
		14	0.70	0.752	0.054	0.0576
		Avg.				0.05
		STDEV				0.0045
		Target				**0.04**
	Low	15	0.72	0.748	0.026	0.0273
		16	0.68	0.713	0.035	0.0375
		17	0.67	0.693	0.021	0.0222
		18	0.74	0.759	0.021	0.0222
		19	0.82	0.836	0.016	0.0173
		20	0.79	0.796	0.008	0.0084
		21	0.76	0.759	0.001	0.0007
		Avg.				0.02
		STDEV				0.0121
		Target				**0.02**

*1 lb = 0.454 kg; ** 1 gal/yd^2 = 4.527 l/m^2

Pull-Off Tests on Field Cores

Pull-off strength test measures the tensile force required to remove two bonded flat surfaces. The results obtained in the Pull-off strength in this study are shown in Table 4. The cores were selected randomly from the seven locations from each test section to get unbiased results. A very high variability in the pull-off strength was observed even considering the same core location, tack application rate and failure mode. In most cases on both project locations, the tensile failure occurred within the HIPR layer material and surface material rather than at the interface of the 4.75 mm NMAS Superpave overlay and the HIPR layer. The results from US 160 imply that complete bonding was achieved between the 4.75 mm NMAS fine mix overlay and the HIPR layer regardless of the tack coat application rate. In general, the HIPR layer material performed well compared to the surface material on US 160. The overall failure in the surface mix overlay was 55%, while 45% of the total failure occurred in the HIPR layer material.

However, the test section with higher tack coat experienced higher percentage (57%) of failure within the HIPR layer.

Table 4. Summary of pull-off test results

Test Sections	Core Location	Pull-Out/Tensile Force (lbs)			
		US 160		K-25	
High	1	404 (SMF[a])	836 (HIPR[b])	581 (PBF[c])	527 (SMF[a])
	2	356 (SMF[a])	801 (SMF[a])	120 (SMF[a])	439 (PBF[c])
	3	617 (HIPR[b])	795 (SMF[a])	453 (SMF[a])	635 (PBF[c])
	4	786 (HIPR[b])	780 (SMF[a])	707 (HIPR[b])	142 (PBF[c])
	5	174 (HIPR[b])	676 (SMF[a])	730 (HIPR[b])	505 (PBF[c])
	6	660 (HIPR[b])	459 (HIPR[b])	615 (HIPR[b])	652 (PBF[c])
	7	645 (HIPR[b])	531 (HIPR[b])	427 (SMF[a])	580 (HIPR[b])
Medium	8	624 (HIPR[b])	321 (SMF[a])	808 (HIPR[b])	412 (SMF[a])
	9	420 (SMF[a])	389 (SMF[a])	374 (SMF[a])	199 (SMF[a])
	10	461 (SMF[a])	459 (SMF[a])	202 (SMF[a])	270 (HIPR[b])
	11	253 (SMF[a])	585 (SMF[a])	504 (HIPR[b])	637 (PBF[c])
	12	668 (HIPR[b])	229 (HIPR[b])	242 (SMF[a])	266 (HIPR[b])
	13	454 (HIPR[b])	673 (SMF[a])	210 (HIPR[b])	505 (HIPR[b])
	14	743 (HIPR[b])	590 (SMF[a])	201 (HIPR[b])	146 (HIPR[b])
Low	15	502 (SMF[a])	452 (SMF[a])	224 (HIPR)	225 (HIPR[b])
	16	675 (SMF[a])	456 (SMF[a])	326 (SMF[a])	328 (HIPR[b])
	17	311 (HIPR[b])	696 (HIPR[b])	305 (HIPR[b])	457 (HIPR[b])
	18	570 (HIPR[b])	420 (HIPR[b])	646 (HIPR[b])	219 (HIPR[b])
	19	869 (HIPR[b])	196 (SMF[a])	795 (HIPR[b])	395 (HIPR[b])
	20	890 (HIPR[b])	460 (SMF[a])	821 (HIPR[b])	502 (HIPR[b])
	21	716 (SMF[a])	230 (SMF[a])	517 (SMF[a])	103 (HIPR[b])

[a] = Surface material failure
[b] = HIPR layer material failure
[c] = Partial bond failure

On K-25, some partial debonding occurred on the test section with high tack coat application rate, while only one core failed at the section with medium tack rate. The test results show that the HIPR layer materials are the weakest compared to the surface mix and interface layer strength. Approximately 57% of the total failure occurred in HIPR while 26% failure was observed in surface material and 17% at interface of the adjacent layer material. However, 43% of the field cores from test section with higher tack coat application rate had failed at layer interface. This finding is notably important as it implies that the high tack application rate might be too high to provide sufficient bond strength for the overlay. Another significant finding is that the bond strength at the HMA interface is highly dependent on the aggregate source and volumetric mix design of the adjacent layer material.

CONCLUSIONS

Based on this study, the following conclusions can be made:

- Although the distributor was set to deliver three distinct tack coat application rates, those were not achieved on US 160, emphasizing the need for better calibration.
- During pull-off testing, the tensile failure occurred within the surface mix and HIPR materials rather than at the interface with the 4.75 mm NMAS overlay. This implies that the top overlay layer is fully bonded with the HIPR layer in most cases.
- Overlay construction with high tack coat application rate may be attributed to the bond failure at HMA interface.
- The failure force during pull-off tests at the HMA interface is highly dependent on the aggregate source and mix design of the adjacent layer material.

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Fatality Risk of Older Drivers under Different Conditions Based on Vehicle Miles Traveled

Abhiteja Rallabandi
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
abhiteja@ksu.edu

Sunanda Dissanayake
Department of Civil Engineering
Kansas State University
2118 Fiedler Hall
Manhattan, KS 66506
sunanda@ksu.edu

ABSTRACT

The rapid increase in older driving population calls for safety in their driving requirements. Fatal crash involvement of older drivers based on per mile driven basis is high, even when the total number of fatal crashes is relatively small, indicating the importance of exposure. Therefore, this study focused on some of the critical characteristics associated with fatality risk of older drivers. This was done by analyzing the crash rates in crashes per vehicle miles traveled under different conditions and situations. Crash data are obtained from the Fatality Analysis Reporting System (FARS) and number of miles traveled is from the National Household Travel Survey (NHTS). Time period considered for the crash data is 1997 to 2006, where the midpoint corresponds with the NHTS data. The average annual fatality rates calculated indicate that the risk for older drivers increases with increased age under different light conditions, vehicle type, road conditions, race, and gender of the driver. Thus, the driving conditions of elderly people have to be enhanced for the betterment of their safety.

Key words: FARS—fatality rates fatality risk—older drivers—safety—vehicle miles traveled

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INTRODUCTION

There has been a rapid growth of eleven fold in older population in the United States since 1900 and a twofold increase is estimated during 2010 to 2030 after the first baby boomers (people born between 1946 and 1964) start turning 65 years old in 2011 (U.S. Administration on Aging [AOA] 2006). For the decade 1996 to 2006, percentage increase in licensed older drivers was 18%, whereas increase in the total number of licensed drivers (inclusive of all ages) was 13% only (National Highway Traffic Safety Administration [NHTSA] 2009). With the estimated increase in older population, an increase in older driver population could naturally be expected. Traffic crashes are one of the main reasons for fatalities of all age groups in the United States (National Conference of State Legislature [NCSL] 2008). Elderly accounted for 8% of all traffic crashes and 14% of all traffic fatalities during 2007. According to the National Highway Traffic Safety Administration (NHTSA), fatality rates for older drivers are much higher than that of middle-aged ones (NHTSA 2009). With the predicted growth in population and increased risk of fatality, older drivers' safety becomes an important component to be considered.

The risk associated with older drivers is not only a factor of number of crashes but also the exposure. Older drivers are safe when exposure is considered in number of licensed drivers or population, but they have a very high risk associated with amount of exposure in vehicle miles traveled. According to the National Household Travel Survey data (NHTS), number of miles driven decrease with increase in age group (Contrino et al. 2006). Driving skills for any individual usually depends on three factors: physical fitness, good perception, and clear vision (U.S. National Institutes of Health 2008). The cessation in driving of older drivers is caused by setback in those required driving skills like deterioration in health, poor visibility, and memory impairment, which are some of the key factors resulting in both decline of driving and increase in number of crashes. Thus, analysis of characteristics of older drivers is done in this study by evaluating the risk involvement using crash rates.

Based on actual amount of driving and availability of data, five major characteristics are considered for this study. They are the following: age and gender of the driver, light conditions, road conditions, vehicle type, and race of the driver. Age is one of the basic characteristics considered in studying the variation in crash risk for older drivers. In 2007, 79% of the traffic fatalities involving older drivers occurred during the daytime (Insurance Information Institute [III], 2009). With deterioration in vision due to increase in age, light condition has been one of the factors chosen for analysis. According to the National Center for Statistics and Analysis (NCSA) of NHTSA, 23%of the U.S. population lived in rural areas in 2006 and rural fatal crashes accounted for 56%of all the traffic fatalities. There was a 7% decrease in rural fatalities and 9% increase in urban fatalities from 1997 to 2006 (NHTSA 2008). Many factors like increase in number of licensed drivers, speed limits, light conditions, blood alcohol content, and urban conditions affect the increase in fatalities, making road conditions an important factor to be studied.

In 2007, older population made up to 14%of all vehicle occupant fatalities. In a two-vehicle fatal crash, an older driver was twice as likely to get struck when compared to a younger driver. This is because they are frailer and more likely to die from their injuries than younger people. Vehicle type was one of the factors considered for this study as weight of the vehicle played an important role in personal injury severity (NHTSA 2009). For example, an SUV might be a difficult vehicle to handle for elderly when compared to a lighter vehicle like passenger car. Migration to the United States has started several centuries ago and has been a continuous process since then (Genealogy.com 2007). Race is a newly emerging issue with large diversity of people in the United States. To study the wide range of race distribution in the population and its changing effects on older population, race has been chosen for the analysis.

LITERATURE REVIEW

According to a recent study by the U.S. Census Bureau, an increase is estimated in the older population due to baby boomers. Older population will be around 70 million in 2030, more than twice their number in 2000, and this estimated increase has increased interest in their driving safety (AOA 2006). Increase in older population might result in an increase in the older licensed driving population. Driving is a vital factor of self-sufficiency, allowing for liberty and mobility. Research has shown that with increase in age, visual, cognitive, and perceptual functions deteriorate and susceptibility to injury increases with increase in age (New York State KWIC 2008).

The Fatality Analysis Reporting System, General Estimates System, Nationwide Personal Transportation Survey, and National Household Travel Survey are the databases used for most of the crash and exposure studies. Driver, vehicle, and environment-related factors are some of the critical crash characteristics responsible for fatal crashes (Baker et al. 2003). The number of miles traveled reveals that older drivers experience higher crash fatality rates when compared to all the other age groups except for younger drivers (Cerrelli 1997). The possible reasons for higher fatality rates on per mile basis are higher crash involvement and higher fragility (Austin & Faigin 2003).

Many previous studies related to older driver safety were mainly revolving around age group, gender, speed limit, location of the crash, and weather condition. Older drivers are overrepresented in intersection-related crashes, failure to yield right-of-way crashes, and traveling at lower speeds on roads and in daylight conditions (McGwin & Brown 1999). They are also overrepresented in left-turn, right-turn crashes and angle collisions (Abdel-Aty et al. 1999). Older drivers are more likely to be hospitalized than any other age group when involved in fatal crashes. Lane changing was also found to be a difficult maneuver for older drivers (Chandraratna & Stamatiadis 2003). Older drivers make relatively low contributions to crash-related injuries or deaths, but their contribution of injuries to self is more than to others (Delinger et al. 2004). Risk ratios and crash rates are the most commonly used units of analyses. Different statistical analysis like log-linear method, logistic regression, and some general methods like decomposition methodology were used to calculate the crash rates and model the characteristics to find out the behavior of older drivers under different conditions.

In order to account for fragile nature of older drivers, NHTSA has recommended appropriate driving practices for individuals with various conditions and developed information to help them in making safe driving choices (NHTSA 2009). Other organizations involved in enhancing the driving safety of older population are the American Association for Retired Persons, the American Medical Association, and the American Automobile Association. Thus, for highway safety of older drivers countermeasures have to be suggested accounting to fragility of older drivers (Delinger et al. 2002).

DATA

Crash Data

The database used in this study for studying fatal crashes involving older drivers is the Fatality Analysis Reporting System (FARS). It is maintained by NHTSA and data are recorded for all the 50 states, the District of Columbia, and Puerto Rico. FARS contains details at crash level, person level, and vehicle level. For a crash to make an entry into this database, it has to involve a motor vehicle traveling on the traffic way open to the public resulting in death of at least one person within 30 days of the crash (Fatality Analysis Reporting System [FARS] 2009).

The data is categorized into accident, person, and vehicle files. Details related to individuals and vehicles involved in crashes are present in person and vehicle files, respectively. Data are present in statistical analysis software (SAS) compatible format. The analysis period for crash data considered in this study is 10 years, from 1997to 2006. This long span is considered to have a detailed analysis of trends for different age groups. Characteristics are studied in this study by considering fatality as the unit of analysis.

Exposure Data

Exposure considered in this study is the number of vehicle miles traveled. Data are taken from the National Household Travel Survey database. It is an endeavor by the U.S. Department of Transportation (U.S. DOT) sponsored by the Bureau of Transportation Statistics (BTS) and the Federal Highway Administration (FHWA) to gather data on both long-distance and local travel by the American public. All the data related to trips such as number of miles traveled, time of travel, duration of the trip, purpose of the trip, weather conditions during the trip, etc. are considered in the survey to aid policy makers and transportation planners in their day-to-day work on travel. This survey is conducted once in every five years on an average for sample number of households in the United States. Latest data available is for the year 2001. Some of the variables in the 2001 file are updated from previous years' travel data, i.e., 1969, 1977, 1983, 1990, and 1995. The Nationwide Personal Transportation Surveys and American Travel Survey was conducted in 1977 and 1995. The database is divided into person, vehicle, travel day, and household files which are easily accessible to public.

Files used for this analysis are person, vehicle, and household files. The number of miles traveled is taken in 10^9 vehicle miles traveled (Billion Vehicle Miles Traveled). According to Highway Statistics 2001, the older drivers sample in the NHTS 2001 file is 0.07% of the entire driving population (Contrino et al. 2006). Year 2001 is in the middle of the time span considered for FARS data; average annual fatality rates were calculated correspondingly.

METHOD

Fatality Rate

The Average Annual Fatality Rate (AAFR) is calculated as the ratio of number of fatalities to the number of miles driven under each category. For detailed analysis, older drivers are categorized into five groups based on age. They are 65 to 69 years, 70 to 74 years, 75to 79 years, 80 to 84 years, and 85 years and older. Using FARS for fatalities and NHTS for vehicle miles traveled, rates are calculated for all age groups and genders under different light conditions, road conditions, vehicle types, and race categories. AAFR is defined using the following equation:

$$\text{Average Annual Fatality Rate} = \frac{\text{Average annual fatalities}}{\text{Number of miles driven in } 10^9 \text{ VMT}} \tag{1}$$

Average Annual Fatalities

Average annual fatalities are calculated for older drivers based on age group for a 10-year analysis period. Number of fatalities decreased with increase in age of older drivers. Average annual fatalities based on gender for all age groups is also calculated similarly and plotted in Figure 1. Number of fatalities for males was around 75% of the total.

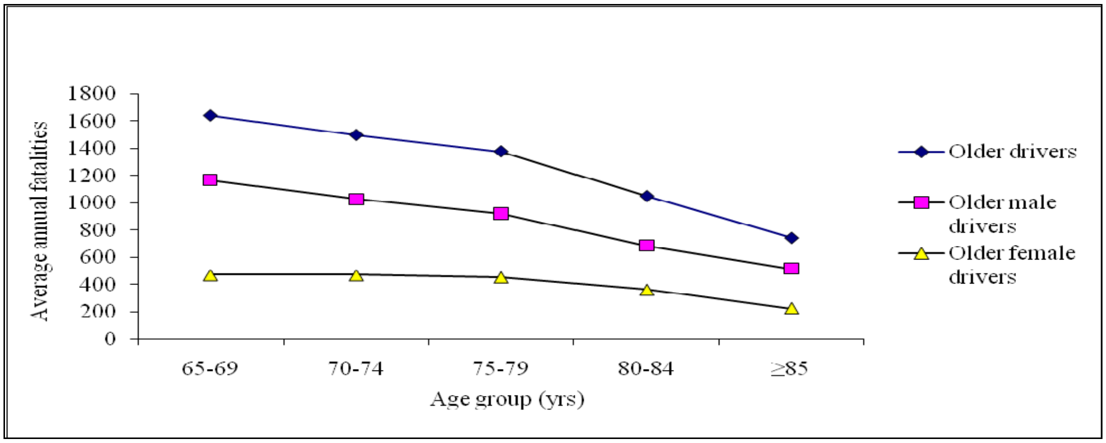

Figure 1. Average annual fatalities based on age and gender of older drivers for 1997 to 2006

Total Miles Traveled

The variable used in NHTS to obtain data for miles driven is YEARMILE. Miles driven by NHTS sample of older drivers and calculated total vehicle miles driven is given in Table 1. Since NHTS contains data for sample population interviewed in the United States, total vehicle miles driven by all age groups is calculated using equation (2).

$$\text{Total miles driven} = \text{Total no. of drivers} \times \frac{\text{Miles driven by sample no. of drivers}}{\text{No. of drivers included in NHTS}} \quad (2)$$

Details for total number of older drivers in 2001 are obtained from NHTSA. The calculated total miles are used for further calculations of AAFR. Total number of miles driven based on gender is calculated using data from highway statistics and it shows that miles driven decreases with increase in age and approximately 75% of the total miles are driven by males.

Table 1. Number of drivers and miles driven from NHTS and corresponding total miles driven

	Number of drivers and miles driven based on NHTS sample		Total number of drivers and miles driven	
Age Group (years)	**Number of older drivers in NHTS sample**	**Miles driven by older drivers in in NHTS sample**	**Total number of older drivers**	**Total number of miles driven in Billion Vehicle Miles Traveled (BVMT)**
65-69	6,721	49,730,499	8,436,274	62
70-74	5,955	36,298,414	7,464,825	45.5
75-79	4,320	21,690,095	5,912,286	29.7
80-84	2,415	10,079,730	3,653,273	15.2
≥85	990	2,778,780	2,106,116	5.9

Light Conditions

AAFR based on light conditions is calculated for daylight and dark conditions. Light conditions vary for different periods of the year. Hence, classification is done by combining four months each and putting them into three categories. Daylight and dark conditions are considered accordingly.

The number of fatalities is filtered using variable LGT_COND which takes a value of 1 for daylight and 2, 3, 4, 5 for dark conditions. Total miles traveled under different conditions are calculated using equation (2).

Vehicle miles driven by all five age groups under daylight and dark conditions are given in Table 2. Approximately 90% of the trips by older drivers are made in daylight conditions.

Table 2. Miles driven by older drivers in different light conditions from NHTS sample

Age Group (yrs)	Miles driven in daylight conditions (%)	Miles driven in dark conditions (%)	Total miles driven for the age group (%)
65-69	216,996 (85.97)	35,408 (14.03)	252,404 (100.0)
70-74	170,289 (89.57)	19,839 (10.43)	190,128 (100.0)
75-79	108,556 (91.35)	10,277 (8.65)	118,833 (100.0)
80-84	47,558 (90.03)	5,268 (9.97)	52,826 (100.0)
≥85	14,101 (92.51)	1,142 (7.49)	15,243 (100.0)

The percentages calculated for miles driven in different light conditions from Table 2 are applied to the total miles driven in Table 1. A sample calculation for total miles traveled by drivers of 65 to 69 yrs in daylight is the following:

$$\left.\begin{array}{l}\text{Total no. of miles driven}\\ \text{in daylight by } 65-69 \text{ yrs}\\ \text{age group}\end{array}\right\} = \left\{\left[\begin{array}{l}\text{Percentage of miles}\\ \text{driven in daylight by}\\ 65-69 \text{ yrs age group}\\ \text{obtained from NHTS}\end{array}\right] \times \text{total miles driven}\right\} \text{ BVMT} \tag{3}$$

$$= \left(0.8597 \times 62 \times 10^{9}\right) = 54.7 \text{ BVMT}$$

Similarly, total number of miles is calculated for other age groups in daylight and dark conditions using equation (3). AAFR for different light conditions is evaluated using equation (4).

$$\left.\begin{array}{l}\text{AAFR under daylight conditions}\\ \text{for } 65-69 \text{ yrs age group}\end{array}\right\} = \frac{\begin{array}{l}\text{Avg. annual fatalities for } 65-69 \text{ yrs}\\ \text{age group under daylight conditions}\end{array}}{\begin{array}{l}\text{Calculated total miles for } 65-69 \text{ yrs}\\ \text{age group driven under daylight conditions}\end{array}} \tag{4}$$

A sample calculation of AAFR for drivers of age group 65 to 69 yrs driving in daylight conditions is the following:

$$\text{AAFR} = \frac{1189 \ \text{fatalities}}{54.56 \ \text{BVMT}} = 21.79 \ \text{fatalities per} \ \ \text{BVMT} \tag{5}$$

Urban and Rural Conditions

AAFR for older drivers driving in different road conditions are evaluated using the Urban/Rural (URBRUR) variable in NHTS for miles driven and Roadway Function Class in FARS for fatalities. The percentages calculated for miles driven in urban and rural areas from NHTS sample are applied to values in Table 1 to calculate total miles driven. AAFR for different road conditions is calculated using equation (4).

Vehicle Type

The variables used to identify the type of vehicle in FARS and NHTS are BODY_TYP and VEHTYPE, respectively. All vehicles are categorized into seven major categories: automobiles, vans, SUVs, pickup trucks, heavy trucks, motorcycles, and other vehicles like snowmobiles and construction equipment, which are used by older drivers in different percentages. The highest percentage of vehicles involved in fatal crashes of older drivers is automobiles. AAFR for different vehicle types is calculated using equation (4).

Race

Race for this study has been divided into five major categories: White, African American, American Indian, Asian, and others. The variable used to filter data in FARS and NHTS is RACE. Race in FARS is recorded for fatal injuries only. AAFR based on race for all age groups and gender is calculated using equation (4).

RESULTS

AAFR calculated using crash and exposure data for five age groups, gender, different driver, vehicle, and environment-related conditions are discussed in this section.

Age and Gender Conditions

AAFR evaluated for different age groups and gender is presented in Table 3. It is lowest for 65 to 69 years group. It has shown a continuous increase with increase in age of the driver and is highest for 85 years and older age group for all the three cases (all older, male, and female drivers), as shown in Figure 2. For all the age groups, female drivers had higher fatality rates than male drivers. This is attributed to the lesser number of miles driven by females comparatively.

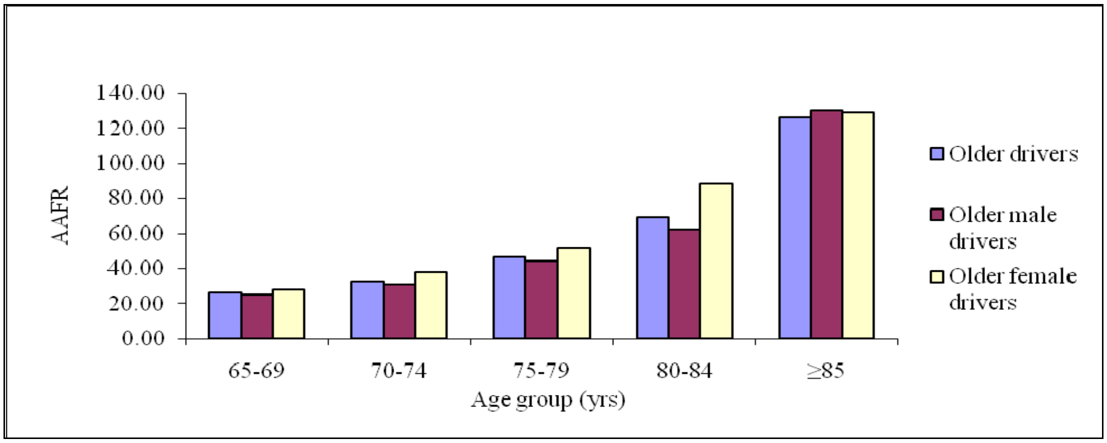

Figure 2. Average annual fatality rates for older drivers based on age and gender

Light Conditions

The fatality rates under daylight and dark conditions are highest for female drivers for all the age groups when compared to male drivers. This might be attributed to the fact that the number of crashes in which female drivers were involved was less than half of the crashes involved by males, and hence, the crashes per 10^9 miles increased radically. There is a sudden rise in the crash rate from 80 to 84 years age group to the 85 years and older group as the number of miles driven by them is less compared to all other age groups. AAFR for different light conditions are plotted in Figure 3.

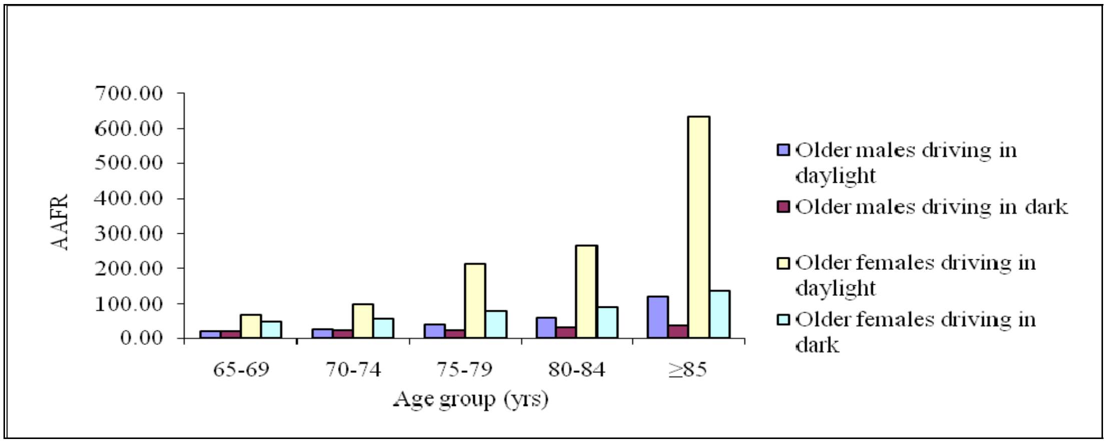

Figure 3. AAFR for older drivers based on light conditions

Table 3. Average Annual Fatality Rates for older drivers based on age group, gender, light, and road conditions

Older Drivers (Rates in average annual fatalities per BVMT)

Age Group (yrs)	Based on persons involved in fatal crashes	Based on light conditions		Based on road conditions	
		Daylight conditions	Dark conditions	Urban	Rural
65-69	26.45	21.79	56.99	14.68	48.97
70-74	32.94	28.02	61.90	18.92	62.00
75-79	46.41	40.33	93.53	26.75	98.67
80-84	69.10	62.17	91.51	42.10	143.68
≥85	126.07	118.46	175.85	72.45	321.18

Older Male Drivers

Age Group (yrs)	Based on persons involved in fatal crashes	Based on light conditions		Based on road conditions	
		Daylight conditions	Dark conditions	Urban	Rural
65-69	25.33	21.41	22.18	14.29	47.02
70-74	31.13	26.74	23.71	18.00	59.38
75-79	44.34	39.34	23.27	25.35	95.62
80-84	62.47	58.93	30.85	37.33	138.55
≥85	130.30	120.93	37.61	74.58	320.73

Older Female Drivers

Age Group (yrs)	Based on persons involved in fatal crashes	Based on light conditions		Based on road conditions	
		Daylight conditions	Dark conditions	Urban	Rural
65-69	27.78	69.77	49.98	14.70	51.96
70-74	37.70	98.60	56.99	21.22	70.86
75-79	51.77	212.29	79.18	29.81	110.67
80-84	88.73	267.81	90.02	56.82	155.22
≥85	129.09	633.13	134.93	74.82	353.57

Road Conditions

Fatality rates in rural conditions are nearly thrice that in urban conditions both for males and females as shown in Table 3. It increases with increase in age and is highest for 85 years and older group. Fatality rates under urban conditions are comparatively higher for females than males. Increase in fatality rates under rural conditions can be contributed to lesser amount of travel and condition of the road. AAFR based on road conditions is plotted in Figure 4.

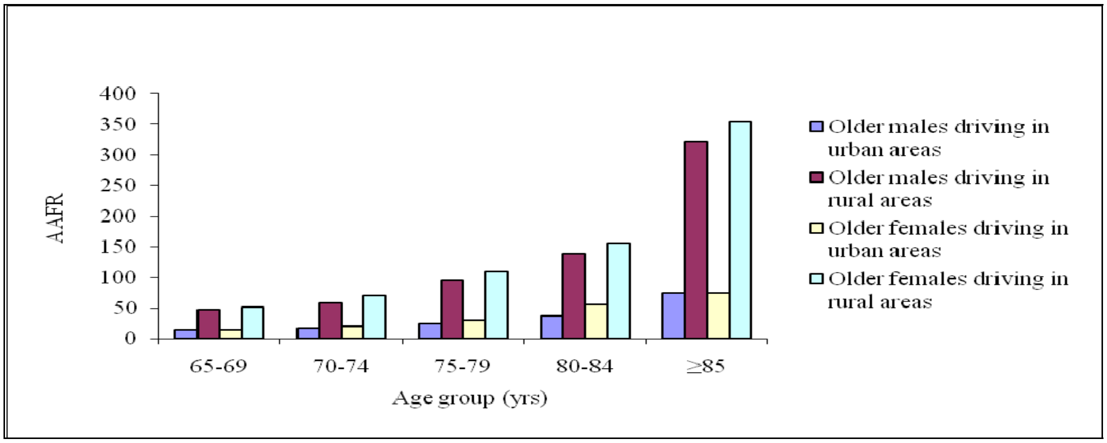

Figure 4. AAFR for older drivers based on road conditions

Vehicle Type

Fatality rate results based on vehicle type is presented in Table 4. Automobiles are the highest involved vehicles of all types in fatal crashes by older drivers. Higher fatality rates in case of automobiles for females might be because of the lesser number of miles driven by them with increase in age. Similar is the case with vans and SUVs. The crash rates in case of heavy trucks and motorcycles are significantly high because of the lesser data entries, i.e., very few older people driving these vehicles. There are no miles driven reported in case of females for heavy trucks for age 75 and older; hence, the rate is reported as 0. Rates are plotted in Figure 5 for automobiles and vans as they are significant compared to all other.

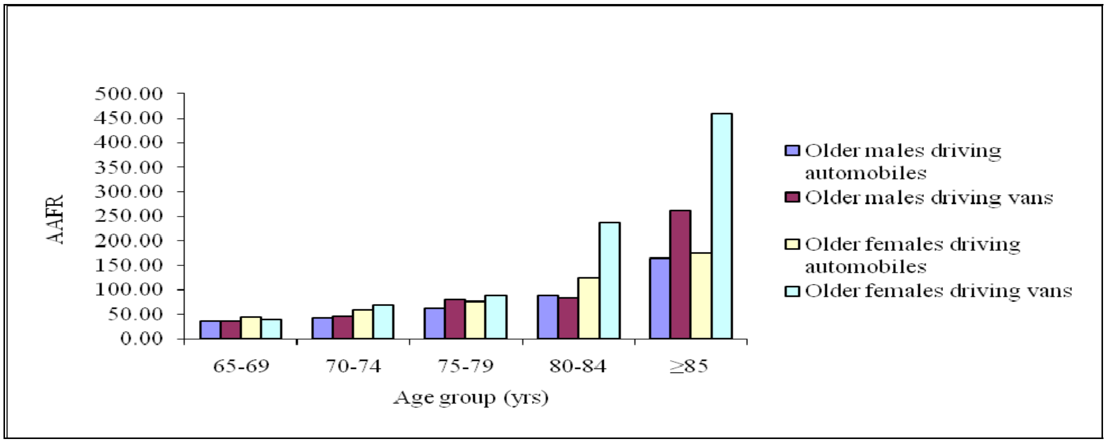

Figure 5. AAFR for older drivers based on vehicle type

Race

Fatality rates based on race are in the following order:

African American > American Indian > American > Asian > Others

Rates for African Americans are higher than Americans because of the lesser number of miles driven reported by them. It is the same with American Indians. Fatality rates for whites, i.e., Americans, show a reasonable increase and they increase with increase in age. Asians and others, which include multiple races, are next in order and rates are very less and insignificant because of their lesser population when compared to the other three categories. Fatality rates based on race are plotted in Figure 6.

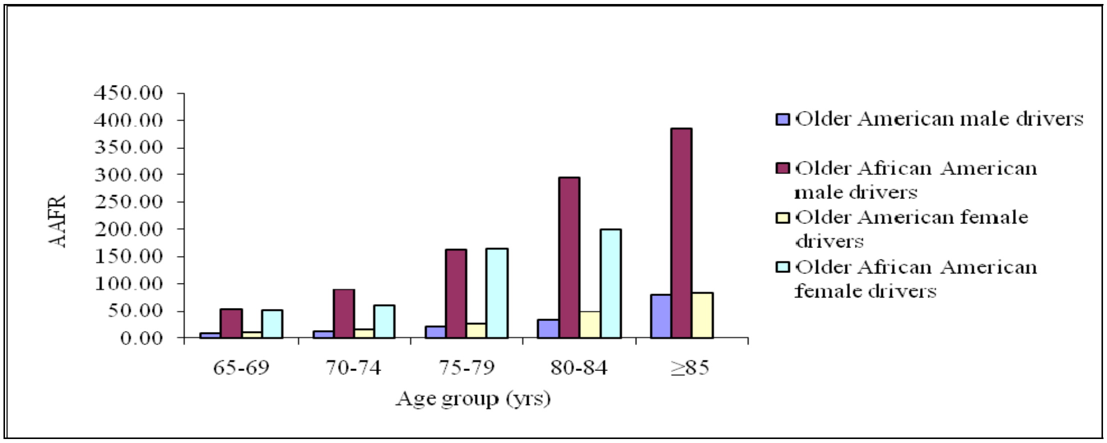

Figure 6. AAFR for older drivers based on race

Table 4. Fatality rates for older drivers based on vehicle type and race for different age groups and gender

Older Drivers

Age Group (yrs)	Based on vehicle type								Based on race				
	Auto	Vans	SUV's	Pickups	Heavy trucks	RV's	Motor cycles	Others	White	African American	American Indian	Asian	Others
65-69	36.6	41.4	59.0	53.2	852.8	8.6	128.1	78.1	10.7	54.4	27.8	2.6	2.2
70-74	49.4	47.5	78.1	62.0	1154.7	14.1	146.8	86.2	14.1	78.9	99.3	5.0	2.6
75-79	74.9	60.9	114.4	87.4	1577.2	14.8	130.8	415.9	22.9	162.1	42.4	4.3	7.4
80-84	99.7	99.7	247.7	203.2	975.6	23.6	478.0	412.0	37.4	258.2	43.8	6.0	8.0
≥85	162.5	286.9	781.5	700.8	13608.1	19.7	591.1	677.7	78.1	446.6	0.0	20.1	4.1

Older Male Drivers

Age Group (yrs)	Based on vehicle type								Based on race				
	Auto	Vans	SUV's	Pickups	Heavy trucks	RV's	Motor cycles	Others	White	African American	American Indian	Asian	Others
65-69	35.9	35.5	46.1	45.9	715.6	7.7	103.4	65.2	10.1	53.8	28.5	2.2	2.1
70-74	42.3	45.9	71.3	67.4	1264.1	15.8	159.3	111.5	13.2	90.4	176.2	5.0	1.8
75-79	61.7	78.6	106.8	103.6	1805.1	15.7	454.0	463.7	21.6	162.3	40.9	3.5	7.1
80-84	88.6	83.6	219.3	181.7	707.8	22.7	364.1	342.7	33.8	295.4	43.3	6.0	6.5
≥85	163.6	261.2	700.5	827.3	10267.0	19.2	483.4	657.2	79.2	384.0	0.0	17.3	2.4

Older Female Drivers

Age Group (yrs)	Based on vehicle type								Based on race				
	Auto	Vans	SUV's	Pickups	Heavy trucks	RV's	Motor cycles	Others	White	African American	American Indian	Asian	Others
65-69	45.0	38.6	74.7	51.3	968.7	7.1	208.0	93.3	11.5	52.9	25.9	4.0	2.5
70-74	58.5	68.2	102.3	77.9	1712.3	7.9	207.4	31.9	16.9	60.3	61.2	4.8	5.1
75-79	76.2	87.9	233.5	146.1	0	6.0	520.7	109.8	26.2	163.4	49.9	5.9	8.6
80-84	124.4	237.1	361.9	372.0	0	30.4	4803.2	0	48.3	199.2	46.1	6.1	11.4
≥85	174.9	459.2	1299.9	485.4	0	0	0	779.7	83.3	0.0	0.0	0.0	8.8

SUMMARY & CONCLUSIONS

This study explored the fatality crash risk of older drivers in relation to few of the selected driver, vehicle, environment, and roadway-related variables. Crash data obtained through NHTSA and exposure data from NHTS were used for this analysis. Time period considered for this study was 10 years, from 1997 to 2006 with midyear corresponding to 2001. Older drivers were divided into five age groups to match the standard grouping followed by NHTSA. They are 65 to 69 years, 70 to 74 years, 75 to 79 years, 80 to 84 years, and 85 years and older. Fatality rates in terms of AAFR were calculated to evaluate the risk for older drivers when involved in fatal crashes. Driver-related factors like age, gender, and race of the driver, vehicle related factors like body type, environment-related like light conditions, and roadway-related like roadway function class were chosen for this analysis. These factors have been chosen based on data availability. Each factor is individually studied for all the age groups based on gender, to identify the effect with increasing age for males and females separately.

Findings of this study show that fatality rates for older drivers increased with increase in age. Number of older male drivers involved in crashes was more than older females, but fatality rates were higher for female older drivers. Based on age, AAFR ranged from 25 fatalities per BVMT to 130 fatalities per BVMT. Fatality rates in dark conditions are higher than daylight for older drivers. Daylight fatality rates show reasonable increase, and this is attributed to more than 90% of driving and fatal crash involvement in daylight conditions. Percentage of elderly driving in dark decrease with increase in age and hence fatality rates are higher for dark conditions. Based on gender, fatality rates are around three times higher for females. Older drivers have high fatality risk in rural areas. Previous studies and governmental reports show that number of crashes that occurred in rural areas was higher than in urban and hence the fatality rates are higher by three times to those in urban areas. Rate of licensed drivers is higher in rural areas by 2%. The value for AAFR ranges from 15.0 to 75.0 fatalities per BVMT in urban areas and 45.0 to 350.0 fatalities per BVMT in rural areas. Range for rates varies from three to five times in rural areas.

Vehicles are categorized into seven major types, and fatality rates are highest for automobiles. Older drivers have a higher percentage of vehicle miles traveled using automobiles. Unlike younger drivers, they have a very small percentage of involvement in fatal crashes using motorcycles. So the crash rates are well explained in the case of automobiles than any other type of vehicle. Vehicle miles traveled for heavy trucks are very insignificant and hence the fatality rates shoot up rapidly. Females have an extravagant increase and a sudden drop to zero in rates for vehicles like heavy trucks and RVs due to their lesser amount of travel in those vehicles.

Based on race of the driver, African Americans have higher rates than Americans which is again an over projection due to their small population and lesser amount of travel correspondingly. Fatality rates increase with increase in age. It ranges from 10.0 to 80.0 crashes per BVMT in case of whites and 50.0 to 500.0 crashes per BVMT in case of African Americans. The range is radical in case of African Americans because of lesser number of observations reported to NHTS.

This study is limited to these five factors because of the non-availability of common variables in crash and exposure data. It can be further extended if data for some of the important variables like travel speed, intersection-related, and junction-related are available in both the databases aiding the rate calculations and evaluation of fatality risk for older drivers based on speed and maneuvers. From the older driver's perspective, with the increased licensed drivers rate and increased involvement in fatal crashes, countermeasures like self regulation of driving behavior, monitoring of their health, regular usage of handbooks, and driver safety programs can improve the safety situation. Research can also be done to find out minute details about other factors responsible for higher involvement of older drivers in rural and daylight conditions.

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Determination of Pre-Treatment Procedure Required for Developing Bio-Binders from Bio-Oils

Mohamed Abdel Raouf
Department of Civil, Construction, and Environmental Engineering
Iowa State University
136 Town Engineering
Ames, IA, 50011
maraouf@iastate.edu

R. Christopher Williams
Department of Civil, Construction, and Environmental Engineering
Iowa State University
136 Town Engineering
Ames, IA, 50011
rwilliam@iastate.edu

ABSTRACT

Most bituminous adhesives or binders that are used for pavement materials are derived mainly from fossil fuels. Due to the availability of large quantities of biorenewable sources such as triglyceride oils, proteins, starch, and other carbohydrates from different organic sources, there are virtuous technical and economic prospects in utilizing them to produce bio-binders. Recently, through the application of scientific research and development, a range of different vegetable oils have been investigated to determine their physical and chemical properties to study their applicability to be used as bio-binders. Bio-binders can be utilized in three different ways to decrease the demand for fossil fuel-based bituminous binders: a direct alternative binder (100% replacement), a bitumen extender (25% to 75% bitumen replacement), or a bitumen modifier (<10% bitumen replacement). In this paper, only the applicability of developing bio-binders from bio-oils to be utilized as a direct alternative (100% replacement) has been investigated.

Recently, according to Superpave specifications and requirements, the design of pavement material should be based upon the pavement distresses. Hence, investigating the rheological properties of a pavement binder is very important in order to determine the pavement distresses and hence, to predict and evaluate the pavement performance. Temperature and the addition of polymer modifiers play major roles in changing the viscosity and the rate of aging of the bio-oils.

The rheological properties, mainly viscosity, of three different types of bio-oils have been investigated to determine the pretreatment temperature and duration required to develop bio-binders. The three different types of bio-oils used are oak flour, switchgrass, and corn stover. In addition, three different polymer modifiers, two polyethylenes, and an oxidized polyethylene, have been blended with the bio-oils to study their effect on the rheological properties of the bio-oils. Notably, the rates of aging and oxidation due to temperature have been studied through measuring the viscosity of the bio-oils during eight hours and then determining the aging index.

Key words: aging index—bio-binders—bio-oils —rheological properties—viscosity

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PROBLEM STATEMENT

The United States, nowadays, is promoting the establishment of a bio-based economy that generates energy from renewable organic matter rather than fossil fuels (Demirbas and Balat 2006). Biofuels have many advantages over fossil fuels, as they are renewable, environmentally friendly, provide energy security, and present a great economic opportunity for the United States (Demirbas and Balat 2006). Bio-fuels are produced from plant matter and residues, such as agricultural crops, municipal wastes, and agricultural and forestry by products (Demirbas and Balat 2006; Mohan et al. 2006). Biomass, which is agricultural and forestry residue, contains a significant amount of carbohydrates, e.g., cellulose, hemicelluloses, and lignin. Therefore, bio-fuels are produced from biomass through biochemical or thermochemical processes. In general, carbohydrates are potential sources for the production of bio-fuels and chemicals (Demirbas 2008). By hydrolysis processes, carbohydrates can be converted to sugars, and then subsequently, through fermentation, such as an anaerobic biological process, sugars are converted to bio-fuels by the action of microorganisms, usually yeast (Demirbas 2008). Depending upon the process of converting plant matter to bio-fuels, different co-products are produced. Some of these co-products are not utilized in any other industry; therefore, more effort should be placed to discover new uses and applications for these co-products. Utilizing the co-products is crucial for the success and profitability of the whole bio-fuels production industry (Bothast and Schlicher 2005; Van Dam and DeKlerk-Engles 2005).

Most bituminous adhesives or binders that are used for pavement materials are derived mainly from fossil fuels. Due to the availability of large quantities of biorenewable sources, such as triglyceride oils, proteins, starch, and other carbohydrates from different organic sources, there are virtuous technical and economic prospects in utilizing them to produce bio-binders. Recently, through the application of scientific research and development, a range of different vegetable oils have been investigated to determine their physical and chemical properties to study their applicability to be used as bio-binders. Bio-binders can be utilized in three different ways to decrease the demand for fossil fuel-based bituminous binders: a direct alternative binder (100% replacement), a bitumen extender (25% to 75% bitumen replacement), or a bitumen modifier (<10% bitumen replacement). In this paper, only the applicability of developing bio-binders from bio-oils to be utilized as a direct alternative (100% replacement) has been investigated.

Recently, according to Superpave specifications and requirements, the design of pavement material should be based upon the pavement distresses. Hence, investigating the rheological properties of a pavement binder is very important in order to determine the pavement distresses and hence, to predict and evaluate the pavement performance. Temperature and the addition of polymer modifiers play major roles in changing the viscosity and the rate of aging of the bio-oils.

RESEARCH OBJECTIVES

Currently, the state of the art for the utilization of bio-oils is concentrated on its uses as biorenewable fuels to replace fossil fuels. However, there are research efforts investigating the applicability of using bio-oils as a bitumen modifier or extender. Based on the conclusion of these investigations, the utilization of bio-oils as a bitumen modifier is very promising. On the other hand, there is no research conducted until now that studies the applicability of the utilization of bio-oils as a bitumen replacement to be used in the pavement industry. As a result, there is scarcity of data that illustrate the procedure to develop bio-binders from bio-oils.

The main objectives of this study are twofold. First, the rheological properties of fast pyrolysis liquid co-products (bio-oils) were investigated. In addition, the effect of polymers on the rheological properties of bio-oils was studied. The rheological properties include the determination of the viscosity-temperature susceptibility “VTS” and aging index (AI) of the bio-oils. Second, the heat pre-treatment procedure required for developing bio-binders from bio-oils was determined. In other words, the second objective was concerned with identifying the temperature and duration for heating the bio-oils in order to be used as a direct alternative binder (100% replacement).

RESEARCH METHODOLOGY

The rheological properties, mainly viscosity, of three different types of bio-oils have been investigated to determine the pre-treatment temperature and duration required to develop bio-binders. The three different types of bio-oils tested were oak flour, switchgrass, and corn stover. In addition, three different polymer modifiers (two polyethylenes and an oxidized polyethylene) were blended with the bio-oils to study their effect on the rheological properties of the bio-oils. Notably, the rates of aging and oxidation due to temperature were studied through measuring the viscosity of the bio-oils during eight hours and then determining the AI. The experimental program of this study was designed to investigate the applicability of developing bio-binders from them to be used in the pavement industry. The experimental materials used, the experimental plans designed, and the experimental procedures followed during testing will be discussed in the ensuing sections.

Experimental Materials

Bio-Oils

By definition, bio-oils can be described as dark brown, free-flowing organic liquids that are comprised mainly of highly oxygenated compounds (Mohan et al. 2006; Oasmaa et al. 1999). In other words, it is the liquid produced from the rapid heating of biomass in vacuum conditions (Oasmaa et al. 2005). Bio-oils have many synonyms that can be listed as the following: pyrolysis oil, pyrolysis liquid, bio-crude oil (BCO), wood liquid, wood oil, liquid smoke, wood distillates, and pyroligneous acid (Mohan et al. 2006; Oasmaa et al. 2005). Since the oil crisis in the mid 1970s, considerable effort has been directed toward the development of processes for producing liquid fuels from biomass. According to Oasmaa et al. (1999), one of the most efficient methods for such conversion is pyrolysis. In this research, the bio-oils were extracted from different biomass materials using an existing 25 kWt fast pyrolysis system developed at Iowa State University by the Center for Sustainable Environmental Technology “CSET,” shown in Figure 1. The different biomass feedstocks were oak flour, switchgrass, and corn stover. The pilot unit consists of a 16.2 cm diameter fluidized bed reactor, a burner to externally heat the reactor, a two-stage auger to feed the solid, two cyclones to remove particulate matter, and a vapor-condensing system consisting of four condensers and an electrostatic precipitator. The system can process 6 to 10 kg/h of solid feed.

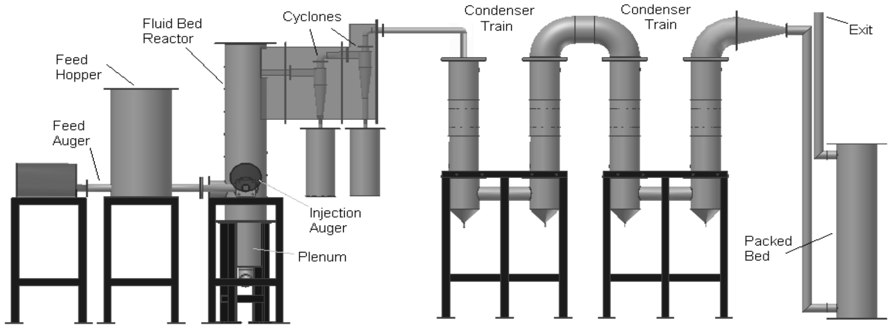

Figure 1. Schematic diagram of the 25 kWt fast pyrolysis reactor with staged condensation unit at CSET

The separation of bio-oils into multiple fractions was conducted using a fractionation condenser system, which facilitated the selection of bio-oil fractions that would be optimal for being used as a pavement binder. As an example, Table 1 shows the properties of bio-oil fractions collected from fast pyrolysis of corn stover. It can be seen that these bio-oil fractions have significantly different properties, especially in water and lignin contents. Bio-oil fractions collected from condensers #1 and #2 and ESP have high lignin content and low water content, which make them most suitable for using as a pavement binder.

Table 1. Properties of bio-oils fractions collected from fast pyrolysis of corn stover

Property	Cond. 1	Cond. 2	Cond. 3	Cond. 4	ESP
Fraction of total oil (wt%)	6	22	37	15	20
pH	-	3.5	2.7	2.5	3.3
Viscosity @40°C (cSt)	Solid	149	2.2	2.6	543
Lignin Content (wt%)	High	32	5.0	2.6	50
Water Content (wt%)	Low	9.3	46	46	3.3
C/H/O Molar Ratio	1/1.2/0.5	1/1.6/0.6	1/2.5/2	1/2.5/1.5	1/1.5/0.5

Polymer Modifiers

Since the early 1970s, the utilization of petroleum-derived polymers has been well developed to be blended with conventional bituminous binders to modify the performance and rheological properties by decreasing temperature susceptibility and increasing cohesion as reported by Airey et al. (2008). In other words, the practical experience has shown that the blending of bitumen binders with polymer modifiers (e.g., polyethylenes) has many advantages that include, but are not limited to, enhanced fatigue resistance, improved thermal stress cracking, decreased temperature susceptibility, and reduced rutting (Gonzalez et al. 2006). In this bio-asphalt study, three types of polyethylene were used, and their properties can be summarized in Table 2. The three polymer modifiers used can be classified as thermoplastics.

Table 2. Properties of polymer modifiers used

Property	Polyethylene 617 (P1)	Oxidized Polyethylene 680 (P2)	Polyethylene 9 (P3)
Drop Point, Mettler (°C)	101	108	115
Density (g/cc)	0.91	0.93	0.93
Viscosity @140°C (cps)	180	250	450
Bulk Density (kg/m^3)	563	536	508

Experimental Plan

The experimental plan was designed in order to determine the pretreatment temperature required for developing bio-based binders for use in flexible pavements. The experimental plan was concerned about determining the AI and the temperature susceptibility of the bio-binders before and after heat pretreatment. In addition, the experimental plan emphasized the effect of the blended polymer modifiers on the overall AI and temperature susceptibility. Twenty-one blends were tested with different bio-oils type, type of polymer modifiers, blending percentages, and experimental temperature. The bio-oils types were oak flour, switchgrass, and corn stover. The polymer modifiers types were P1, P2, and P3 as previously described. The blending percentages were (1) the control (100% bio-oil), (2) 98% bio-oil + 2% polymer modifier, and (3) 96% bio-oil +4% polymer modifier.

Aging Index

It is well agreed that the rheological properties of any binder can affect its pavement performance. The rheological properties change during the binder production and subsequently in service. Since bio-oils are chemically organic, they react with oxygen from the environment, and this kind of reaction is called "oxidation," which can change the structure and the composition of the bio-oil. Oxidation can cause the material to become more brittle (stiffer), which leads to the term oxidative or age hardening. The rate of oxidation increases rapidly at high temperatures. On the other hand, oxidative hardening or aging occurs at a slower rate in a pavement, but this rate increases in warmer climates. Age hardening is considered to be one of the most important factors that leads to the change in the rheological properties.

There are many factors that contribute to age hardening of binders, such as oxidation, volatilization, and polymerization, as stated by Roberts et al. (1996). According to Mohan et al. (2006), the viscosity of bio-oils increases due to the aging effect. Temperature is the most driving variable that leads to the aging effect, and hence, the change in viscosity of the bio-oils. The amount of aging that occurs in a binder during production and in service can be quantified in terms of viscosity as the aging index (AI), as shown in equation (1) (Roberts et al. 1996). This AI has been employed to evaluate the relative aging of asphalt cements of different grades and/or from different sources. According to Roberts et al. (1996), there is no threshold or limiting value for the AI, but most of the bitumen binders widely used in the United States have an AI below 12. Therefore, this number was used in this study as the limiting value for AI to compare the effect of aging and hardening on the viscosity of the bio-oils and bitumen binders.

$$\text{AI} = \frac{\text{Viscosity of Aged Binder}}{\text{Viscosity of original Binder}} \tag{1}$$

Temperature Susceptibility

Temperature susceptibility, as defined by Roberts et al. (1996), is the rate at which the consistency of a binder changes with a change in temperature. The temperature susceptibility of a binder is a very crucial property as binders having high susceptibility to temperature are not desired. For a number of years, asphalt technicians have employed the VTS method of binder temperature susceptibility classification (Rasmussen et al. 2002; Roberts et al. 1996). Even though it has not been a common index value used for evaluating temperature susceptibility of binders, it does inherently possess a simple formulation (as shown in equation [2]).

$$\mathrm{VTS} = \frac{\log[\log(\eta_{T_2})] - \log[\log(\eta_{T_1})]}{\log(T_2) - \log(T_1)}, \tag{2}$$

where T_1 and T_2 = temperatures of binders at known points (R = degrees Rankine) and η_{T1} and η_{T2} = viscosities of the binder at the same known points (cp).

The temperature susceptibility of the binder can be characterized using VTS. Based on the literature review conducted, Rasmussen et al. (2002) reported a simple method to predict the parameters for a binder based on conventional test results. A least squares fit is employed between log-log viscosity and log temperature to determine the ‘‘best’’ VTS and AI values to use to classify the binder (Rasmussen et al. 2002; Roberts et al. 1996). The larger the magnitude of the VTS value is calculated to be, the more susceptible the binder is to changes in viscosity with temperature. As a reference, Puzinauskas (1967) calculated the VTS values for over 50 binders commonly used in the United States at that time and concluded that the VTS values were ranging from 3.36 to 3.98 based on the aforementioned equation (Rasmussen et al. 2002; Puzinauskas 1967). Although there is no standard limit or specified range for the VTS of bitumen binders, the VTS values of the most widely used bitumen binders in the United States are considered to be the limiting values or the specified range. Subsequently, in this study, the VTS of the bio-oils were compared to VTS of bitumen binders.

Experimental Procedure

The experimental procedure consisted of blending the bio-oils and polymer modifiers and then measuring the viscosity of the different bio-oils and polymer modifiers bio-oils. Initially, the polymer modifiers were blended with the bio-oils using a high-speed sheer mill. The bio-oils were heated to 110°C before the shear mill was started. Once the bio-oil reached temperature, the shear mill was set to approximately 6,000 rotations per minute for 30 minutes. Once the bio-oil and polymer modifier were thoroughly blended, the blends immediately underwent viscosity testing. The viscosity testing was used to determine the flow characteristics of the bio-oils. The data acquired by rotational viscometer were used to determine the temperature and duration required for pretreatment, to evaluate and quantify the amount of oxidation and aging that occurred, and to determine the VTS. The rotational viscometer procedure was based on ASTM D 4402 (2006) with some deviations that can be summarized as the following:

- 30 grams of bio-oil were heated in an oven until sufficiently fluid to pour.
- The sample was stirred during heating to remove trapped air.
- 8 or 11 grams were used typically according to the size of spindle.
- The temperature was kept constant.
- The motor was set to operate at 100 rpm.
- The viscosity reading and the percent torque should be between 2% and 98%. If the percent torque was out of the range, the size of the spindle should be changed.

- The five readings required for the report were viscosity, test temperature, spindle number, speed, and percent torque.
- Three viscosity readings were recorded at one-minute intervals, and the reported value was the average of them
- The viscosity readings were recorded at 0, 30, 60, 120, 240, and 480 minutes at two different temperatures of 125° and 135°C

RESULTS AND ANALYSIS

Table 3 lists the viscosity measurements for all blends over the 8 hours. Figure 2 to Figure 5 display the viscosity over time for all blends at different temperatures.

Table 3. Measurements of viscosity testing over time for all blends

Blend #	Bio-oil Type	Modifier Type	Blending (%)	Temp. (°C)	Time (hrs)/Viscosity (Pa·s)					
					0	0.5	1	2	4	8
1	OF	Control		135	0.03	0.06	0.10	0.16	0.34	0.98
2		P1	2		0.12	0.16	0.18	0.23	0.43	1.06
3			4		0.05	0.07	0.10	0.17	0.32	0.76
4		P2	2		0.05	0.08	0.11	0.18	0.43	1.25
5			4		0.13	0.19	0.25	0.39	0.75	1.58
6		P3	2		0.18	0.38	0.46	0.64	1.00	1.72
7		Control		125	0.03	0.04	0.05	0.09	0.15	0.45
8		P1	2		0.08	0.12	0.14	0.12	0.21	0.50
9			4		0.05	0.07	0.08	0.11	0.19	0.44
10		P2	2		0.05	0.06	0.07	0.10	0.16	0.51
11			4		0.04	0.06	0.09	0.14	0.25	0.75
12		P3	2		0.05	0.08	0.12	0.21	0.48	1.06
13	SG	Control		145	0.05	0.05	0.06	0.09	-	-
14		Control		135	0.01	0.03	0.04	0.06	0.12	0.24
15		P1	2		0.05	0.07	0.10	0.15	0.30	0.65
16			4		0.09	0.08	0.09	0.15	0.32	1.07
17		Control		125	0.02	0.04	0.06	0.08	0.15	0.35
18		P1	2		0.08	0.09	0.10	0.14	0.23	0.46
19			4		0.07	0.08	0.10	0.14	0.24	0.52
20	CS	Control		135	0.19	0.26	0.30	0.50	1.08	5.71
21		Control		125	0.17	0.21	0.21	0.22	0.33	1.15

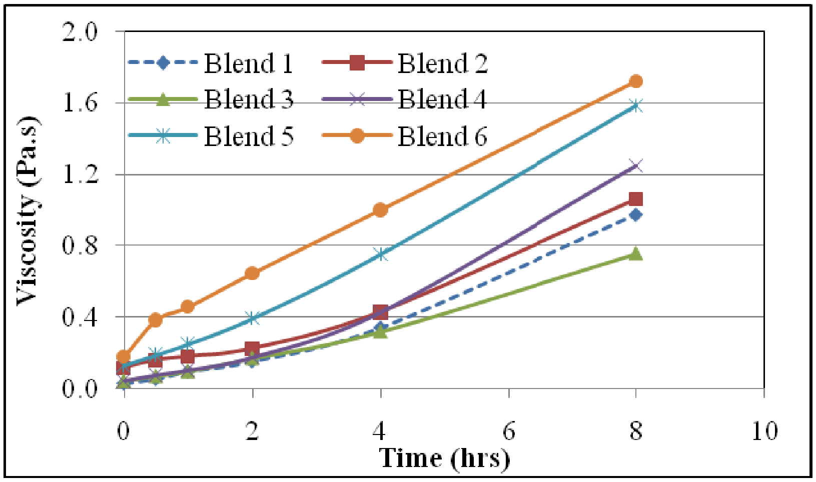

Figure 2. Viscosity over time for all oak flour blends at 135°C

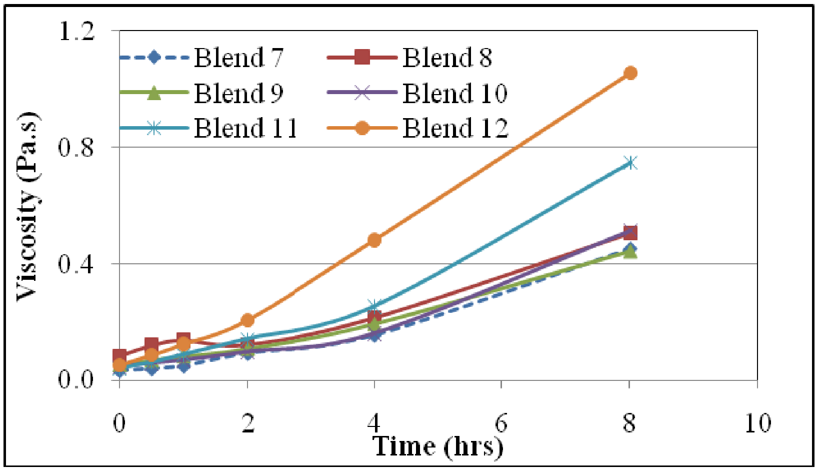

Figure 3. Viscosity over time for all oak flour blends at 125°C

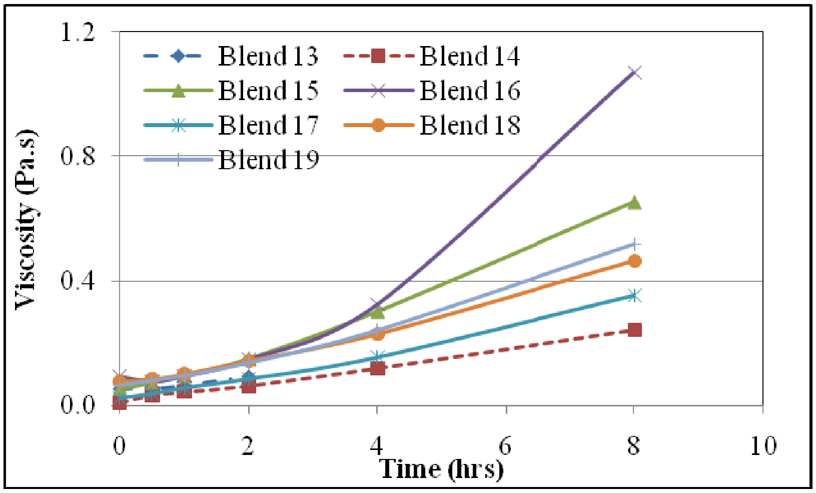

Figure 4. Viscosity over time for all switchgrass blends at 145°C, 135°C, and 125°C

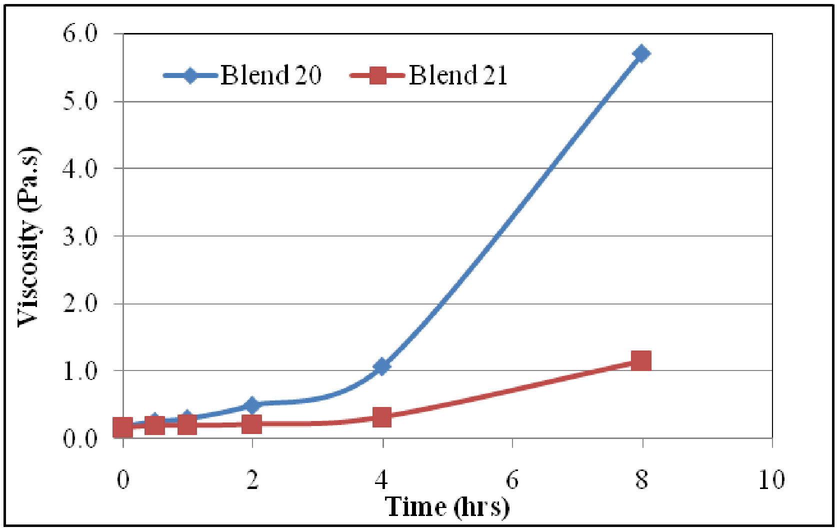

Figure 5. Viscosity over time for corn stover blends 135°C and 125°C

Based on Figures 2 through 5, the following observations are noted. First, some viscosity measurements at the first two hours were almost zero due to the presence of water and volatile materials. Second, the rates of change of viscosity over time for most of the blends were not constant. In other words, the rate of change of viscosity at the first two hours was different than the rate of change of viscosity between two and eight hours. During the first two hours, significant amount of evaporation and boiling took place due to the water and volatile materials. This may be the reason that the rate of change of viscosity during the first two hours was less than the rate of change of viscosity between two and eight hours. Third, the addition of the polymer modifier led to a significant increase in the viscosity of the bio-oils (e.g., oak flour and switchgrass). However, no specific optimum content for polymer modifiers could be determined. Fourth, increasing the testing temperature led to an increase in the viscosity of all blends, and this increase became significant after the first two hours. However, when the testing temperature increased to 145°C (Blend 13), the viscosity measurements could not be measured due to the high rate of evaporation and boiling that led to substantial sample loss. Fifth, since the bio-oils tested had a high content of water and volatile materials, the pretreatment temperature could be considered to be between 100°C and 110°C, which is the temperature required for the evaporation of water. Importantly, the pretreatment temperature should be below the decomposition temperature of the chemical constituents of bio-oils (cellulose, hemicellulose, and lignin). Sixth, a pretreatment duration could be considered to be a two-hour period. From the results, it was noted that the aging and hardening of bio-oils after two hours were high, so the pretreatment duration should be less than two hours because developing a bio-binder, initially, having a high viscosity may lead to mixing and pavement performance problems. Importantly, the viscosity of the bio-oils after two-hours heating were below the viscosity specified by the Superpave at 140°C, which is 3 Pa·s.

From Table 3, the AIs for all blends were calculated according to equation (1) and listed in Table 4. The AIs were determined relative to the zero hour, which represented at the beginning of the test, and relative to two hours, which represented after the pretreatment temperature. From AIs relative to the zero hour, the following conclusions could be established. First, the AIs after four hours were below 12 (the threshold value of bitumen binders) for all blends (except Blend 14). In addition, the AIs after eight hours were ranging between 6.0 and 32.0, and most of the blends were exceeding 12. However, when the AIs were determined relative to two hours, they were below 12 for all blends. Therefore, it can be concluded that after the pretreatment procedure, the AIs of the bio-oils were below the limiting value (12), so they were acceptable and comparable to bitumen binders.

Table 4. AI relative to zero and two hours

Blend #	AI relative to zero					AI relative to two	
	0.5	1	2	4	8	4	8
1	1.76	3.08	4.89	10.84	30.84	2.22	6.30
2	1.34	1.51	1.87	3.58	8.80	1.91	4.70
3	1.57	2.17	3.80	7.11	16.80	1.87	4.42
4	1.67	2.25	3.75	9.05	26.25	2.41	6.99
5	1.44	1.90	2.97	5.69	11.95	1.91	4.02
6	2.13	2.54	3.58	5.56	9.54	1.55	2.67
7	1.21	1.53	2.89	4.87	14.32	1.68	4.95
8	1.44	1.67	1.48	2.63	6.23	1.77	4.19
9	1.31	1.58	2.08	3.74	8.60	1.80	4.13
10	1.28	1.52	2.15	3.56	11.41	1.66	5.31
11	1.50	2.17	3.52	6.31	18.65	1.79	5.30
12	1.66	2.41	4.03	9.46	20.75	2.35	5.15
13	1.00	1.19	1.78	-	-	-	-
14	4.33	5.67	8.33	15.78	32.00	1.89	3.84
15	1.39	1.84	2.88	5.82	12.63	2.02	4.38
16	0.83	1.01	1.59	3.50	11.54	2.20	7.28
17	1.63	2.44	3.74	6.81	15.63	1.82	4.18
18	1.09	1.29	1.83	2.93	5.90	1.60	3.23
19	1.23	1.47	2.10	3.71	7.94	1.76	3.77
20	1.38	1.62	2.67	5.76	30.60	2.16	11.48
21	1.19	1.21	1.29	1.90	6.65	1.47	5.14

From Table 3, the VTS after the two hours for all blends were determined based on equation (2) and listed in Table 5. The VTS were not determined at the beginning of the test (zero hour) because the viscosity measurements of many blends were almost zero due to the high content of water and volatile materials. Therefore, in this study, the VTS was determined based on the viscosity measurements after two hours. The VTS was calculated between the same blends but tested at different temperatures. For instance, Blends 1 and 7 were the same (oak flour and no polymer modifier) but were tested at 135°C and 125°C, respectively. In addition, Blends 2 and 8 were the same (oak flour, polymer modifier P1, and blending percentage of 2%), but tested at different temperatures. From Table 5, the following observations were noted. First, the VTS ranged between 0.34 and 8.46. Some blends (e.g., 15 and 18 and 16 and 19) were having almost zero VTS, which indicated that the viscosity measurements were not increasing significantly when the temperature changed from 125°C and 135°C. Second, the VTS for some blends were almost higher than 5.3, which indicated that the viscosity measurements were increasing significantly when the temperature changed (e.g., Blends 4 and 10 and Blends 5 and 11). In this study, since there is no accepted range for VTS of bitumen binders, the accepted range was considered to be between 3.36 and 3.98, which is the measured VTS range for over 50 bitumen binders commonly used in the United States. It was expected that the VTS values for the bio-oils to be higher than the VTS values of bitumen binders due to the presence of high content of water and volatile materials. Third, some of the blends (e.g., 1 and 7 and 3 and 9) were having acceptable VTS values that were comparable to VTS of bitumen binders; thus, they can be used as pavement binders.

Table 5. VTS for different blends

Blend #	Viscosity (cP) T_2	Log Log Viscosity	Log T (°K)	VTS
1	155.00	0.34	2.61	4.78
7	91.67	0.29	2.60	
2	225.83	0.37	2.61	5.39
8	120.00	0.32	2.60	
3	170.83	0.35	2.61	4.10
9	107.50	0.31	2.60	
4	178.33	0.35	2.61	5.46
10	96.67	0.30	2.60	
5	394.17	0.41	2.61	8.21
11	140.83	0.33	2.60	
6	644.17	0.45	2.61	8.46
12	205.00	0.36	2.60	
13	93.33	0.29	2.62	4.02
14	62.50	0.25	2.61	
13	93.33	0.29	2.62	0.50
17	84.17	0.28	2.60	
14	62.50	0.25	2.61	3.02
17	84.17	0.28	2.60	
15	149.00	0.34	2.61	0.34
18	143.33	0.33	2.60	
16	146.67	0.34	2.61	0.62
19	136.67	0.33	2.60	
20	497.67	0.43	2.61	5.97
21	224.17	0.37	2.60	

Statistical Analysis

A statistical analysis was conducted using the computer software JMP 7.0 to study the significant factors that affect the viscosity and the AI. An analysis of variance (ANOVA) using the method of least squares was used for examination to evaluate the bio-oil type, modifier type, modifier percentage, and temperature. Type I error (α) of 0.05 was used for all statistical analyses as the confidence level was 95%. An α of 0.05 states that there is a five percent chance of rejecting the null hypothesis when it is in fact true. Since the p-value is less than 0.05, then this factor is significant. Table 6 summarizes the p-values for the ANOVA of the viscosity results. The ANOVA was conducted for all the viscosity readings at the six different times (0, 0.5, 1, 2, 4, and 8). Based on the results, it could be concluded that the bio-oil type was the main significant factor to affect the viscosity during the first hour. However, after the two hours, the viscosity results of the bio-oil were affected by the bio-oil type, the modifier type, and the temperature.

Table 6. P-Values for ANOVA of the viscosity results

Source	Prob>F					
	V0	**V0.5**	**V1**	**V2**	**V4**	**V8**
Bio-oil Type	***0.0009***	***0.0149***	***0.0290***	***0.0247***	***0.0103***	***0.0050***
Modifier Type	0.3596	0.1021	0.0592	***0.0215***	***0.0238***	0.6430
Modifier %	1.0000	0.8955	0.9691	0.5975	0.5907	0.8125
Temperature	0.1246	0.0913	0.0505	***0.0113***	***0.0029***	***0.0333***

Table 7 summarizes the p-values of the ANOVA of the AIs. The AI 0.5 to AI 8 represent the AIs relative to the zero hour (before the pretreatment), while AI 4-2 and AI 8-2 represent the AIs relative to the two hours period (after the pretreatment). From the results, it could be noted that the AIs before the pretreatment was affected significantly by the temperature. On the other hand, after the pretreatment, the AIs were not affected by any of these factors, which indicated that the temperature was not a significant factor.

Table 7. P-Values for ANOVA of the AI

Source	Prob>F						
	AI 0.5	**AI 1**	**AI 2**	**AI 4**	**AI 8**	**AI 4-2**	**AI 8-2**
Bio-oil Type	0.2310	0.0738	***0.0302***	0.0590	0.7541	0.9385	0.0989
Modifier Type	0.4490	0.3203	0.1363	0.1762	0.3136	0.9009	0.6274
Modifier %	0.8892	0.9406	0.6117	0.7823	0.8624	0.9683	0.9842
Temperature	0.1119	***0.0477***	***0.0185***	***0.0365***	***0.0329***	0.0795	0.1870

KEY FINDINGS AND CONCLUSIONS

Based on the results of this study, the established conclusions could be listed as the following:

- The bio-oils could not be used as a direct alternative binder (100% replacement) in the pavement industry due to the presence of water and volatile materials.
- The bio-oils require pretreatment step, which includes heating at a specific temperature for a specific duration. The pretreatment temperature and duration should be determined according to the bio-oil type and its water and volatile materials contents.
- For the different bio-oils tested in this study, the pretreatment temperature and duration were established to be 110°C and two hours, respectively.
- The viscosity of the developed bio-binder (after pretreatment of bio-oils) were complying with the Superpave specification which reported that the initial viscosity of the bitumen binders should be below 3 Pa·s at 140°C.
- The AIs of the untreated bio-oils were higher than the AI of the bitumen binders. However, after the pretreatment, the AIs of the treated bio-oils were comparable to bitumen binders as they were below 12.
- The VTS range of the bio-oils tested was found to be wider than the VTS of the bitumen binders. This is may be due to the high water and volatile materials contents present in the bio-oils in comparison with bitumen binders.
- From the statistical analysis of the viscosity, it was found that the bio-oil type was the main significant factor to affect the viscosity during the first hour. However, after the two hours, the

viscosity results of the bio-oil were affected by the bio-oil type, the modifier type and the temperature.

- From the statistical analysis of the AIs, it was noted that the AIs before the pretreatment was affected significantly by the temperature. On the other hand, after the pretreatment, the AIs were not affected by any of these factors, which indicated that the temperature became no longer a significant factor.

RECOMMENDATIONS

More research effort should be conducted to study the applicability of using bio-oils as a direct alternative binder (100% replacement) in the pavement industry using different sources of biomass. It is recommended that the pretreatment temperature and duration to be specified based on testing wide variety of bio-oils derived from different biomass. In addition, the viscoelastic behavior of the bio-oils should be studied extensively and determine their resemblance with the asphalt behavior. Moreover, the temperature and shear dependence of the viscosity of the bio-oils should be studied and compared with the asphalt binders behaviors.

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Review of Congestion Pricing Established Systems in Europe and Asia: Evaluation of Success

Aikaterini "Catherine" Rentziou
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
rentziou@iastate.edu

Konstantina "Nadia" Gkritza
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
nadia@iastate.edu

ABSTRACT

Congestion pricing consists of a widespread method of traffic management, especially in areas with major traffic problems. The first system of congestion pricing was established in Singapore in 1975, and since then, various cities have established similar systems in order to reduce traffic congestion. Bergen, in Norway, was the first European city to establish a system of congestion pricing in 1986, and a few years later, Oslo and Trondheim followed. The most recent examples of cities with system of congestion pricing are London and Stockholm.

The major aim of all these systems is to reduce traffic congestion in the central area of cities and to improve the daily trips by decreasing time of travel and delays. Meanwhile, congestion pricing systems target the improvement of people's lives in metropolitan areas by improving public transportation through investments on this sector and by reducing air pollution.

Although the results from the already established congestion pricing systems are encouraging about the success of such systems, many transportation engineers are still critical of this type of traffic management. Congestion pricing is considered as the last step of traffic management, following other less-strict strategies (or policies) such as increases on tax fuels, promotion of public transportation, construction of bike routes, or building of new infrastructure.

The aim of this study is to present the already established cordon systems of congestion pricing and to evaluate their results. The systems are presented in terms of their characteristics, the way of establishing them, and their goals. The success of those systems is measured by how well they achieved their goals. Measures of evaluation of congestion pricing schemes include the percentage increase in vehicle speeds during peak hours, the percentage of people who were encouraged to change their mode of transportation due to congestion pricing, and the decrease in air pollution. As more and more cities around the world face considerable problems due to traffic congestion, understanding and learning from the already established congestion pricing systems and their results is vital. In particular, various cities in the United States (such as Chicago, Los Angeles, or New York) are looking into establishing cordon systems of congestion pricing instead of (or supplemental to) high-occupancy vehicle lanes. The results of this study

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can be beneficial in the progressive establishment of similar systems in the United States and can assist transportation agencies and policymakers select the most appropriate pricing scheme for their jurisdiction.

Key words: congestion pricing—cordon system—system evaluation

Predicting and Mitigating Wind-Induced Truck Crashes on I-70 in Kansas

Robert Rescot
University of Kansas
1530 West 15th Street, Suite 2160
Lawrence, KS 66045
rrescot@ku.edu

Yue Li
University of Kansas
1530 West 15th Street, Suite 2160
Lawrence, KS 66045
ylkx7@ku.edu

Romika Jasrotia
University of Kansas
1530 West 15th Street, Suite 2160
Lawrence, KS 66045
romikajasrotia@ku.edu

Steven D. Schrock
University of Kansas
1530 West 15th Street, Room 2150
Lawrence, KS 66045
schrock@ku.edu

Kelly Hovey
Kansas Department of Transportation
700 Southwest Harrison Street
Topeka, KS 66603
kellyhov@ksdot.org

ABSTRACT

Dangerous weather conditions are a common contributing factor in truck crashes and one such condition is high wind. It is believed that frequent high-wind episodes cause several truck crashes every year in Kansas. This paper presents a literature review and an analysis of wind-related crashes. The data compiled include truck crashes on I-70 throughout Kansas over a three year period from 2005 to 2007 along with independent weather data. The crash data were obtained for all heavy-vehicle crashes on I-70 that involved strong winds. The data were analyzed to determine the correlations between the vehicle characteristics, crash occurrences, and weather conditions. The goal was to construct a model that can predict the likelihood of such wind induced truck crashes, thus providing a tool for increasing safety to both truck drivers and the traveling public.

Key words: crash—truck—wind

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INTRODUCTION

Traffic in the state of Kansas is susceptible to frequent severe wind conditions that contribute to several crashes. A significant portion of these traffic crashes involve freight trucks or high-profile vehicles. Statistics show that in Kansas during the years 2005, 2006, and 2007, there were 1,147, 892 and 1,399 wind-related crashes, respectively (1). These crashes contributed to 28%, 27%, and 37% of the total crashes in Kansas in the three respective years (2) (3) (4). Kansas is served by two major Interstate highways—I-70 and I-35—and multiple connector interstate routes—I-135, I-335, I-435, and I-635—covering over 874 miles (1,407 km) in all. Interstate 70 extends from the western border to the eastern border, covering 424 miles (682 km) and passing through many of the state's principal cities (5). The I-70 corridor carries an average annual daily traffic (AADT) of 7,990 to 20,300; the AADT increases from the western border to Topeka. These daily counts consist of 2,990 and 4,100 heavy commercial vehicles (6). I-35 extends from Kansas City through Wichita into Oklahoma covering 235 miles (5). I-35 south of Kansas City carries 21,400 vehicles daily, of which 4,960 are classified as heavy commercial vehicles, and I-35 East of Emporia carries 13,500 vehicles daily, of which 4,260 are classified as heavy commercial vehicles (6).

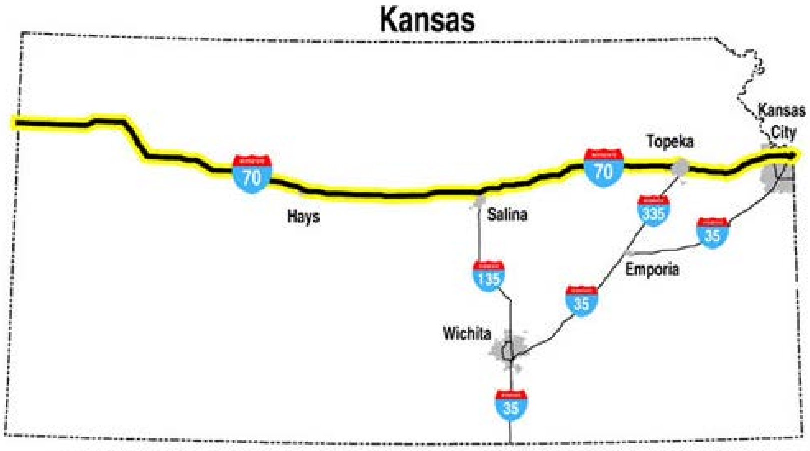

Figure 1. I-70 in Kansas (5)

The western and central parts of the state are more prone to high winds. On March 23, 2009, the Kansas Highway Patrol reported 13 vehicles blown over from high winds in the central and western part of the state. The same day, one semi rollover in Topeka forced cleanup crews to close part of I-70 (7). The combination of large truck volume and high wind speeds leads to a high probability of crashes. These crashes may cause interstate closures, creating significant delays and economic loss. Safety has to be considered not only by the trucking industry but also by engineers; thus, this research looks at the crash data compiled from the Kansas Department of Transportation (KDOT) to determine correlation between wind speed and truck accidents for I-70. Interstate 70 was selected for this detailed analysis as KDOT is actively engaged in a program to deploy dynamic message signs along I-70 between Topeka and the Colorado border (8). Therefore, at the outset of this project, the research team aimed to create a multivariate model to predict wind induced truck crash rates that could be used in conjunction with the dynamic message signs.

LITERATURE REVIEW

The effect of wind on the stability of high-profile vehicles is an important safety consideration as strong winds may lead to the overturning of trucks.

A study was conducted at the University of New Brunswick to investigate the impact of wind forces on heavy truck stability. The goal of the experiment was to calculate the rollover threshold of a truck driving a loop ramp in New Brunswick under varying wind speeds. During the study period, the wind speeds were not significant enough to allow for the calculation of a rollover threshold; however, they were able to determine that wind does affect the stability of trucks. The recommendation was made to use similar methods in high wind areas to investigate wind forces which induce truck rollovers (9). Another study by the University of Manitoba used computer simulation to determine how combinations of weather conditions affect truck traffic. The angle the trailers moved, or yaw angle, was used to measure instability. The program was set up to simulate life-like wind gusts. The study determined that the maximum wind speed a heavy truck with a 48 ft trailer could safely travel in was between 31 and 43 mph when empty and between 62 and 74 mph when loaded (10).

The University of Wyoming suggests using Intelligent Transportation Systems (ITS) technology to communicate weather advisories directly to travelers. Four different mitigation levels were identified. Level 1 operation uses Road Weather Information System (RWIS) data and Dynamic Message Signs (DMS). Level 2 operation uses the same technology as Level 1, but instead of just imposing advisory warnings, a wind and surface condition threshold would be adopted that would lead to roadway closures. Level 3 requires the same decision tree as Levels 1 and 2, with the addition of selective closures to high-profile vehicles on the basis of wind and surface condition thresholds. Level 4 operations take the Level 3 application an additional step by identifying vehicles for partial closure on the basis of vehicle classification, or height and length characteristics, as well as weight characteristics (11).

The September 2007 American Public Works Association *Reporter* newsletter discussed a few other ways to efficiently communicate with drivers. The article focused on Dynamic Message Signs (DMS) and AM/FM radio, emphasizing AM/FM radio usage, considering it more advantageous than any other technology and believing that it has the ability to reach the majority of motorists. Also, the motorist has a better chance to receive the entire message as signs are easily missed, but the radio will continue to play. The report mentions other popular technologies that are being considered, such as cell phones, navigations systems, and satellite radio. Global Positioning System (GPS) and Highway Advisory Radio (HAR) look very promising because of their unique system to broadcast messages throughout a large area, such as an extended roadway,or an entire city or county (12).

STATE OF THE PRACTICE

There are several states that are currently taking a proactive approach to reduce wind-induced truck crashes. The states of Nevada, Montana, and California utilize Environmental Sensor Stations (ESS) installed on the highway or freeways to collect and transmit environmental data to a central control computer in the Traffic Operations Center/Traffic Management Center (TMC). The ESS measures wind speed and direction, precipitation type and rate, air temperature and humidity, as well as pavement temperature and condition (i.e., wet, snow or ice). During high-wind conditions, advisory or regulatory messages are displayed on DMS (13). Additionally, the state of Oregon also operates a similar system.

The Nevada Department of Transportation operates a high wind warning system on a seven mile section of US Route 395. An ESS is installed on the highway. During high-wind conditions, advisory or regulatory messages are displayed on DMS. If the average speeds are 15 to30 mph or the maximum wind gust is over 20 mph, the computer displays “High-Profile Vehicles ‘NOT ADVISED’” on the DMS and for extreme wind conditions when average wind speeds are greater than 30 mph or wind gusts exceed 40 mph. DMS displays “High-Profile Vehicles ‘PROHIBITED.’” Traffic managers may also broadcast pre-recorded messages via three HAR transmitters in the area (13).

The Montana Department of Transportation also uses a similar system to warn motorists and manage vehicle access. Severe wind tunnel conditions pose a safety risk to high-profile vehicles traveling on a 27-mile section of freeway. Traffic and maintenance managers are alerted by the RWIS when wind speeds in the area exceed 20 mph. A warning message—“CAUTION: WATCH FOR SEVERE CROSSWINDS”—is displayed on DMS when wind speeds are between 20 and 39 mph. When severe crosswinds (i.e., over 39 mph) are detected, a restriction message is posted on DMS to direct specified vehicles to exit the freeway and take an alternate route. A typical restriction message reads “SEVERE CROSSWINDS: HIGH PROFILE UNITS EXIT” (13).

Similarly, the California Department of Transportation collects the data from 36 vehicle detection sites and nine ESS that are deployed along freeways. The DMS displays “HIGH-WIND WARNING” if the wind speeds are greater than 35 mph (13).

Oregon operates two wind advisory systems. One system is for the southwest coast of the state and the other system is for the Yaquina Bay Bridge System. Both systems are triggered by anemometers mounted near the roadway. The thresholds for activation are the same for both systems. The first level activation is made when the average wind speed for any two min interval is in excess of 35 mph, and the second tier of activation is made when the same two minute wind speed average exceeds 80 mph. In the event of tier one activation, a warning is posted to a DMS or flashing beacons are activated (depending on site), the alert is posted to the Internet, and for the Yaquina Bay Bridge System, maintenance crews are notified with faxes to other agencies, along with creating an archived file of the details. When tier two is reached, there is no specific change for the southwest coastal roads; however, for the Yaquina Bay Bridge System, the road is closed (14).

An emerging web publishing platform, Twitter, presents potential for communicating with the public. The service limits each message to 140 characters or less, and thus, each message must be clear and concise, similar to limitations on DMS. The Washington State Department of Transportation sends updates of various traffic alerts and route changes for ferries (15). KDOT has also found Twitter to be a valuable tool. Public affairs managers in the cities of Garden City, Topeka, and Wichita provide “tweets” to followers of traffic situations, construction or maintenance lane closures, and KDOT news. This tool has been used to keep travelers up-to-date on emergency closures and construction progress (16). Some other alert systems such as anemometers, text messaging, and wind socks are also being considered. Anemometers, if maintained and operated properly, can provide precise weather documentation. Text messaging is quicker and can be a vital tool for many truck companies. Wind socks are economical and helpful because they show direction and velocity of the wind (17).

DATA ANALYSIS

The crash data used for the analysis were obtained from KDOT’s Kansas Accident Record System (KARS) (1). This system is a compilation of motor vechicle accident reports, investigation reports for fatalities, and truck and bus supplement reports. Crashes involving large/heavy trucks and strong winds were separated out for the years of 2005, 2006, and 2007. This resulted in 247 crash reports being obtained from the original pool of 3,438. In the reports, a large/heavy truck is defined by vehicle body type and includes single-unit large trucks, truck and trailers, or tractor trailers. To be sure each report met the critera, the research team individually reviewed each of the 247 crash reports. As a result of this careful review, a number of crashes were found to not be related to a truck causing a crash due to wind (acting alone or in combination with other weather phenomena). The most frequent example was a car sliding out of control in windy/rainy conditions (possibly hydroplaning) and striking a truck. Another common occurrence was that a pickup truck (such as a Ford F150 or Dodge Ram 3500) was inadvertently coded in the crash report as the wrong type of truck and, thus, was also removed. After the team reviewed all 247 crash reports, 52 crash records were confirmed to be the exclusive result of an interaction between wind and a truck.

The team also obtained supplementary data to augment the crash reports. This data included both truck ADT for the segment where the crash occurred and independent weather data. Truck average daily traffic (ADT) information was extracted from KDOT's statewide traffic volume map by matching the crash location to the ADT for that same location. Weather data were obtained from the Weather Underground website on a county-by-county basis for each day in the three-year study period. The research team then was able to determine for each crash the wind speed and gust speeds as recorded by the Weather Underground (18).

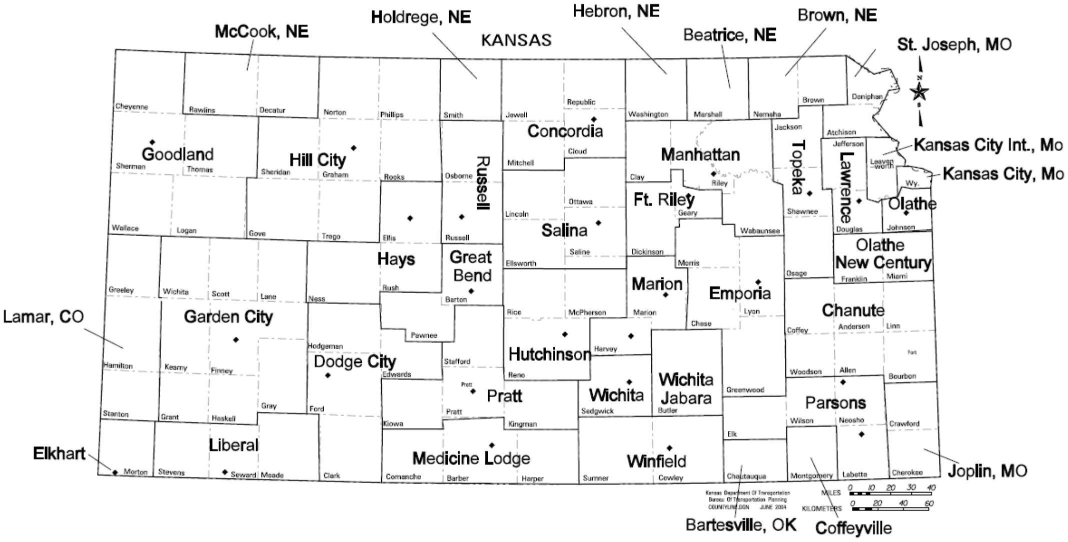
KANSAS
McCook, NE
Holdrege, NE
Hebron, NE
Beatrice, NE
Brown, NE
St. Joseph, MO
Kansas City Int., Mo
Kansas City, Mo
Goodland
Hill City
Concordia
Manhattan
Topeka
Lawrence
Olathe
Russell
Ft. Riley
Salina
Hays
Great Bend
Marion
Emporia
Olathe New Century
Chanute
Lamar, CO
Garden City
Dodge City
Hutchinson
Pratt
Wichita
Wichita Jabara
Parsons
Elkhart
Liberal
Medicine Lodge
Winfield
Joplin, MO
Bartesville, OK
Coffeyville
MILES
KILOMETERS

Figure 2. Location of weather stations in Kansas (18)

Figure 3 shows a distribution of the 52 studied crashes and the assocaited wind gust speeds. What stands out is that crash frequency seems to peak around 40 mph. The research team then compared Figure 3 to a histrogram of all the maximum gust speeds for the entire corridor for the entire three-year period, as shown in Figure 4.

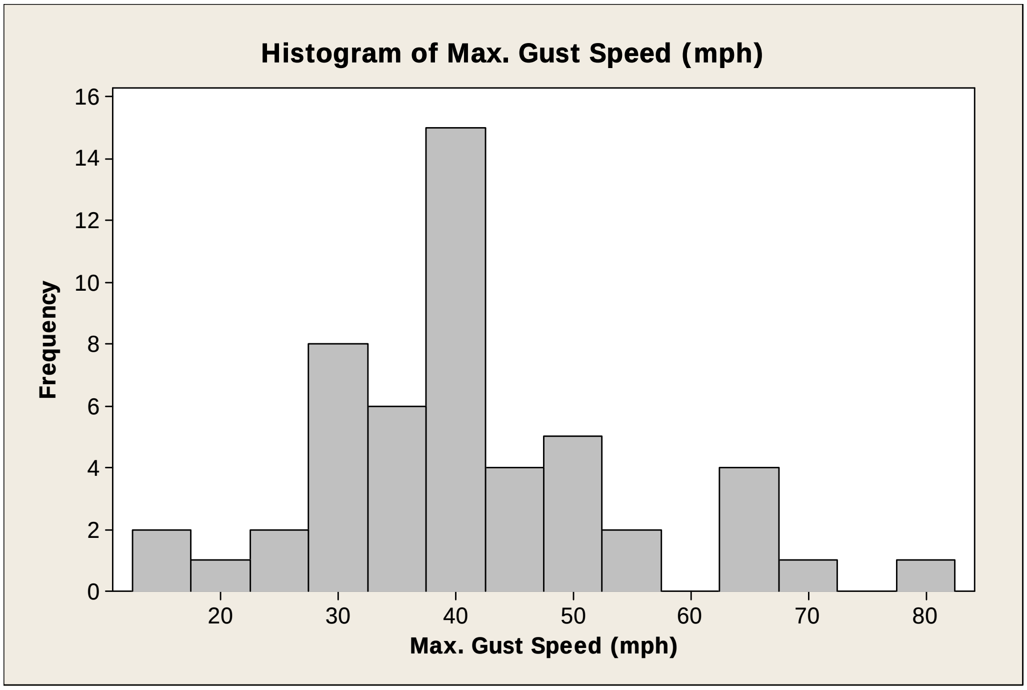

Figure 3. Histogram of maximum gust speed for each crash, N=52

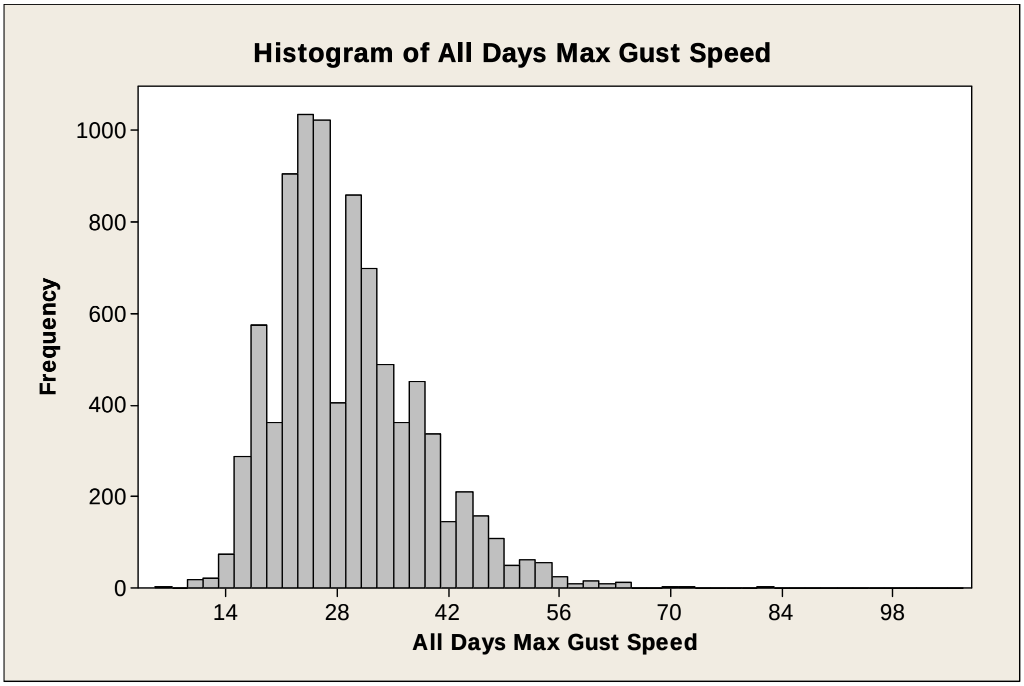

Figure 4. Histogram of maximum gust speeds for I-70 corridor, 2005-2007, N=10950

The team also was interested in how the changing truck ADT varied by milepost and this resulted in Figure 5.

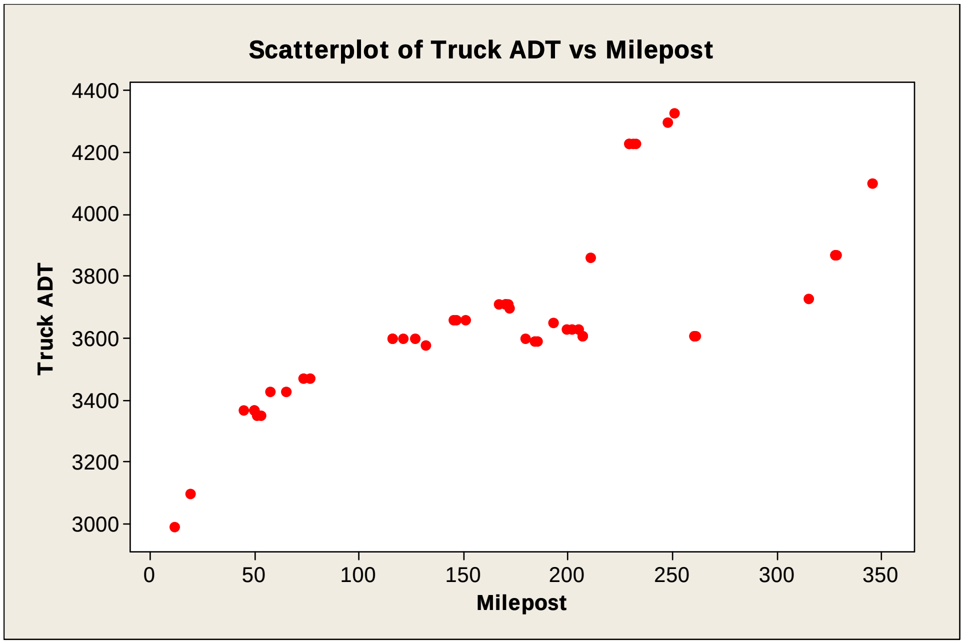

Figure 5. Scatterplot of truck ADT vs. milepost on I-70 in Kansas

Taking a closer look at where along I-70 the crashes studied occurred resulted in Figure 6.

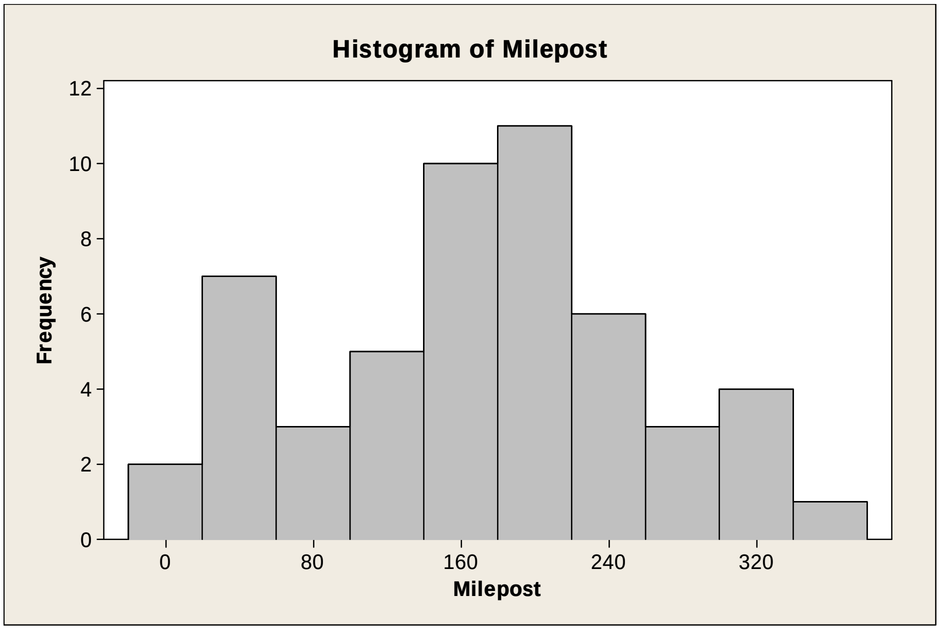

Figure 6. Histogram of wind induced truck crashes by milepost, N=52

In constructing a statistical model, a response variable is required to be predicted. In this case, the response variable chosen was crash rate. The team noted that of the 52 records in the study, none occurred within the same hour of the same day at the same location. It is also important to note that the weather conditions are not constant with location or date. Therefore, the team used the truck ADT data for each location and converted it into an hourly truck ADT; thus, the hourly crash rate was found by dividing one by the hourly truck ADT. Finally, a logit transformation of the crash rate was taken to normalize the data. Expanding on Figure 6, the transformed crash rate was plotted against milepost to produce Figure 7.

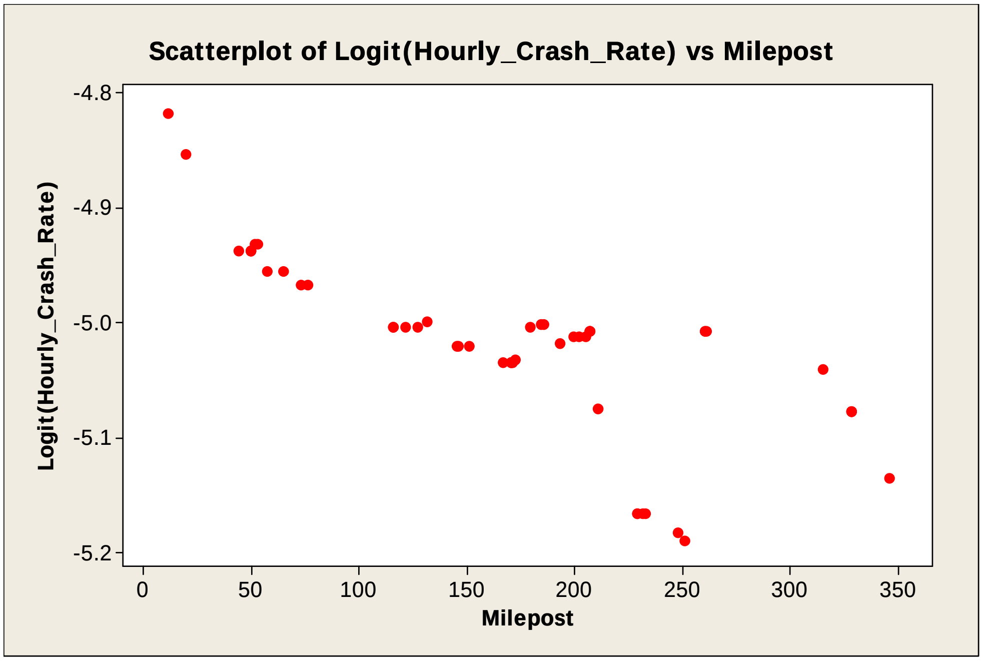

Figure 7. Scatterplot of I-70 wind related truck crash rates versus milepost, N=52

A multivariate linear regression model was then constructed over multiple iterations. The process began considering over 65 variables, including interaction terms, and then evaluating the p-values for the coefficients. Early in the process, it was noted that values for wind speed, wind gust, and empty (no cargo) had p-values in excess of 0.05 but the team forced their inclusion up until the very end. After repeated iterations, including a stepwise regression, the final model for predicting the truck crash rate was found.

$$\mathit{Logit(Hourly\_Crash\_Rate)} = -4.92 + 0.0443\ \mathit{THUNDERSTORM} - 0.000721\ \mathit{Milepost} + 0.0641\ \mathit{Concrete} - 0.127\ \mathit{Hopper} - 0.157\ \mathit{Flatbed} \quad (1)$$

SUMMARY

When one looks at the final form of the regression equation for predicting windblown truck crashes in Kansas on I-70, one important thing stands out—namely that neither wind speed nor wind gust speed was found to be a factor. The factors that were found to be statistically significant were the presence of a thunderstorm, the milepost (the further west, or closer to milepost zero, the more risk), presence of concrete pavement, if the truck is pulling a hopper cargo trailer, and if the truck is pulling a flatbed trailer. This absence is accounted for due to driver behavior changes. For an ideal statistical model, drivers would not alter their behavior due to any adverse weather conditions, and obviously there would be more crashes to study, and thus, a better model that would perhaps be better at isolating specific dangerous wind gust speed thresholds. However, that is simply not the case; drivers do in fact take defensive measures and either alter their driving (increased vigilance, decreased speed, etc.) or get off the road and stop driving until conditions improve.

Looking beyond the probability model, the data do tell an interesting story. Since gust speeds were not a factor in the probability model, it might be expected that mean gust speed for the corridor for the study period could be the same as the mean gust speed associated with the studied crashes. However, over the course of the three-year study period for the entire I-70 corridor that contained the 52 crashes that formed the model, the average maximum gust speed was 28.8 mph, while the mean gust speed for all the crashes in the study was found to be 40.8 mph. A 95% confidence interval for the difference between the means was found to be between 9.6 and 14.5 mph. Subsequently, a null hypothisis that the means are the same can be rejected using a two sample t-test.

The difference in mean gust speeds has several possible implications. First, it is important to recognize what this means. The average gust speed for the entire corridor was found to be 28.8 mph, which means that this is the most probable wind speed and that as the wind speed increases above 28.8 mph, the probability of such an occurence decreases. However, as the wind speed increases above 28.8 mph, the probability of an associated crash increases, until reaching 40.8 mph. This gap is believed to be analagous to the deliemea zone at a signalized intersection. In other words, this gap possibly represents a range or wind speeds where drivers are not taking porportionally precautionary measures as they are taking when the gust speed is in excess of 40.8 mph; thus, they are facing an increased risk but are unaware of doing so. It is theorized that around a gust speed of 40.8 mph is a threshold where drivers begin changing their behavior as previously discussed.

Looking geographically at the data, another interesting observation can be made. Looking at Figures 5, 6, and 7, one observes that the highest frequencies of the crashes studied occur between mileposts 140 and 220 (Figure 6), which accounts for 21 of the 52 crashes (40%). This location bookends around the city of Hays, Kansas, located between mileposts 157 and 161, and is approximatly an inflection point on the plots of truck ADT by milepost (Figure 5) and crash rate by milepost (Figure 7).

CONCLUSION

The findings of this research are consistent with the other literature identified. Like the University of New Brunswick study, wind speeds were not found to be a statistically significant part of any model based on available data. The University of Manitoba simulation study, which found that an empty truck could drive safely in winds up to a range between 31 and 48 mph with an empty trailer, intersects around the dilemma zone identified for I-70 in Kansas. This overlap in the studies provides mutual support for the idea that this may be a critical range and that based on the Kansas data, suggests that while I-70 trucks in Kansas may possibly have a larger safety cushion (up to 48 mph instead of 40.8); they are being more conservative and making behavioral changes at a lower threshold.

The current state of the practice in reducing wind-related truck crashes also supports this Kansas data. The practices in Idaho and Nevada to warn trucks when wind speeds are above 20 mph and to altogether restrict truck traffic when winds exceed 39 mph is in congruence with this study. The lower bound of 20 mph may be if anything slightly conservative; however, it is only slightly lower than the mean corridor wind speed for I-70 in Kansas. What is more interesting is that the critical speed for trucks to be restricted from the highway is within a 95% confidence interval for the mean gust speed associated with the Kansas I-70 truck crashes. This provides additional support for the anecdotal notion that in Kansas, drivers are altering their behavior when winds gust above 40 mph. If such cross-country drivers have encountered similar restrictions in these other states, it may have conditioned them to respond in a similar manner when transiting Kansas.

The critical element that underpinned this study is the crash report. It cannot be overstated that this is the weakest link in the puzzle. In sorting through the KARS data, a very large number of reports were found to not be relevant or had miscoded information. This study only focused on crashes that met very narrow criteria that pared down an initial dataset of 3,438 into 247 (7%), which was then further reduced to just 52 (21% of the 247, 1.5% of the 3,438). At the scene of a crash, the various officers who file the report are first and foremost concerned with the safety of the crash victims and providing for suitable (if any) traffic management and often do not write the report that gets submitted into the KARS system until well after the crash has been cleared. Thus, it is understandable that mistakes may unfortunately creep into the system. However, short of dispatching a dedicated researcher to the scene of every such crash, this is the reality that we have to live with.

The implications of this research are assorted. First and foremost, more data are clearly needed to be able to properly construct a model that hopefully would contain a wind (gust) speed variable. However, it is important to note the context sensitive nature of this data. Weather and wind patterns are based on a wide array of factors, not the least of which is effected by the surrounding geography. While it may be convenient to simply lump crashes together from all geographical locations, it would likely not result in a model that can have any location specific relevance. Secondly, this research suggests that more in-depth study of wind-truck interactions around the 28.8 to 40.8 mph spectrum of wind gusts. To better test a hypothesis that drivers are misjudging risks in this dilemma zone, a driver survey could be conducted in coordination with an interactive driving simulator study.

Turning the lessons learned in this study into “shovel-ready” practice, there are several possibilities. Any solution would have two components: namely the system itself and one or more chosen installation locations. Firstly for the warning system, one option would be to trigger messages on the future ITS system’s DMS to report wind (gust) speeds when they reach a threshold (being careful to not use any language with possible legal liability implications). Another option might be to install windsocks along vulnerable stretches of the highway to provide an instantaneous visual representation for the current wind conditions. In both of these possibilities, drivers would not be mandated to exit the highway, but hopefully would have more available information to make a better decision. Another option would be to follow the model of the several states previously discussed and have a formal warning system, perhaps with a regulatory threshold such that if the wind (gust) speed exceeds a given threshold, truck traffic would be temporarily prohibited. Recognizing that KDOT does not have unlimited resources based solely on the frequencies observed, the greatest potential return on investment would be for the stretch of I-70 that surrounds Hays, Kansas, possibly extending all the way to the western border with Colorado. However, any system utilized and installation locations chosen should be given full engineering and legal scrutiny prior to construction.

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The authors would like to acknowledge the numerous contributions by the various staff at the Kansas Department of Transportation, without whose help this paper would not have been possible.

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Self-Centering Bridge Piers with Structural Fuses

Jon “Matt” Rouse
Department of Civil, Construction, and Environmental Engineering
Iowa State University
408 Town Engineering
Ames, IA 50011
jmr19@iastate.edu

Mark C. Currie
HNTB

F. Wayne Klaiber
Bridge Engineering Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
klaiber@iastate.edu

ABSTRACT

An innovative structural system for pier columns was investigated through a series of laboratory experiments. The columns and connections examined were comprised of precast concrete segments to accelerate construction. In addition, some of the columns employed elastic elements to self-center the columns when subjected to lateral loads and structural fuses to control large lateral deflections, dissipate energy, and expedite repair in the event of a catastrophic loading event.

Six cantilever columns with varying component materials and connection details were subjected to a regimen of vertical dead loads and cyclic, quasi-static lateral loads. One column was designed as a control column to represent the behavior of a conventional reinforced concrete column and provide a basis for comparison with the remaining five jointed columns designed with the proposed structural system. After sustaining significant damage, the self-centering, jointed columns were repaired by replacing the structural fuses and retested to failure to investigate the effectiveness of the repair.

The experiments identified both effective and unsatisfactory details for the jointed system. Two of the jointed columns demonstrated equivalent lateral strength, greater lateral stiffness, and greater lateral deformation capacity than the control column. The self-centering capability of the jointed columns was clearly demonstrated as well, and the repair technique proved effective as demonstrated by nearly identical pre- and post-repair behavior. The authors believe the proposed system to be a feasible alternative to conventional pier systems.

Key words: jointed column—pier columns—repair—self-centering

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SHRP2 R02—Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform

Vernon R. Schaefer
Department of Civil, Construction, and Environmental Engineering
Iowa State University
488 Town Engineering
Ames, IA 50011
vern@iastate.edu

ABSTRACT

The second Strategic Highway Research Program (SHRP2) was created by the U.S. Congress to address challenges of moving people and goods efficiently and safely on the nation's highways. Geotechnical transportation issues are addressed under the SHPR2 Renewal Focus Area, in which the goal is to develop a consistent, systematic approach to the conduct of highway renewal that is (1) rapid, (2) causes minimal disruption, and (3) produces long-lived facilities. Although in existence for several decades, many geoconstruction technologies face both technical and non-technical obstacles, preventing broader and more effective utilization in transportation infrastructure projects. SHPR2 R02 is investigating the state of practices of transportation project engineering, geotechnical engineering, and earthwork construction to identify and assess methods to advance the use of these geoconstruction technologies. The identified technologies are often underutilized in current practice, and they each offer significant potential to achieve one or more of the three SHRP2 Renewal objectives listed earlier. Project R02 encompasses a broad spectrum of materials, processes, and technologies within geotechnical engineering and geoconstruction that are applicable to one or more of the following "elements" of construction: (1) new embankment and roadway construction over unstable soils, (2) roadway and embankment widening, and (3) stabilization of pavement working platforms.

Phase 1 of the project, completed in August 2008, consisted of six tasks focused on identifying those geotechnical materials, systems, and technologies that best achieve the SHRP2 Renewal strategic objectives for the three elements. Explicit in the tasks was the identification and evaluation of technical issues, project development/delivery methods, performance criteria and quality assurance and control (QA/QC) procedures, and non-technical issues that constrain full utilization of geotechnical materials, systems, and technologies. A total of 47 applicable geoconstruction technologies were identified in the Phase 1 work. Seventeen technical issues and project development pros and cons were identified that interfere with more widespread use of geoconstruction technologies. Fifteen non-technical issues and project specific parameters limiting use of the technologies were identified. In addition to identification of issues and obstacles, Phase 1 included the identification of the most promising mitigation methods to overcome the obstacles.

Phase 2 of the project includes evaluation of the effectiveness of mitigation measures; a catalogue of materials and systems for rapid renewal projects; guidance for design and QA/QC procedures; methods for estimating costs; and sample specifications for the identified geotechnical materials, systems, and technologies. The research team proposes to incorporate the development of the catalogue and the requirements for guidance on design, QA/QC, costs, and specifications into an integrated catalogue and guidance system. This system will provide the data necessary for determining the applicability of specific

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technologies to specific projects and will guide the user to information needed to apply the selected technologies. The catalogue will include information necessary for initial screening as well as design methodologies, QA/QC, costs, and specifications.

Key words: geoconstruction—geotechnical—Renewal objectives

Pervious Concrete Mix Design for Wearing Course Applications

Vernon R. Schaefer
Department of Civil, Construction, and Environmental Engineering
Iowa State University
488 Town Engineering
Ames, IA 50011-3232
vern@iastate.edu

Kejin Wang
Department of Civil, Construction, and Environmental Engineering
Iowa State University
492 Town Engineering
Ames, IA 50011-3232
kejinw@iastate.edu

John T. Kevern
Department of Civil and Mechanical Engineering
University of Missouri, Kansas City
370H Flarsheim Hall
5100 Rockhill Rd.
Kansas City, MO 64110
kevernj@umkc.edu

ABSTRACT

Portland cement pervious concrete (PCPC) has shown great potential to reduce roadway noise, improve splash and spray, and to improve friction as a surface wearing course. This paper presents the results of studies conducted at Iowa State University and the National Concrete Pavement Technology Center (CP Tech Center) to develop mix designs and procedures for the use of PCPC overlays for highway applications. Issues related to characterization of workability, the need for air entrainment and the effect of air entrainment on durability, the development of design mixture proportions for mechanized placement, the evaluation of the curing requirements for surface abrasion resistance, and the development of overlay design procedures are presented and discussed. The results of these studies show that effective PCPC overlays can be designed for wearing course applications.

This presentation is part of the Sustainable Concrete Pavement Technologies session.

Key words: abrasion resistance—curing—overlay design—PCPC mix design—Portland Cement Pervious Concrete—workability

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INTRODUCTION

Portland cement pervious concrete (PCPC) contains the same material components as conventional concrete of cementitious binder, aggregate, water, and chemical admixtures but through specific mixture proportioning, maintains around 20% porosity for water percolation. PCPC has shown great potential to reduce roadway noise, improve splash and spray, and to improve friction as a surface wearing course. A pervious concrete mix design for a surface wearing course must meet the criteria of adequate strength and durability under site-specific loading and environmental conditions. To date, two key issues that have impeded the use of pervious concrete in the United States are that strengths of pervious concrete have been lower than necessary for required applications and the freeze-thaw durability of pervious concrete has been suspect.

The strength and freeze-thaw durability of pervious concrete mix designs has been addressed in a recently completed study by Schaefer et al. (2006) who showed that a strong, durable pervious concrete mix design that will withstand hard, wet-freeze environments. The strength and durability is achieved through the use of a small amount of fine aggregate (i.e., concrete sand) and/or latex admixture to enhance the particle-to-particle bond in the mix. The preliminary results were reported in Kevern et al. (2005).

The National Concrete Pavement Technology Center (CP Tech Center) at Iowa State University is currently conducting a study entitled “An Integrated Study of Portland Cement Pervious Concrete for Wearing Course Applications.” The objective of the study is to conduct a comprehensive research program focused on the development of pervious concrete mix designs with adequate strength and durability for wearing course pavements. Mixtures are being designed to possess surface characteristics that reduce noise and enhance skid resistance while providing adequate removal of water from the pavement surface and structure. It is anticipated that, ultimately, a range of mix designs will be necessary to meet requirements for wearing course applications. Additionally, constructability issues for wearing course sections must be addressed to ensure that competitive and economical placement of the pervious concrete can be done in the field. Hence, during the evaluation phases, both laboratory and field testing of the materials and construction are being conducted, focusing on the development of a durable wearing course that can be used in highway applications for critical noise, splash/spray, skid resistance, and environmental concerns.

To maximize the potential benefits of pervious concrete as an overlay material for noise reduction and skid resistance, the mixture must possess the following properties:

- Adequate strength for long-term durability
- Highly durable aggregate
- Sufficient porosity (around 25%) to maximize noise reduction and minimize maintenance
- High workability for ease of placement and uniform porosity across the pavement thickness
- Ability to maintain voids when compaction is applied by the paver for uniform surface porosity

A number of tasks were developed to meet these objectives. This paper describes the results of research efforts to date, including (1) development of a method to characterize workability, (2) determination of the need for air entrainment and the effect of air entrainment on durability, (3) design mixture proportions for mechanized placement, (4) evaluation of the curing requirements for surface abrasion resistance, and (5) overlay design procedures and development. Work continues on other tasks of the project related to evaluation of the effect of deicing on pervious concrete durability, evaluation of clogging and permeability test development, and monitoring of field trials at the Minnesota Department of Transportation Cold Weather Road Research Facility (MnROAD), including measurement of road noise.

CHARACTERIZATION OF WORKABILITY

PCPC mixtures that have excellent performance in the lab may stiffen during transport, resulting in poor compaction or requiring additional water in the field. Addition of water at the jobsite increases water-to-cementitious binder ratio (w/c), impacting concrete strength and durability. Many practitioners of pervious concrete have experienced instances when principles applied from traditional concrete to pervious concrete have resulted in a less-than-optimal final product. To date, determining the workability of pervious concrete has been considered an art form since the conventional slump test does not provide useful information for such stiff concrete. The current method is to evaluate the concrete's ability to form a ball with the plastic pervious concrete (Tennis et al. 2004). This method is impossible to specify due to lack of quantifiable values and individual bias. A more scientific method of workability determination is required if PCPC is to be used for large-scale parking areas and surface overlays.

Pervious concrete is designed to transport stormwater into the underlying layers through a series of interconnected voids, while providing the designed load-carrying capacity. The interconnected voids are produced from a balance between aggregate gradation and binder content. In the concrete mixture design, the objective is to provide a sufficient number of voids to infiltrate the design stormwater intensity. There is a direct relationship between voids and compressive strength, where lower void contents produce more intraparticle contact and consequently higher load-carrying capacities (Schaefer et al. 2006). The void content of the plastic and hardened pervious concrete can be determined from the unit weight. Determination of plastic workability becomes increasingly important since the required parameters (permeability and strength) are based on unit weight, which is achieved through proper placement. A highly workable mixture requires less compaction energy to achieve higher unit weight than a stiffer mixture. By quantifying pervious concrete workability, mixtures can be designed to produce certain void contents using specified compaction methods, and the workability can be verified and adjusted accordingly before placement.

A Superpave gyratory compactor (SGC) was modified to develop a test method to characterize the workability of pervious concrete. In this procedure, pervious concrete samples were produced using a SGC that allows for simulating various field compaction conditions. Workability of the concrete is then defined by the density versus gyration relationship. A matrix of concrete mixtures that consists of various w/c and cement contents were tested. The effect of mixing time on concrete workability is also evaluated so as to identify "slump loss" of field pervious concrete. The results show the SGC is able to produce consistent pervious concrete specimens, and the output of the test method well quantifies the workability and compactibility of the plastic concrete. The discussion includes a range of suggested values to allow design and verification of production pervious concrete workability.

It is well-understood that gyratory compactors better simulate the type of compaction utilized by the asphalt industry, primarily steel drum and pneumatic compaction (AI 2001). Since pervious concrete is loosely placed and then finished/compacted with either a weighted drum or roller-screed, the use of a gyratory compactor is appropriate to simulate field conditions. Normal conditions for Superpave asphalt design require a 600 kPa (87 psi) pressure for laboratory compaction to simulate field compaction (AI 2001). For this study, a gyratory compactor was modified to achieve compactive effort of 60 kPa (8.7 psi), within a tolerance of 2 kPa (0.3 psi), for 150 mm (6 in.) diameter samples.

A matrix of mixtures was tested across a variety of mixture properties, including cementitious material contents, w/c ratios, and duration of mixing times. Observation of the compaction curve defines both the initial concrete workability and the resistance of the mix to further compaction. A typical low-pressure compaction density relationship is shown in Figure 1. The Workability Energy Index (WEI) represents the mixture's initial workability, while the Compaction Densification Index (CDI) represents the resistance to additional compaction. Values are presented for acceptable ranges in Table 1.

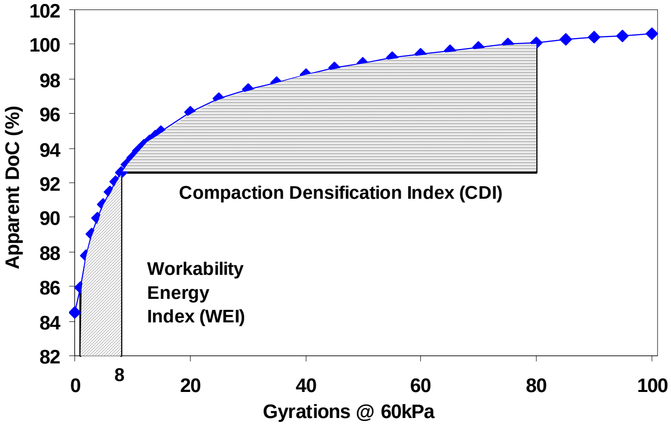

Figure 1. Compaction density and compactibility parameters for pervious concrete

Table 1. Ranges of pervious concrete workability values

Workability (WEI)	
Behavior	**Range**
Highly Workable	> 640
Acceptable Workability	640>WEI>600
Poor Workability	WEI<600

Compactibility (CDI)	
Explanation	**Range**
Self-Consolidating	CDI<50
Normal Compaction Effort Required	50<CDI<450
Considerable Additional Compaction Effort Required	CDI>450

In pervious concrete, a larger amount of paste is exposed to the air than traditional concrete, making the fresh concrete especially susceptible to paste stiffening caused by loss of moisture and admixture effectiveness. A wide variety of materials are used to modify the workability and extend the placing window, and admixtures are used in pervious concrete without a method to quantify initial effects and effectiveness over time. Research was also performed to determine the effect of common admixtures, supplementary cementitious materials (SCMs), and physical modifiers on pervious concrete. The results show that admixtures have a shorter working window in pervious concrete. More detailed information on the workability of pervious concrete can be found in Kevern et al. (2009a).

DETERMINING THE NEED FOR AIR ENTRAINMENT IN PERVIOUS CONCRETE

The number of PCPC installations is increasing, especially in cold climates. It is widely accepted that air entrainment increases the freeze-thaw durability of traditional concrete (Kosmatka et al. 2002). The microscopic air-void system can provide spaces in the concrete to accommodate expansive materials, such as water that is expelled from ice formation, thus reducing hydraulic and osmotic pressures. PCPC has a more complicated void system than traditional concrete, containing not only the small-sized entrapped and entrained air in the paste or mortar but also porosity, the larger-sized interconnected void space between the paste-coated aggregate particles. While air content is a standard property of traditional concrete, no method is currently used to characterize the air voids in pervious concrete.

Due to the large porosity in pervious concrete, commonly used methods of concrete air measurement, such as pressure or volumetric air meters, do not provide useful data for pervious concrete, although the National Ready Mixed Concrete Association (NRMCA) suggests air-entraining pervious concrete at standard dosage rates to produce concrete curb mixtures (NRMCA 2005).

The RapidAir system is a relatively new device that automatically determines entrained air properties using ASTM C 457-98 (CEI 2002). Sample cross sections are stained black and then the voids are filled with a white material, such as zinc paste. The contrast allows the device to distinguish between air voids and the hardened matrix of either paste or aggregate. Recent studies have shown that the RapidAir system has a high degree of multi-lab reproducibility and has less variation than the manual technique (Jakobsen et al. 2006).

The air structure of pervious concrete was determined using the RapidAir system. The test results provide insight into whether or not the use of air-entraining agents (AEA) in pervious concrete is necessary and if the dosage rates of AEA used were sufficient to impact durability. Figure 2 shows wide and close-up views of samples prepared for RapidAir testing. Sample A has no air entrainment, while sample B has a double dosage of a synthetic air-entraining agent. The grey paste in sample B clearly distinguishes between aggregate, voids, and air-entrained paste. The results from freeze-thaw testing of mixtures with various levels of air entrainment are shown in Figure 3. The samples with no air entrainment (PG) had poor durability, while samples with air entrainment were much more durable. The best performing samples had a double dosage of a synthetic AEA.

From the study it was found that air entrainment increased the paste volume and improved workability of pervious concrete, thus reducing overall porosity and increasing density. The effect of air entrainment on porosity and workability is more pronounced for concrete made with the rounded pea gravel aggregate than for concrete made with the angular crushed limestone. Concrete having lower porosity and consequently higher unit weight displayed higher strength, better freeze-thaw resistance, and lower permeability. The RapidAir test results indicated that even without air entrainment, pervious concrete still had spacing factor values less than 0.2 mm (200 μm). This implies that it is the improved density resulting from air entrainment that enhanced freeze-thaw resistance. The recommended dosage of synthetic air entrainer produced equivalent contents of entrained air as the double recommended dosage of the natural air entrainer. Synthetic air entrainer produced higher amounts of air entrainment than the natural air entrainer at a given dosage. The entrained air-void structure of pervious concrete can be characterized using the RapidAir device. More information on the relationship between air entrainment and the freeze-thaw durability of pervious concrete can be found in Kevern et al. (2008a).

OPTIMIZATION OF PERVIOUS CONCRETE MIXTURES FOR MECHANIZED PLACEMENT

As PCPC progresses from full-depth parking lot type applications to surface wearing course use in the United States, certain obstacles must be overcome to produce a durable surface. Pervious concrete used as a surface course will be subjected to much more extreme conditions, and the mix design must be optimized for strength, permeability, and especially better freeze-thaw durability. The objectives of the overlay mixture design are to produce a self-consolidating mixture that retains an interconnected void structure after mechanized placement. Mechanized placement will be required to produce a consistent surface texture (porosity in the top ½ in.) for uniform noise generation. A sufficient flexural strength will be important for the ability to withstand traffic volumes higher than those experienced in parking lot applications. Sufficient bond strength will be required between the pervious overlay and the standard concrete to transfer loading without debonding.

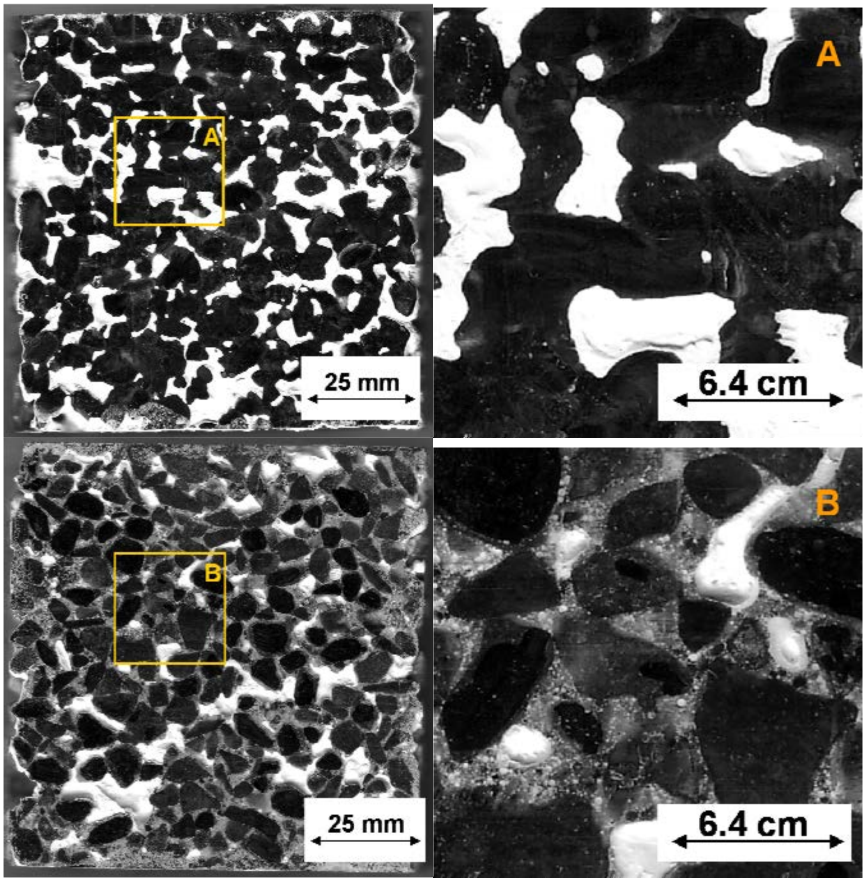

Figure 2. RapidAir samples showing various levels of air entrainment

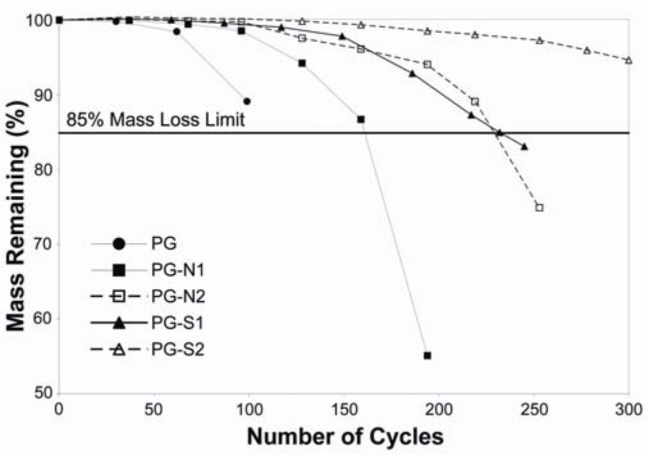

Figure 3. Freeze-thaw test results

The mixture design experimental phase was divided into eight sections determining aggregate type, binder-to-aggregate amount, optimized sand content, w/c ratio, fiber type, fiber addition rate, cementitious material composition, and chemical admixture scheme. The objective was to produce a self-consolidating concrete that also required considerable additional compaction efforts, had porosity between 20% and 25%, and seven-day tensile strength greater than 2.1 MPa (305 psi). Typical pervious sections are opened to traffic after seven days, so mixtures were iterated on seven-day tensile strength and workability.

The selected coarse aggregate was crushed granite, with 98% passing the 9.5 mm sieve and 18% passing the 4.75 mm sieve, selected due to previous performance and availability. The granite had specific gravity of 2.65, absorption of 0.6%, micro-deval abrasion loss of 7%, and compacted voids of 45%.

Two types of fibers were used, a shorter fibrillated polypropylene previously investigated in pervious concrete and a cellulose micro-fiber (9). Fibers were included at 0.9, 1.8, and 3.0 kg/m^3. Cementitious materials included Type II portland cement, class C fly ash, and grade 120 blast furnace slag investigated up to 50% replacement for cement with SCMs. The baseline mixture included a high-range water reducer and air-entraining agent; additional admixtures included individual viscosity modifiers and combinations of viscosity modifiers, hydration stabilizer, two latex-based workability aids, and slipform rheology modifying admixture.

Once the aggregate type and initial gradation were selected, the optimized binder content was investigated. Binder-to-aggregate (b/a) ratio was varied between 18% and 24% by volume. Mixture proportions were adjusted to maintain equal differential voltage contrast (DVC). A slight increase in initial workability occurred with increased binder, while a significant drop in required compaction energy occurred between 21% and 22.5%, with a small additional decrease at b/a = 24%. For all mixtures, porosity was between 25% and 30%, although seven-day compressive strength increased from 14.8 MPa (2150 psi) for b/a = 21% samples to 17.9 MPa (2600 psi) for b/a = 24% samples. At b/a greater than 24%, the samples were impermeable. In addition to the binder, w/c was varied for all three binder contents. Traditionally, water is added to pervious concrete to improve workability; at least for this combination of aggregate and binder volume, additional water did not improve workability. At w/c greater than 0.33, the paste drained from the aggregate creating the potential for imperviousness. A w/c of 0.29 was selected for subsequent iterations.

The effect of sand content on the original gradation was investigated for mixtures containing 21.5% binder and w/c of 0.29. Workability response for sand addition is shown in Figure 4 for 0% to 15% sand-to-gravel ratio (S/G) by mass. Initial workability increased slightly between 0% and 12.5%; however, between 7.5% and 10% there was a significant decrease in the required compaction energy. Sand increases the paste/mortar volume and the mortar viscosity, allowing the coarse aggregate particles to support a thicker paste layer. The increased paste viscosity did not significantly improve workability but separated the particles allowing better compaction. The mixture response is shown in Figure 5 for porosity and seven-day compressive strength. At S/G up to 10%, the fine aggregate bulks the mortar volume, creating better compaction and strength. Above S/G of 10%, the additional surface area demand of the fine aggregate begins to negatively impact the mixture properties.

For both types of fibers, there was no effect on initial workability with addition rate. Compactibility increased linearly with addition rate for the polypropylene fibers, while no increase in compactibility was observed until the 3 kg/m^3 (5 pcy) rate for the cellulose fibers. A maximum seven-day tensile strength of 2.2 MPa (319 psi) occurred at the 3.0 kg/m^3 (5 pcy) rate for the polypropylene fiber and of 2.0 MPa (290 psi) at the 0.9 kg/m^3 (1.5 pcy) rate for the cellulose fibers. Cellulose fibers were selected due to the ability to maintain initial workability while requiring a higher level of compaction.

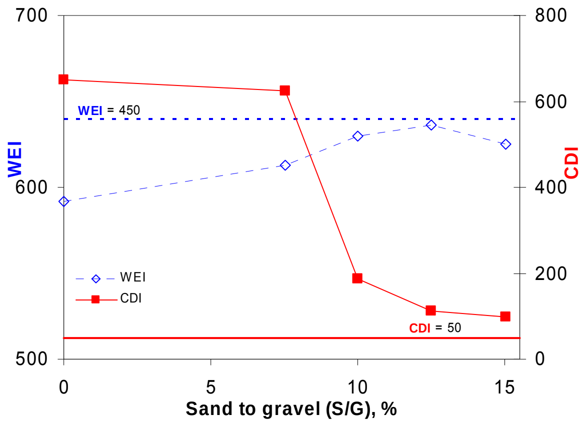

Figure 4. Effect of fine aggregate on workability

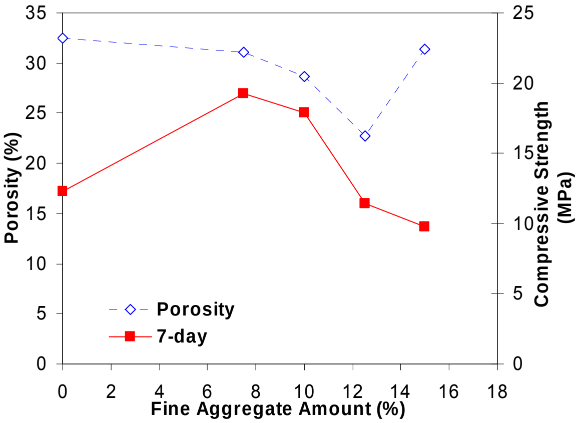

Figure 5. Effect of fine aggregate on concrete properties

A range of SCM combinations were evaluated at 50% replacement for ordinary portland cement (OPC). The highest compressive and tensile strength occurred for samples containing 50% blast furnace slag. However, only a 0.10 MPa decrease in seven-day tensile strength occurred when the concrete contained 35% slag and 15% fly ash. Compressive strength was lower than 100% OPC for all SCM combinations, while all tensile strength results were higher. Due to the potential for greater long-term strength development, the (50% OPC, 35% slag, 15% fly ash) ternary mixture was selected. The highest workability occurred for the (50, 25, 25) and (50, 0, 50) mixtures, while the lowest required compaction energy was observed in the 50% fly ash (50, 0, 50) mixture. All combinations of SCMs had higher workability and lower required compaction energy than the OPC mixture.

All combinations of chemical admixtures improved the initial workability and most decreased the required compaction energy. The selected non-polymer mixture contained a viscosity-modifying agent (VMA) and hydration stabilizer (HS), which did not affect the WEI but caused a drop in CDI. To potentially increase bond strength during subsequent testing, a polymer-modified mixture was also selected. Testing was performed in the saturated state without drying to coalesce the polymer film, which is expected to further increase tensile strength. The latex admixture caused a slight drop in WEI but a substantial increase in CDI. Future testing may include workability behavior with more realistic increased mixing times and drying cycles to mimic field conditions. Additional information on the overlay mixture proportions can be found in Kevern et al. (2008b).

CURING AND SURFACE ABRASION RESISTANCE TESTING

When pervious concrete is applied to pavements in areas that undergo freeze-thaw, durability also refers to the surface abrasion resistance against snow-clearing operations. If pervious concrete is to progress from parking lot applications to low-volume and potentially high-volume applications, the pavement must be resistant to all aspects of cold weather maintenance.

Concrete curing is required to maintain sufficient moisture to allow cement hydration and concrete microstructure development (Wang et al. 2006). Also, curing has been shown to impact concrete durability as well as concrete strength (ACI 2000). Many techniques exist to control moisture loss in traditional concrete, although most are not appropriate for pervious concrete. Because of the high porosity of pervious concrete, rapid loss of moisture from the fresh concrete due to evaporation can occur. Since the w/c of the concrete is generally low, loss of moisture can result in rapid desiccation, low strength, and excessive surface raveling. Thus, curing is especially important for pervious concrete, because unlike traditional concrete, the bottom of the slab is exposed to air as much as the surface. On the other hand, protecting the surface may allow proper curing throughout. For PCPC, water misting or fogging washes the cement paste from the coated aggregate particles. Due to potential surface damage of the fresh concrete, wet burlap cannot be applied until final set has been reached, which results in excess surface desiccation. Liquid membrane-forming compounds prevent surface moisture loss but do nothing to prevent evaporation from within pervious concrete. Curing compounds are designed to prevent moisture loss from the surface of freshly placed concrete, which presents an obstacle for proper pervious concrete curing.

The current method of curing PCPC involves covering the fresh concrete with plastic sheets and allowing the pavement to cure for seven days before removal of the plastic. In most cases, the plastic sheets must be rolled onto a pipe for rapid application after placement, and aggregate or sand bags must be used to seal the edges and prevent wind from ballooning under the plastic and drying the surface. Covering with plastic is the preferred method to cure pervious concrete but can be problematic and no studies have been performed to determine if covering is sufficient or even required. Pervious concrete samples were evaluated for nine different curing methods or curing materials by flexural strength and surface abrasion resistance.

Flexural strength was determined using modulus of rupture of the beams tested at 28 days according to ASTM C 78. Once the samples were tested for modulus of rupture, the fractured pieces were tested for surface abrasion. Surface abrasion was determined according to ASTM C 944, in which a constant load of 98 N (22 lbs) is applied through rotary cutter dressing wheels in contact with the sample surface for two minutes. The diameter of the circular abraded area is 80 mm (3.25 in.). The beams were first cleaned with a stiff-bristled brush and vacuumed on all sides to remove any loose materials. After each abrasion test, the beams were again brushed clean and vacuumed to remove loose debris. The mass loss between trials was recorded and a total of six abrasion tests were performed on each set of beams. Figure 6 shows the abrasion device with the shaft-mounted container for load calibration and abrasion head-cutting device. The physical result of an abrasion test is shown in Figure 7 for a beam cured with the standard white-pigment curing compound. The left portion of the sample had not undergone testing while the tested portion is the exposed aggregate circular section on the right. The Abrasion Index (AI) was taken as the ratio of the average abraded mass loss for a particular sample divided by the average for the control mixture with no curing method.

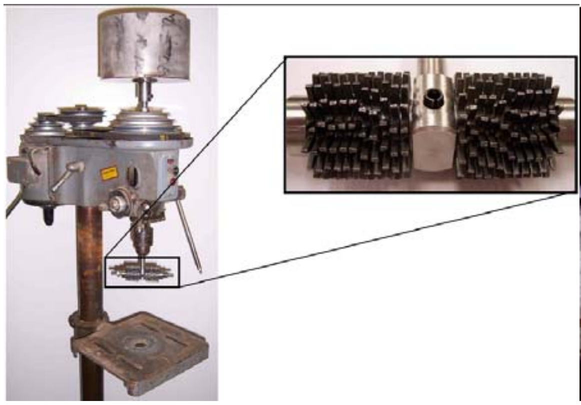

Figure 6. Surface abrasion device

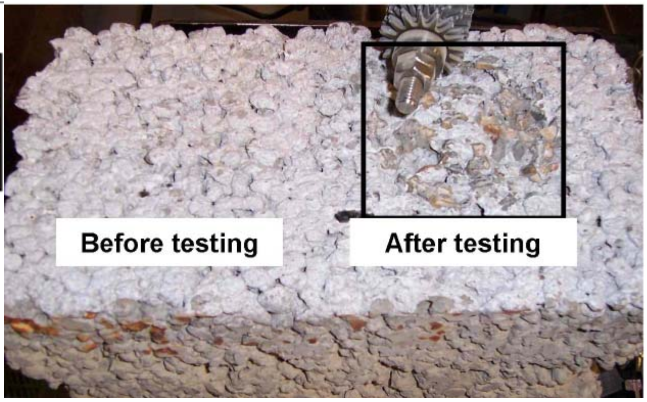

Figure 7. Abrasion physical test results

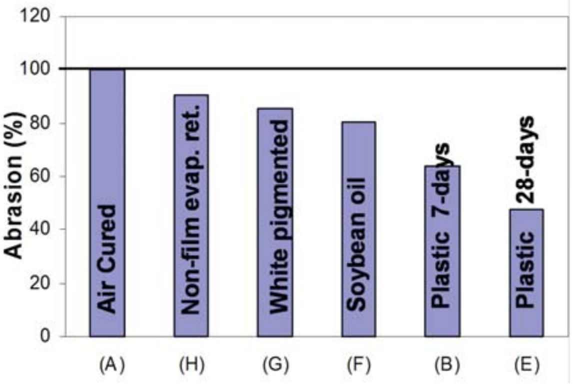

Figure 8. Abrasion results

Results show that the samples cured under plastic had the best abrasion resistance and highest flexural strength. There was no significant difference in flexural strength between samples cured under plastic for 7 or 28 days, although abrasion resistance did increase with the duration of curing. Soybean oil has the potential to be used as an effective curing compound. In this study, the soybean oil emulsion produced the best surface durability and increase in flexural strength of the surface-applied curing agents. The “birds nest effect” caused by the fibers increased the porosity by 7.9% and yet produced a flexural increase of 21% over the control, without significantly impacting surface abrasion. The rotary-cutter surface abrasion ASTM C 944 method has the ability to differentiate between curing methods, allowing relative surface durability comparisons. More information on curing and abrasion of pervious concrete can be found in Kevern et al. (2009b).

OVERLAY DESIGN PROCEDURES AND DEVELOPMENT

One important aspect of a bonded overlay is the bond strength between the substrate and the overlay. Pervious concretes have been placed wet-on-wet, which produces very high bond strength. However, traditional pervious concrete does not lend itself to high bond strength over existing pavement due to the dry nature of the mixture and the reduced contact area. Fortunately, pavement modeling of pervious concrete wearing course indicates a relatively low strength is required of 145 psi (Bax et al. 2007).

Simulated overlay mixtures have been placed with the two selected pervious concrete mixtures over standard concrete: one containing the VMA and the other the latex additive. Samples were placed without any compaction or vibration to textureless prepared concrete specimens. Various bonding methods included a clean and dry surface, poly-modified grout, standard grout, and a polymer tack coat. Samples were tested using the shear device shown in Figure 9 and the Iowa Department of Transportation (Iowa DOT) test method 406 C, which applies load at 400–500 psi/min. The absence of vibration or compaction was designed to produce the lowest and most conservative bond values possible for these particular mixtures and techniques. It has been observed that the normal amount of vibration present during placement will tend to draw some pervious paste to the bond interface and is expected to greatly improve strength. Samples have been placed at less conservative, more realistic vibration levels to determine the effect on bond strength.

Results from the overlay shear testing indicate variability was high with bond strength either good or poor; one example includes a clean, dry polymer mixture with variability from 22 psi to 215 psi. The average for the clean and dry polymer concrete samples was below the limit at 111 psi, although some samples were well above the required value. Additional research is needed to determine a realistic placement technique to reduce the testing variability and determine the actual required pavement conditions.

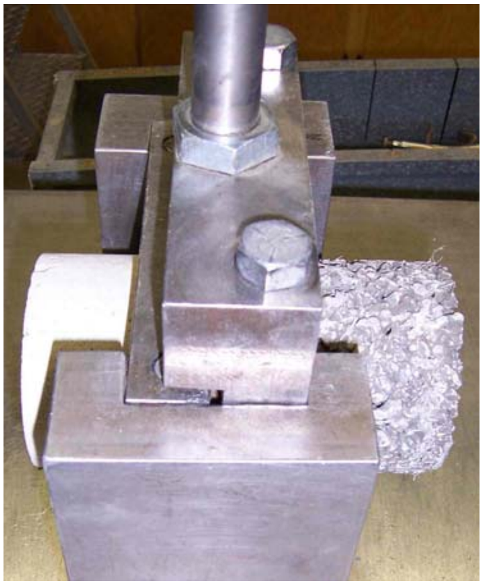

Figure 9. Testing shear strength of overlay bond

SUMMARY

PCPC has shown great potential to reduce roadway noise, improve splash and spray, and to improve friction as a surface-wearing course. This paper has presented the results of studies conducted to develop a method to characterize workability, determine the need for air entrainment and the effect of air entrainment on durability, develop design mixture proportions for mechanized placement, evaluate the curing requirements for surface abrasion resistance, and develop overlay design procedures. The results of these studies show that effective PCPC overlays can be designed for wearing course applications.

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The Use of Hydroacoustics and Sediment Coring in Bathymetric Mapping and Modeling

Douglas J. Schnoebelen
IIHR-Hydroscience & Engineering
University of Iowa
323-A C. Maxwell Stanley Hydraulics Laboratory
Iowa City, IA 52242-1585
douglas-schnoebelen@uiowa.edu

Nathan C. Young
IIHR-Hydroscience & Engineering
University of Iowa
133 C. Maxwell Stanley Hydraulics Laboratory
Iowa City, IA 52242-1585
nathan-young@uiowa.edu

ABSTRACT

The record floods of 1993 and 2008 in Iowa's rivers and streams have highlighted the need of a better understanding of scour, deposition, and sediment transport. Sediment is listed as a major impairment in Iowa streams by the Iowa Department of Natural Resources. Understanding the fate and transport of sediment from agricultural land and stream banks is critical. The use of hydroacoustics and sediment coring are providing new tools for bathymetric mapping, channel morphology, scour, deposition, and potential effects on riverine structures. The Lucille A. Carver Mississippi Riverside Environmental Research Station (LACMRERS)/IIHR-Hydroscience and Engineering are using a multibeam hydroacoustic system and sediment coring to better evaluate sediment process and problems. The multibeam sonar can map at the centimeter scale. Continuous sediment cores of 3−4 m length can be obtained for grain size analysis, chemical composition, and age dating. Research to date has been involved with three different scales and projects on the Mississippi River, Iowa River, and Des Moines River. The Mississippi River data are presented on modeling and potential dredging; the Iowa River in and near Iowa City, Iowa, using bathymetry in conjunction with LIDAR in streamflow modeling is discussed. On the Des Moines River, scour around bridge piers has been conducted, clearly showing areas of scour and rip-rap placement.

Key words: bathymetric mapping—floods—sediment

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Missouri's Use of Recycled Asphalt Shingles (RAS) in Hot Mix Asphalt

Joe Schroer
Missouri Department of Transportation
1617 Missouri Blvd.
P.O. Box 270
Jefferson City, MO 65102
Joe.Schroer@modot.mo.gov

ABSTRACT

Missouri has been a leader in the use of asphalt shingles as an addition to hot mix asphalt (HMA). A provision was added in 2006 to the Missouri Standard Specifications for Highway Construction to allow the addition of reclaimed asphalt shingles (RAS) in any mixture requiring the use of our standard paving grade of asphalt, PG 64-22. A maximum of 7% RAS may be used and it can be waste, manufacturer, or new shingles; or post-consumer, old shingles removed from roofs. The use of shingles in HMA started in Joplin, Missouri, using manufacturer's waste shingles. After being approached by Pace Construction and Peerless Resource Recovery, it became evident that post-consumer, also called tear-off shingles, would provide greater volume and make a larger impact to Missouri's landfills. The mixtures have worked so well that there are currently 12 entities across the state collecting shingles stockpiled to produce RAS. With high oil prices the last two years and shortages of asphalt cement, it has been advantageous to the HMA industry to look at RAS both as an additive of a percent of the asphalt binder to realize cost savings and also to recycle a waste material that has added millions of tons to landfills. The Missouri Department of Transportation (MoDOT) has encouraged advancement of the use of RAS with a workshop sponsored by government and industry to promote and inform others of "Missouri's Experience with RAS in HMA" held in Joplin, Missouri, in September 2008. Missouri is also the lead state in a proposed national Transportation Pooled Fund Study to do research on RAS in HMA to encourage other state department of transportation acceptance and use of this innovative, cost saving, and environmentally friendly technology.

Key words: additive—asphalt binder—post-consumer—reclaimed asphalt shingle (RAS)

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SUMMARY

In 2002, the Missouri Department of Transportation (MoDOT) received a request by Pace Construction in St. Louis, MO, to use reclaimed asphalt shingles (RAS) from tear-off shingles in hot mix asphalt (HMA). We began an investigation of national and other state specifications to find if others were using shingles in this manner. While shingles had been in use in other states, it was found that tear-off shingles were specifically prohibited by specification or environmental laws in those states. In December 2004, Pace invited MoDOT to observe and test mixture from a project at their plant site. Based on favorable volumetric and stripping test results, a pilot project was placed on Rte. 61/67 in St. Louis County in 2005. A specification to allow RAS in all HMA mixtures using a non-modified asphalt binder (PG 64-22) were incorporated into the Missouri Standard Specifications in February 2008 as a result of this pilot and additional testing by MoDOT and other agencies.

The primary MoDOT concerns from adding tear-off shingles to HMA have been demolition debris and resistance of the mixture to fatigue and cold weather cracking due to the shingle binder being much stiffer than roadway asphalts. By limiting the amount of RAS in a mixture, the effect of both of these can be somewhat offset. Originally, the deleterious material content was limited to 0.5%. The limit was raised to 3.0% based on demolition material that was picked as cleanly as was practical. Wood may only be one-half of this quantity. The small percentage of shingles allowed in mixtures still keeps the deleterious material added by the shingles much lower than allowed in the aggregates. In general, nails are rarely included in this material because the processors effectively remove them.

Binders from the tear-off shingles, new shingles, and roadway were blended to determine the performance grading (PG). When greater than 70% roadway or virgin asphalt was added to the blend, the low temperature grading was not greatly affected by the shingle asphalt. The shingle asphalt affected the blend more rapidly as the percentage of virgin asphalt decreased below 70%. Additional testing of the mixture will be required to determine the actual effect in the mixture. Until this testing is complete, PG 58-28 will be required in mixtures designed for PG 64-22 with less than 70% virgin binder. Mixtures requiring a polymer modified binder may not use shingles due to a lack of information at this point.

Eleven contractors are currently using mixtures that include shingles from tear-offs or manufacturing waste. Ten contractors and recycling companies are processing the shingles for use in HMA. The Missouri Department of Natural Resources (MoDNR) allows them to process the shingles under the National Emissions Standards for Hazardous Air Pollutants (NESHAP) rules, which do not require asbestos testing as long as the shingles are from residential demolition. Some testing has been required by the St. Louis County Department of Health.

Changes made for the 2008 construction season allowed increased use of RAS. The maximum amount was raised from 5% to 7%. By using the 70% limit for virgin binder, mixtures may contain shingles closer to the maximum amount without changing the grade of binder. Also, a standard gradation for shingle aggregates is included to reduce exposure of laboratory technicians to the high levels of dust after removal of the asphalt binder. Twenty percent of the projects in 2008 used mixtures containing RAS. The average amount of shingles in those mixtures was 3.5%. Figure 1 demonstrates the increased rate of use for RAS.

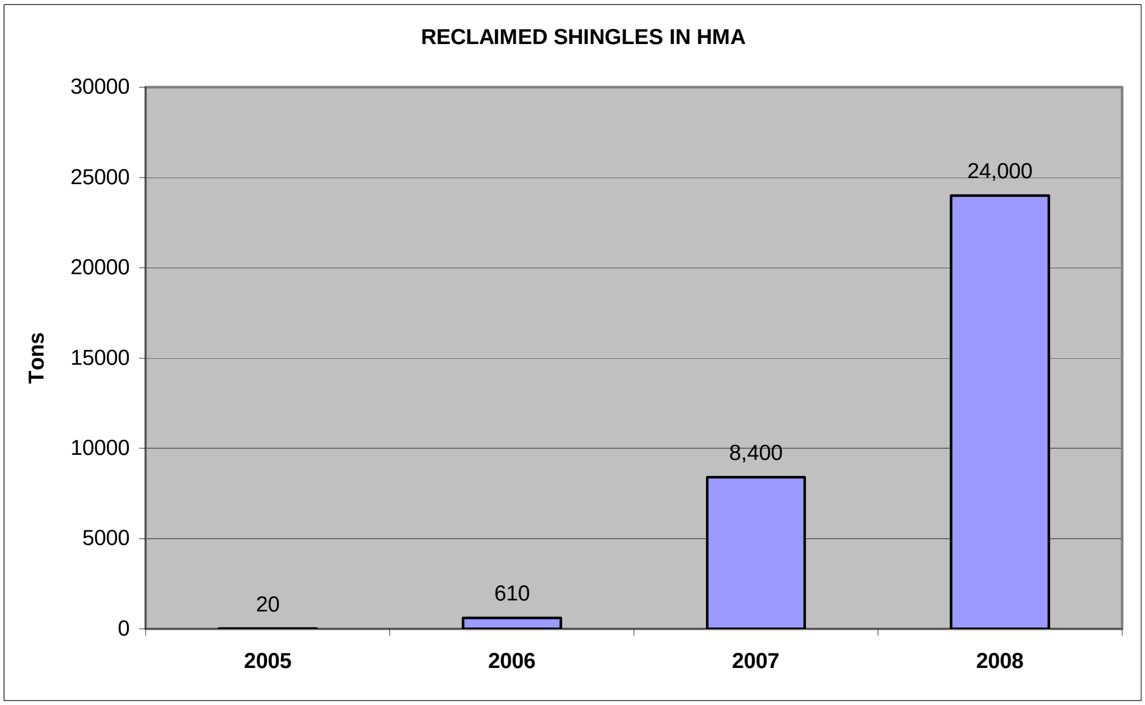

Figure 1. Increased rate of use for RAS

DISCUSSION

Landfills are filling up not only with garbage but also with material from construction and demolition projects. With landfill permitting becoming more difficult and expensive, owners are shifting their focus from disposing of everything to reclaiming materials for reuse or new uses. A particular problem with landfilling asphalt shingles is that if they are not properly dispersed, they can form an impenetrable layer that will trap methane gas. One solution is to remove the shingles from the waste stream.

On December 12, 2002, Roger Brown of Pace Construction, St. Louis; Dale Ann Behnen of Peerless Resource Recovery, Valley Park; and Dan Fester of the Missouri Department of Natural Resources (MDNR) made a presentation to MoDOT personnel about the possible use of RAS in HMA mixtures. First impressions on the use of the material were not favorable. The sample of ground shingles had a large amount of material considered deleterious such as wood, plastic, fiberboard, and insulation. Roger also had specimens of HMA made in one of Pace's plants where 20% of the aggregate portion was replaced by RAS. The specimen had the appearance of one that had been lying in the sun for a considerable time. Dan stated that some asbestos testing had been completed with negligible results and that MoDNR's opinion was that as long as proper monitoring of the material is performed, asbestos would be of no consequence. They were informed that MoDOT would look into the use of RAS.

Swift Asphalt, at their plant in Joplin, Missouri, has used manufacturing waste in their commercial mixtures for a number of years. There are no known complaints of poor performance from these mixtures; however, there has been no documentation of performance.

Asphalt shingles typically are manufactured with approximations of the following ingredients: 20% asphalt, 15% fiberglass or organic matting, 25% mineral filler and 40% granules. Post-consumer or tear-off shingles have lost granules during their life on the roof resulting in a content of typically 30% asphalt. The asphalt in shingles has the same origins as the asphalt used in roadways. However, it has been stiffened in order to perform differently in the shingles. More discussion on this issue is to follow. The aggregates used for the granules are generally harder than those used in Missouri in HMA, therefore, are beneficial to the mixture. Neither mineral filler nor matting will have any detrimental effects to HMA.

A search of other states' construction specifications showed some states allowed the use of manufacturing waste but were explicit in disallowing tear-off shingles. North Carolina was the only exception but when the North Carolina Department of Transportation was contacted, it indicated that none of its contractors used the tear-offs. Later it was discovered that the testing frequencies mandated by the North Carolina Department of Environment and Natural Resources were too stringent to make recycling economically feasible.

The primary concerns with RAS in HMA are the hardness of the asphalt, deleterious material, and the potential presence of asbestos. Roadway asphalts are generally soft in comparison to the asphalt used in shingles. These asphalts have to be stiff enough to resist pavement rutting in the summer months but soft enough to resist fatigue cracking due to repeated loading as well as cracking due to cold weather shrinkage of the pavement. Deleterious material may degrade the performance of the pavement or increase moisture damage in the mixture. Nails in tear-off shingles are removed by using a magnetic head pulley on the conveyer during the grinding operation. Complete removal of the nails can be attained by using multiple passes over the magnetic pulley. While asbestos was an initial concern, this is an issue that has been put to rest in Missouri.

For clarification of the remainder of this document, the following section of this report includes terms which will be defined.

Other foreign material—Deleterious material (shown in Figure 2) that is not inherent to the aggregate or RAS. In a demolition project, this may include any building material where in a re-roofing project it is most likely packaging, plywood, bush trimmings, and other trash generated on the work site. A typical specification for other foreign material in asphalt aggregate is a maximum of 0.5%, which is MoDOT's specification. Nails are to be included as deleterious material.

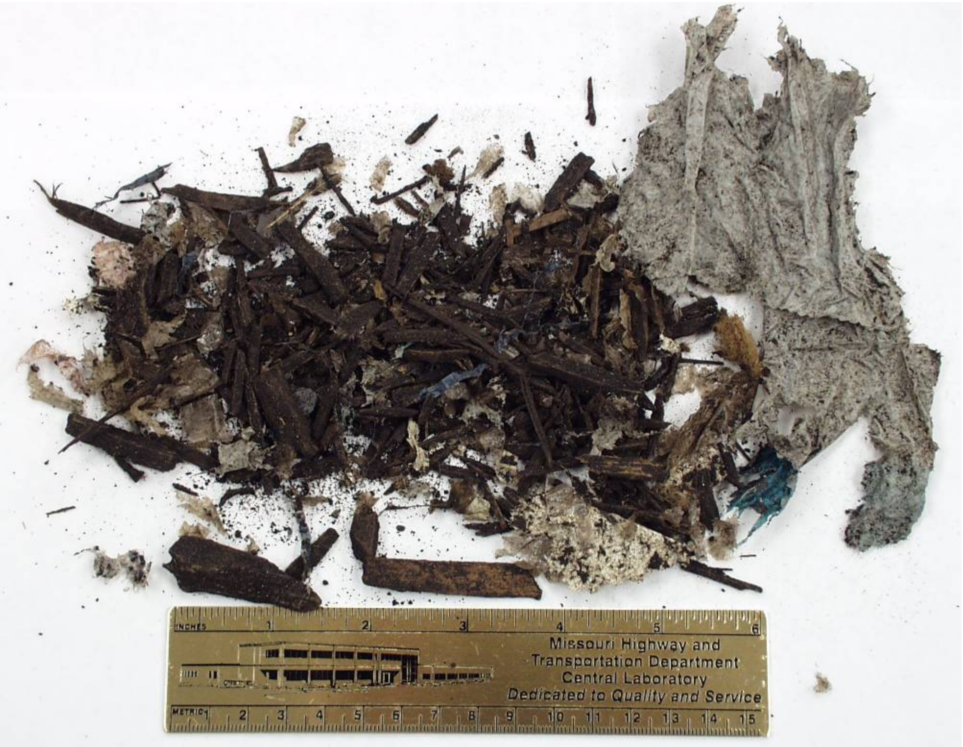

Figure 2. Other foreign materials

Binder—Asphalt in a mixture is referred to as binder since it binds the components together. Roadway binders are classified by performance grade (PG), which describes the high and low temperature operating temperature in degrees Celsius. Missouri's standard paving grade of PG 64-22, 64 minus 22, should perform well over a range from 64°C to -22°C. The high temperature allows for stiffness to minimize rutting during hot weather. The low temperature allows for flexibility in cold weather to minimize cracking. The PG grade is changed in increments of 6°C. Heavier traffic creates more potential for rutting. A stiffer binder is specified by raising the high temperature grade in order to offset the increased traffic loading. Therefore, interstates are paved with PG 76-22, which is the stiffest binder specified for Missouri's highways. To demonstrate the difference in hardness between a roadway binder and shingle binder, pictures were taken of the two binders side by side. A ball of PG 76-22 was rolled up and set beside the shingle binder as it was originally when poured from a can four years ago. As can be seen in Figures 3 through 5, the shingle binder has retained its shape while the roadway binder flattened out in a relatively short time frame.

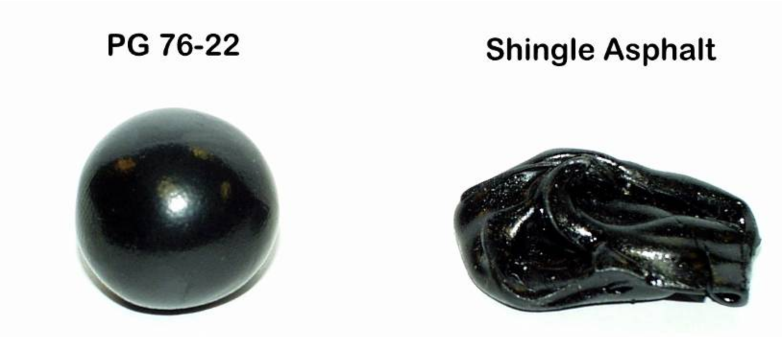

Figure 3. Original forms of the roadway and shingle binder

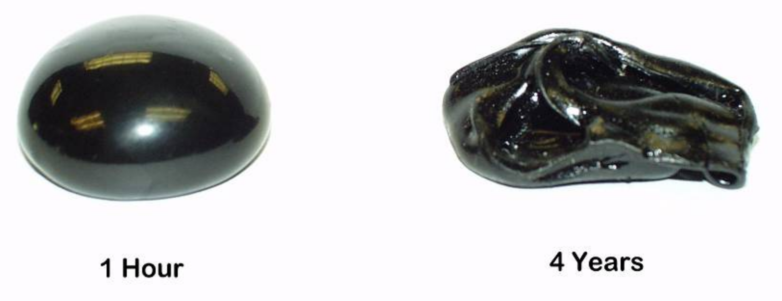

Figure 4. The roadway binder after one hour and the shingle binder after four years

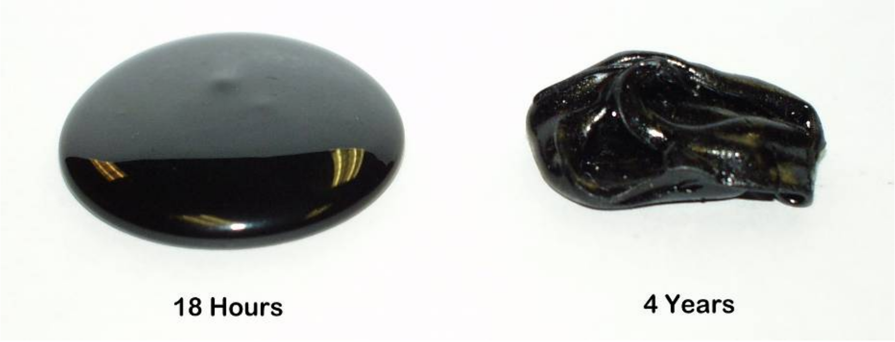

Figure 5. The roadway binder after 18 hours and the shingle binder after four years

In order to understand the properties of the binders after mixing in the plant, several different grades of roadway binder were blended with the shingle binder to PG grade. The binder from tear-off shingles was obtained by chemical extraction. Binder representing shingle waste was donated by TAMKO Building Products, Inc., in Joplin, Missouri. This is called flux in the roofing industry. The material was obtained prior to incorporation into the shingle manufacturing process. Roadway paving binders blended with the shingle binder were provided by Conoco Phillips Company, Wood River, IL; and Ergon Asphalt and Emulsions, Kansas City, MO. Since it was known that the shingle binder is much harder than a roadway binder, blending was performed using softer PG grades than the standard PG 64-22 since the goal was to maintain the -22 grading. The tear-off binder was so hard, in fact, that a cheese grater had to be used to get the binders to properly blend. A summary of the true grade (degrees Celsius) determined for each binder and blend is shown in Table 1.

Table 1. Summary of the true grade determined for each binder and blend

			Percent Virgin Binder in Blend								Shingle Binder			
PG Grade	**Binder**		**80%**		**60%**		**40%**		**20%**		**Tear-off**		**Flux**	
58-22	59	-28	73	-25	108	-17	105	-1	123	+8	143	*		
52-28	56	-31	64	-28	80	-19	99	-4	126	*	143	*		
58-28	60	-30	73	-24	78	-14	107	+2	123	+43	143	*		
58-28	60	-30	68	-22	79	-16	86	-10	98	-4			116	+8

* indicates temperature above testing limits.

Low temperature cracking remained the primary concern after evaluating the results of the blending. The true grade determined for the shingle binders were above 0°C (32°F.) The tear-off binder results were higher than could be determined by the equipment. Blending with the flux, new shingle binder, was fairly linear through the range of normal use. It was noted that the effect of the tear-off binder increased at the same rate as the flux at higher percentages of virgin binder. Between 60% and 80% virgin binder, the tear-off binder began to take control of the low temperature properties, as seen in Figure 6.

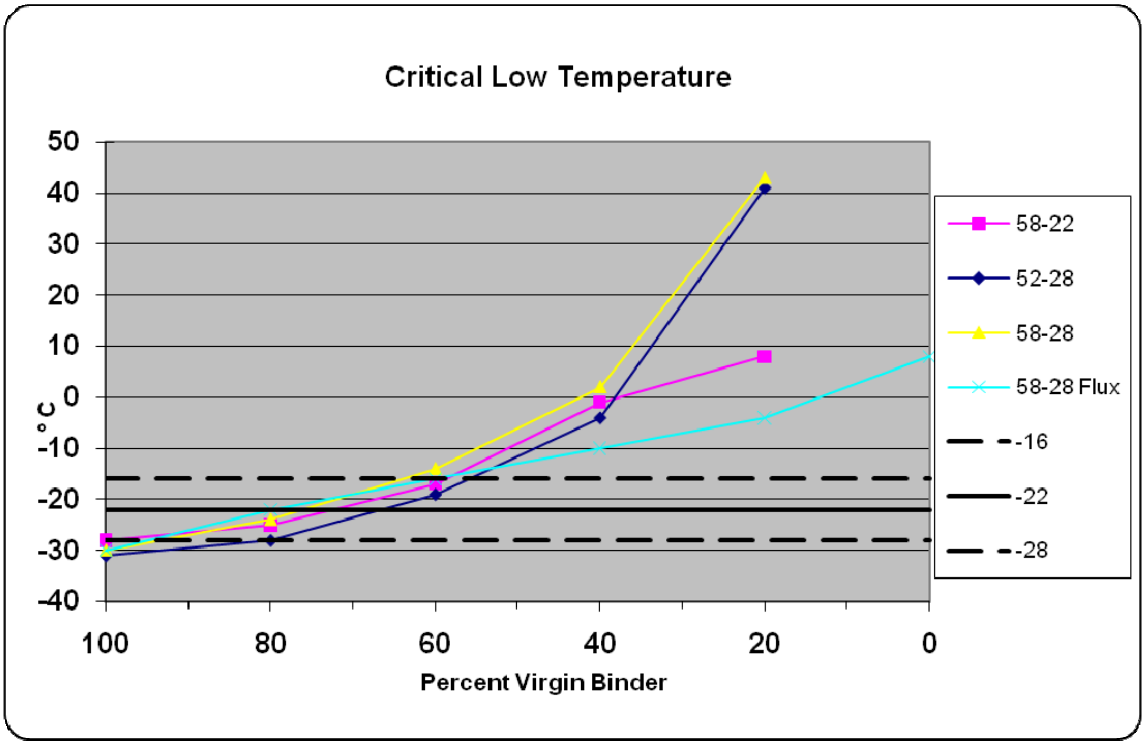

Figure 6. Critical low temperature

Pace Construction set up a demonstration at their plant location known as Bussen Quarry, Antire, in December of 2004. Pace was paving the roadway into the quarry, which is in the St. Louis commercial hauling area. The ground shingles used for the trial were tear-offs processed at Peerless. Peerless had collected the shingles as part of the construction and demolition landfill at their location. They had been handpicked to remove as much of the other foreign material (OFM) as was possible and cost-effective. Initial samples contained approximately 10%, but the amount was reduced below 3%. Roger Brown had set up the demonstration to show that the OFM would have no deleterious effects in the mixture. Also, he wanted to see how the mixture performed using the standard PG 64-22 instead of using a softer grade. The mixtures used for the demonstration were 12.5 mm and 19.0 mm (MoDOT SP125 and SP190) Superpave designs with 5% RAS. Four in. of SP190 and 2 in. of SP125 were placed. There were no visible differences noticed during production and placement. Normal volumetric tests revealed no difference, as did moisture susceptibility testing by AASHTO T 283. The RAS contained pieces of plywood up to 2 in. long and 1/2 in. wide. The only evidence of the wood found in loose mix samples were basically splinters approximately 1/4 in. long. A visual examination of the roadway in the fall of 2008 revealed no distress in the roadway.

Based on a successful demonstration by Pace Construction (Figure 7) and use by other agencies, a material special provision was prepared for use by contractors. The primary difference between MoDOT and other agency specifications was that no distinction was made between tear-off and manufacturing waste. The RAS had a limit of 5% whether used in the mixture as the only reclaimed asphalt or in conjunction with recycled asphalt pavement (RAP). When added with RAP, the limit of RAP is reduced by the amount of RAS before additional binder tests are required to determine the virgin binder grade. The maximum deleterious content for OFM was set at 3%, with a limit of one-half of that amount being wood. Asbestos had no restrictions other than those by other regulatory agencies. Also, when considering a 5% to 10% RAS allowance in HMA, the maximum amount of deleterious content contributed by the shingle would only be 0.3%, which is below the 0.5% allowed in the aggregate. Using data from binder blending, RAS in HMA required PG 58-28 as the virgin binder.

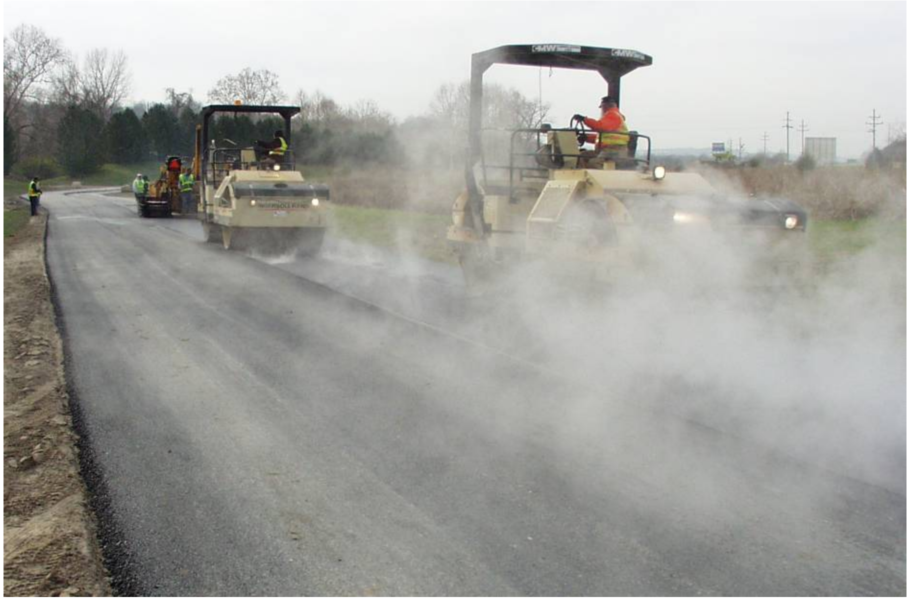

Figure 7. Pace construction demonstration

Softer asphalt grades are generally unavailable in the St. Louis area or have a higher cost removing the incentive to use RAS. Roger Brown requested substituting shingle mixtures for some of the binder course, SP190 containing PG 70-22, on Rte. 61/67 or Lindbergh Boulevard. He wanted to see if there were any performance differences in using PG 64-22 as opposed to the PG 58-28 required in the special provision. In August of 2005, a trial was set to look at the combinations shown in Table 2 with the remainder of the project as control using 20% RAP and the PG 70-22. Figures 8, 9, 10, and 11 show views of Lindberg before construction and after construction began.

Table 2. Locations and percentages of RAS and RAP used

PG Grade	Percent RAS	Percent RAP	Location
58-22	0	20	Ronnie Lane to Workbench Dr.
58-22	5	15	Workbench Dr. to n/o Gravois
64-22	5	15	Gravois to Lindbergh H.S.
64-22	0	20	Lindbergh H.S. to Fox Meadows

Pace Construction’s work consisted of cold milling 3.75 in. from the existing surface and replacing with 2 in. of SP190 and 1 3/4 in. of SP125. The trial sections were placed south of Rte. 366 basically between Ronnie Lane and Lindbergh High School in the southbound driving lane and Lindbergh High School to Fox Meadows in the northbound driving lane. Pace hauled the PG 58-28 from the SemMaterials terminal in Tulsa, Oklahoma, since it was not available locally at the time of the paving.

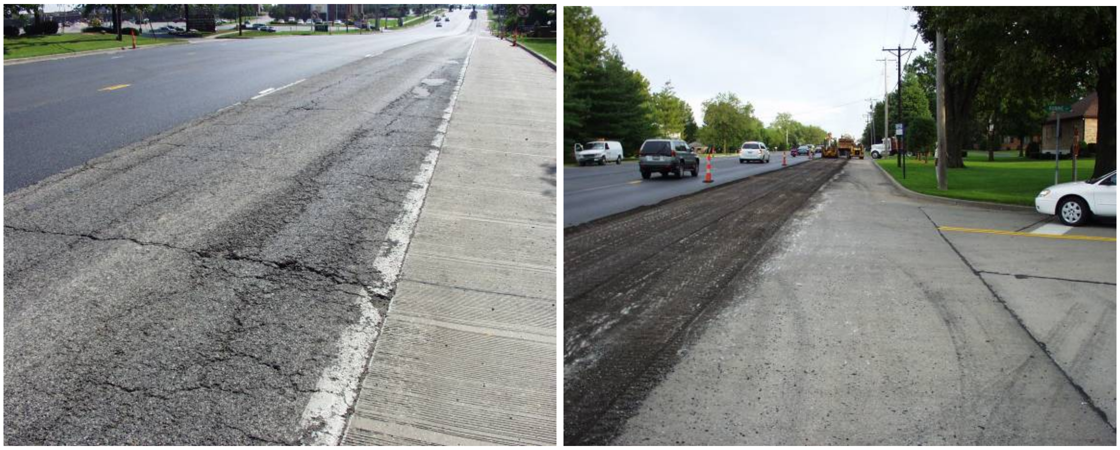

Figure 8. Lindbergh before paving (left) and Lindbergh during construction (right)

While this project was being paved, the Minnesota Department of Transportation (Mn/DOT) was paving a trial project comparing tear-off shingles and manufacturing waste. The University of Minnesota offered to perform indirect tensile strength (IDT) testing in conjunction with this study so Mn/DOT could look at cold weather cracking potential. The results of this study were published in the Association of Asphalt Paving Technologists (AAPT) Journal in 2007. The stiffness of the shingle mixtures exceeded the capability of models to accurately determine the temperature at which cracking would occur. With the increase in stiffness also came an increase in strength, which may explain why the conclusions of the report have not been realized in the field.

An evaluation observing rutting and reflective cracking was made after two years in service. Since the RAS mix was in a subsurface layer, these were the only observations made. No rutting or cracking was noted in the experimental sections as there was none noted in the other areas of the project. At the three year observation in the fall of 2008, no rutting was observed, but cracking had begun to appear in some of the control sections. Two cracks noted in the RAS section with PG 64-22 just south of Gravois Road may be the result of pavement geometry due to pavement widening and the end of the concrete shoulder. The following photographs show the typical appearance of the roadway at that time and cracking that was beginning to appear in the center lanes of the roadway. It was noted the transverse cracking in the center and passing lanes stopped at the joint adjacent to the driving or near lane, which contained RAS.

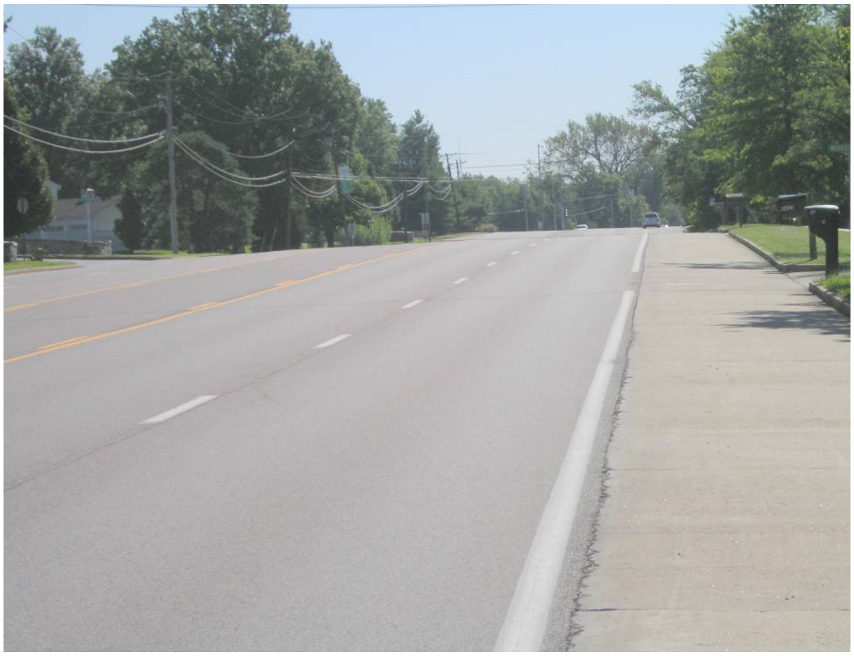

Figure 9. Typical appearance of Lindbergh, 2008

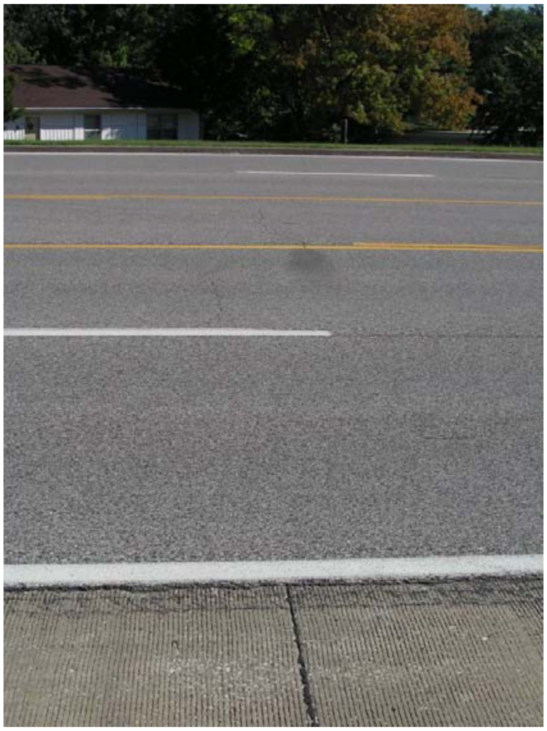

Figure 10. Cracking in center lanes of Lindbergh, 2008

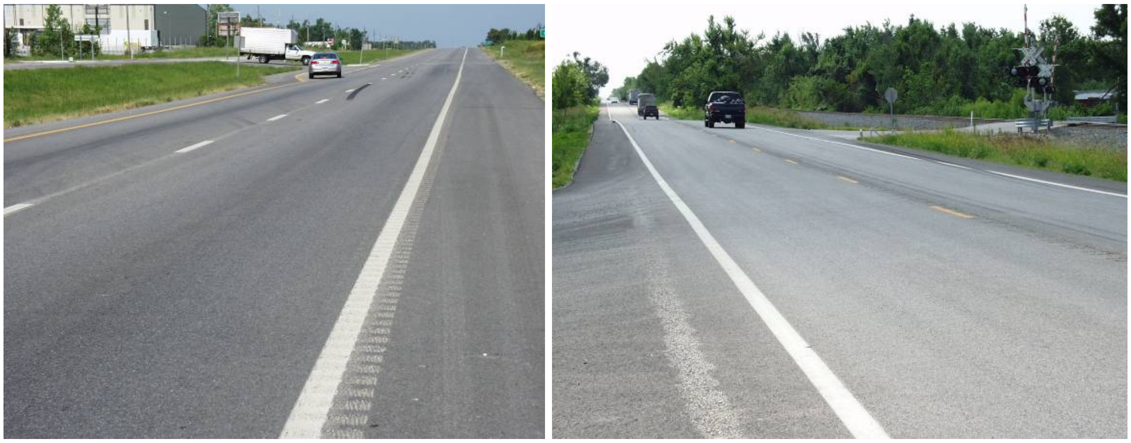

Figure 11. MO Rte. 66 (left) and MO Rte. 171 (right)

The special provision used in the experimental project was incorporated into Section 403 of the Missouri Standard Specifications for Highway Construction effective February 1, 2008. Two changes made as a result of the Lindbergh project and other states' experiences were that the limit for RAS was raised to 7% and PG 64-22 was allowed as long as the virgin binder content remained at or above 70% of the total binder. Since the amount of RAS is limited, large changes in the gradation of the RAS aggregate makes small changes in the combined gradation of the mixture. The large percentage of mineral filler in the shingles causes determination of the gradation to be messy in the laboratory. Several gradations determined in the MoDOT Central Laboratory were compared with contractor gradations to set a standard gradation that was added to the specification as an option for contractors.

Asbestos has been a chief concern of many agencies nationwide. Use of asbestos was discontinued in the early 1970s in residential shingles in the United States. The National Emissions Standards for Hazardous Air Pollutants (NESHAP) under the United States Environmental Protection Agency (EPA) has an exemption based on this fact. In the Appendix of Code of Federal Regulations (CFR) Section 40 Subpart M, shingles from four-plex or smaller residential dwellings are exempt from asbestos testing in accordance with local regulations. The Michigan Department of Natural Resources (MDNR) has no additional testing requirements for the shingles, but some local agencies, such as the St. Louis County Department of Health, have a requirement to prove the absence of asbestos. Tests performed on shingles to date by MoDOT and processors in St. Louis County have been clear of asbestos for the most part. A few tests have had a trace of asbestos but no measurable levels. It is believed the asbestos was contained in mastics used for sealing joints in roofs. Most shingle processors in Missouri document the source of the shingles but do not routinely test following the NESHAP guidance.

The projects constructed thus far, including RAS, have not shown a reduction in performance to this point. A more detailed evaluation is planned for projects after more time has elapsed. The primary concern in using RAS from tear-off shingles from the beginning has been cold weather and fatigue cracking. There has been little cracking noted in projects since placement, even with temperatures approaching -20°C (-4°F) with the critical cracking temperature of -22°C (-7°F).

Few problems have been reported on construction projects. Most contractors have progressed toward using finer grinds of the RAS in order to prevent plant feed problems. The normal introduction of RAS into the mixture is through the reclaim port on the plant, which is usually about a 2 ft square. Mixture tenderness evidenced by movement of the asphalt during placement has occurred in some mixtures necessitating a reduction in the amount of RAS. More contractors are beginning to be aware of the amount of moisture held by ground shingles since they can hold as much as 25% of their weight in water.

Cold weather placement has resulted in low pavement density, and when used in conjunction with warm mix asphalt, clogging at the plant discharge has been reported.

The use of RAS grew rapidly from 2006 through 2008. The large increase in 2008 was fueled by a steep increase in asphalt prices. In one year, the price rose from just under $400 per ton to over $900 per ton in the fall of 2008. As the contractors sought ways to cut costs, a reduction in the virgin asphalt content appeared to be the most reasonable. A 20% to 30% reduction in virgin asphalt can be achieved in most mixtures within the 7% RAS limit. As processing facilities increase, more and more people are becoming environmentally aware while looking to recycle shingles in order to avoid more expensive landfill costs. Figure 12 shows the use of any means to recycle shingles.

Figure 12. Using any means to recycle shingles

Since long-term performance data are unavailable, the unknown has been the main slowdown in use of the RAS in HMA, not only in Missouri, but nationwide. A comprehensive study comparing laboratory testing with results on the roadway does not exist for tear-off residential shingles. The results from tests appear to be contradicted by performance. Testing would indicate a drastic rise in low temperature cracking; however, the pavements have not exhibited this behavior. The unknowns are the actual amount of blending between the harder shingle asphalt and roadway asphalt. Also unknown is the contribution of the fibers and filler in the RAS. To know more about these unknowns will require testing of the entire mixture and cannot be determined by only testing the binders.

CONCLUSIONS

One indicator of the performance of RAS mixtures is the determination of the dynamic modulus of an SP125 mixture containing 3% RAS in the Asphalt Mixture Performance Tester (AMPT). Comparing the master curve to that of another SP125 mixture containing 20%RAP, the performance level should be equal, as shown in Figure 13.

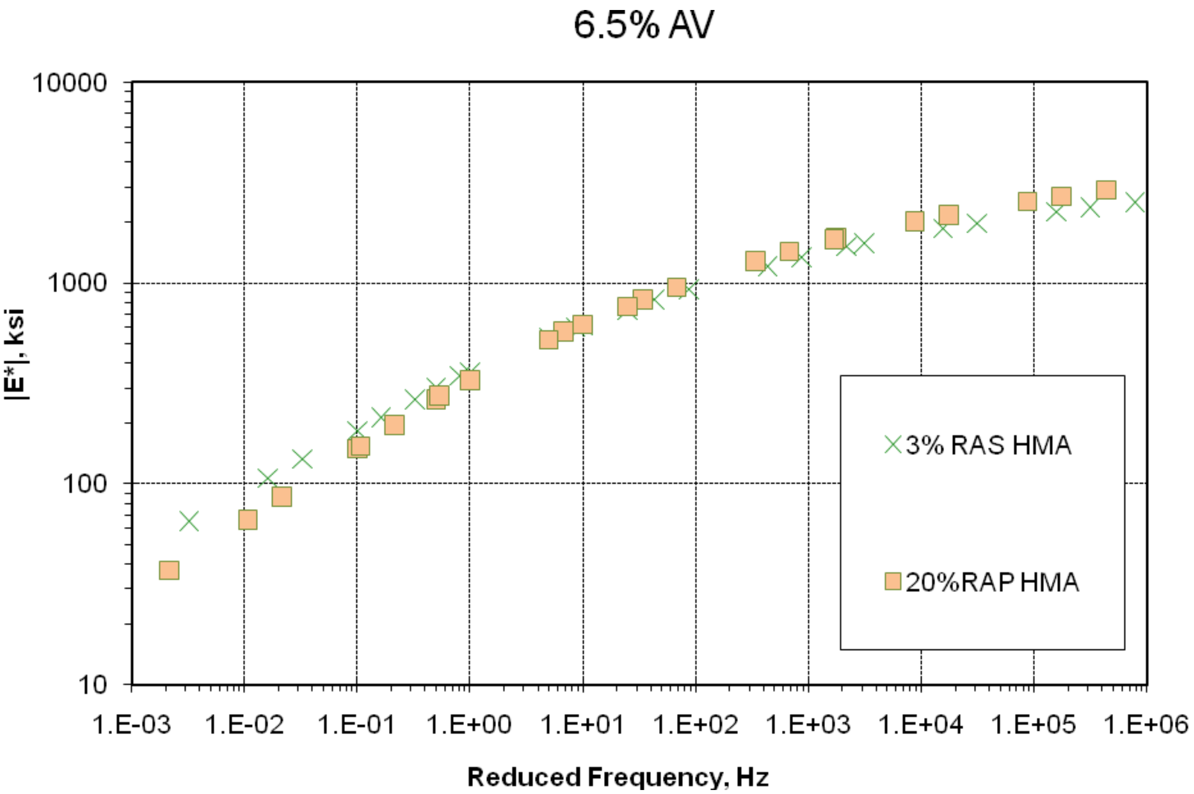

Figure 13. Determining the dynamic modulus of an SP125 mixture containing 3% RAS in the Asphalt Mixture Performance Tester compared to the master curve of another SP125

The introduction of RAS into HMA mixtures came at an opportune time for MoDOT. It is hard to evaluate the cost savings due to various decisions made by the contractor, but all but one or two of the suppliers of HMA to MoDOT projects are now using RAS. At the same time that the RAS specification became the standard, the cost of asphalt binder began its rapid rise and MoDOT adopted environmental responsiveness as a tangible result of its core values. Reduction of landfill waste was not MoDOT’s goal in the use of RAS, but it has led to an emphasis on use of other recycled materials that not only make use of valuable resources but are taken out of the waste stream. As landfills run out of space, other environmental regulations and public opinion are discouraging the construction of landfills. Beneficial use of waste materials helps MoDOT to both control costs and fulfill its mission as a public agency.

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Evaluation of Technology-Enhanced Flagger Devices: Focus Group and Survey Studies in Kansas

Chen Fei See
Transportation Research Institute
University of Kansas
2160 Learned Hall
1530 West 15th Street
Lawrence, Kansas 66045-7609
cfsee@ku.edu

Steven D. Schrock
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
2159B Learned Hall
1530 West 15th Street
Lawrence, Kansas 66045-7609
schrock@ku.edu

Wai Kiong "Oswald" Chong
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
2134C Learned Hall
1530 West 15th Street
Lawrence, Kansas 66045-7609
oswald@ku.edu

Yong Bai
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
2135B Learned Hall
1530 West 15th Street
Lawrence, Kansas 66045-7609
ybai@ku.edu

Jamila Saadi
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
2160 Learned Hall
1530 West 15th Street
Lawrence, Kansas 66045-7609
tunis123@ku.edu

ABSTRACT

Flagger-controlled work zones are often in place for only a short duration, so adding protection beyond the minimum guidance is rarely done. Several vendors have begun marketing devices equipped with various technologies. While a wide variety of studies have been undertaken to evaluate the technology-enhanced flagger devices, there has been little effort to examine these devices on the basis of perceived usefulness to field personnel and understanding by motorists. This study was aimed at obtaining responses from field personnel regarding the perceived usefulness of these devices, while synthesizing the effects of these devices based on flagger focus groups and driver survey responses.

The focus groups results revealed that weight of devices, conspicuity of flaggers, and awareness of drivers were among the influential criteria for field personnel to opt for a flashing STOP/SLOW paddle. Additionally, 72% of participants agreed that 4 in. red/amber light appeared to have best potential for visibility gains and versatility of applications. From the surveys, only 28% of drivers indicated that they saw the STOP sign or flagger in work zones. When asked about the displayed STOP sign, 74% of in favor drivers stated that it commanded their attention or fulfilled a need, whereas 86% of those not in favor indicated that they either did not see it or thought it was hard to see. More than half of the surveyed drivers did not think that the flashing paddles indicated a more important situation. Only 26% of drivers stated that they drove differently because of the flashing paddles.

Key words: flaggers—focus groups—temporary traffic control—work zone safety

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INTRODUCTION

In the United States, over 1,000 fatalities occur every year in the construction sector (Bureau of Labor Statistics Undated [a]). In 2007 alone, approximately 94 of these fatalities were highway- , street-, and bridge construction-related, whereas 30% (or 28 fatalities) of these incidents were due to construction workers struck by vehicle (Bureau of Labor Statistics Undated [b]). Specifically, highway maintenance workers consisted of approximately 13% of these fatalities in the year 2007 and 50% of these fatalities were reported as workers struck by vehicles (Bureau of Labor Statistics Undated [b]). Previous studies revealed that driver inattention and excessive speed are among the contributing factors most frequently reported for work zone crashes (Chambless et al. 2002; Mohan and Gautam 2002).

In a flagger-controlled work zone, the flagger is the key to effective traffic control, and thus, his/her visibility and conspicuity are critical in keeping motorists and workers safe. Nonetheless, these short-duration work zones often utilize fewer traffic control measures than other work zones, so adding signing or positive protection beyond the minimum guidance directed by the *Manual on Uniform Traffic Control Devices* is rarely done. Based on prior findings, approximately 6% of the examined fatalities from years 1980 to 1992 were either flaggers or surveyors (Ore and Fosbroke 1997). In an effort to increase flaggers' visibility and conspicuity, several vendors have begun marketing STOP/SLOW paddles, personal protective equipment, and other ancillary devices equipped with various technologies typically including embedded LED lighting.

Many studies document the usability, advantages, and disadvantages of various new devices or technology as soon as they are available in the market (Trout and Ullman 1997; Fontaine and Hawkins 2001). It is important to understand the functionality and feasibility of these devices and how they will improve the safety of workers in the work place. Devices or technologies such as radar drones, intrusion alarms, and flashing LED lights have been receiving the attention of the industry for over a decade. Nevertheless, the effectiveness of these technology-enhanced devices tends to be evaluated on the basis of observational tests, speed tests, or even their availability. While the effectiveness of these devices is important, understanding of motorists and the underlying reasons for specific precaution measures, such as slowing down or stopping earlier, are also critical. There are many factors that can influence the drivers to decrease their speeds or take precaution measures. However, only the drivers themselves can explain what was actually happening. Aimed at obtaining the true effectiveness of each selected technology-enhanced flagger device, this study used the responses from field personnel regarding the perceived usefulness and workability of these devices, while synthesizing the effects of these technology-enhanced devices based on driver survey responses.

LITERATURE REVIEW

Morena (2002) evaluated five different flashing STOP/SLOW paddles for a group of federal, state, and local highway workers in Wisconsin. Specifically, these STOP/SLOW paddles, including a halogen lights paddle, were tested in bright sunlight at 285 feet from the observers. The results of the tests revealed that only the halogen lights paddle commanded the attention of drivers from as far as 285 feet or greater. Additionally, a contractor from Princeton, Wisconsin, commented that these flashing STOP/SLOW paddles should be used in all areas where there is high traffic volume and low visibility. The interviewee added these highly visible paddles can help to increase the conspicuity of flaggers, while improving the safety of workers in work zones. The only drawback observed in these flashing paddles was the cost. The author revealed that the extra cost of these flashing STOP/SLOW paddles was priced from $175 to as high as $530 for the halogen paddle. In essence, the question regarding the worthiness of the flashing STOP/SLOW paddles investment lay upon individual work needs and site conditions.

In recent years, many new techniques or devices were proposed for implementation to improve flagger safety in work zones. In Texas, Trout and Ullman evaluated ten new devices or technologies to improve worker safety in work zones: opposing traffic lane dividers, drum wraps, direction indicator barriers, radar drones, water-filled barriers, blinking reflectors, portable curbs, portable rumble strips, intrusion alarms, and queue length detectors. In a newer report, Fontaine and Hawkins cataloged and added several devices that produced positive impacts in short-term work zones: fluorescent yellow-green worker vests and hard hat covers, portable variable message signs, speed display trailers, fluorescent orange signs, radar-activated flagger paddles, radar drones, retroreflective magnetic strips for work vehicles, portable rumble strips, and worker strobe lights. In both of these reports, radar drones repeatedly appeared as the device that had the potential to improve work zone traffic control. This device helped decrease vehicle speeds by emitting a K-band radar signal, which can be detected up to a mile through the radar detectors. The radar drones can incessantly emit radar signals by powering it through the car cigarette lighter until turned off. However, no significant reductions in average speeds of the approaching and traveling vehicles were found in the study. Fontaine and Hawkins added that radar drones may be suitable for rural work zones as it provides limited benefits in speed reductions.

The radar-activated flagger paddle was another device that was found to be useful in improving flaggers' safety. This prototype device was developed and modified exclusively by the Texas Transportation Institute (TTI). The paddle was comprised of a plastic STOP sign that was modified by incorporating detectors and LEDs in the sign face areas to detect vehicles traveling above the preset speed range. The device was observed to have a few usability problems. First, the unit was top-heavy due to the location of the battery within the sign face. Second, the wiring of the radar that was exposed to the elements was found to be very fragile. The research concluded with recommendations to further improve and examine the effectiveness of this device.

The intrusion alarm was another device that was proposed by Trout and Ullman to improve work zone safety. This device was intended to detect vehicles that breached into the buffer area in a work zone. The alarm of this device will sound almost instantly to warn construction crews regarding the potential danger if intruders were detected. There are three types of intrusion alarms available in the market: microwave transmissions, infrared light beams, and pneumatic tubes. The microwave and infrared models can be set up simply by mounting them on the traffic cones or traffic drums, whereas the pneumatic tube systems require the tubes to lie flat on the roadway. All three intrusion alarm models had not been receiving positive reviews; many issues regarding the functions and workability of this device were revealed. False alarms were the major concerns that were repeatedly reported in all three models. The alarm system of the infrared unit was found to be overly sensitive and resulted in many false alarms, whereas the pneumatic tube systems were found to be ineffective in warning the workers. Similar issues were observed in the microwave models, where false alarms could be triggered by movement of the drums, rain, or even dust. The ease of use for all three models was rated as low by various agencies as the required setup time was longer than anticipated, and on top of that, these devices were neither lasting nor reliable. However, researchers believe that there may be still room for intrusion alarms to improve by providing training programs to construction workers regarding the pros and cons of each device under certain conditions.

FOCUS GROUP EVALUATIONS

Focus group meetings were conducted in order to gain a better understanding of the views and opinions of the highway contractor flaggers, department of transportation maintenance personnel, and emergency services personnel, such as police and firefighters, regarding the current state of technology available for flaggers, any experiences with previous technologies that they may have tried, and the innovations that they found helpful in promoting increased driver compliance and/or worker safety. Focus groups have advantages over other survey methods in that they are able to cover a topic in more depth, and due to the open-ended nature of the discussions, the potential exists for innovative concepts to be suggested by participants (University of Texas Undated).

Three focus groups were conducted in Kansas in order to provide a diverse group of participants. Focus groups were conducted in LRM Industries, Inc., Lawrence Fire Department, and Douglas County Public Works. Four techniques were used during the focus group discussions:

- **Listing.** Participants in each focus group were asked to list the training and equipment they received or used in order to perform in temporary traffic control work zones.
- **Evaluating.** After each participant listed the equipment they used, participants evaluated eight different technology-enhanced flagger devices and three standard traffic control equipment.
- **Ranking.** The participants were also asked to rank the equipment in their category based on the usability, conspicuity, durability, preference, and importance. The ranking was to assess the relative preference and importance of the equipment.
- **Building desirable technology-enhanced flagger device.** At the end of each focus group, participants were asked to create a flagger device that they think would be helpful in assisting the flaggers in flagging operations.

As the final step of the focus group discussions, participants were guided to answer each selected question before an open-ended discussion. This measure was intended to obtain consistency in the data, while providing participants an opportunity to relate their duties to the technology-enhanced devices before any discussions. Additionally, the measure also helped minimize any biases that may arise during the discussions. During each questionnaire session, focus group participants were asked to rank the pros and cons as well as the perceived usability, conspicuity, durability, and preferred technology-enhanced flagger device. This part served as the important data collection section, which will be used to compare against the results of the motorist surveys.

The panel evaluation sessions comprised of four parts. The first part was an orientation to understand participant backgrounds, responsibilities, and devices that they have while performing the flagging operations. The second, third, and fourth part consisted of the following:

- Important criteria of technology-enhanced flagger devices
- Perceived usefulness and effectiveness of the technology-enhanced flagger devices
- Flagger device-creating sessions

Once the participants had undergone a brief orientation, the first question that was asked in the focus group session was to rank the pros and cons of the flashing STOP/SLOW paddle as opposed to a standard paddle on a scale from most important of 1 to least important of 5. In an effort to maintain the accuracy and consistency in the data, only the scores of focus group participants that indicated they used the STOP/SLOW paddle in traffic control operations were included for analysis (e.g., firefighters and police were not included in this analysis). Participants revealed that the following issues were most important to them regarding the use of the STOP/SLOW paddles:

- Weight
- Conspicuity of flaggers
- Alerting drivers sooner

Table 1 shows the results described. On the basis of the results, conspicuity and mobility of flaggers emerged to be top priorities for participants in these focus groups. Overall, the safety of flaggers and longevity of the flagging operations were found to be the most important criteria.

Table 1. Results for question four: "What are the advantages/disadvantages of the flashing STOP/SLOW paddle compared to a standard paddle?"

Advantages/Disadvantages	Average Ranks
1. Weight	1.63
2. Increase conspicuity of flaggers	1.75
3. Alert aging and inattentive drivers sooner	2.00
4. Command respect	2.13
5. Mobility	2.13
6. Positive accident prevention tool in work zones	2.13
7. Battery life	2.25
8. Works great at dawn, dusk or night	2.25
9. Positive protection for flaggers	2.50
10. Cost	2.63

Note: Based on the following scale:
1 = most important, 2 = very important, 3 = important, 4 = somewhat important, 5 = least important

At two focus groups, participants were asked which STOP/SLOW paddles were the most desired devices that they would use in a flagging operation (police and firefighters were not asked this question). Most respondents ranked the standard 24 in. STOP/SLOW paddle the highest in categories of usability (1.17), durability (1.42), and preferred (1.17), whereas in the category of conspicuity, Paddle C received the highest votes with an average score of 1.83. Table 2 shows the results described.

Table 2. Results for question five: "How would you rank these four STOP/SLOW paddles?"

	Paddle A	Paddle B	Paddle C	Paddle D
Usability	1.17	3.00	3.08	2.83
Conspicuity	2.92	2.83	1.83	2.75
Durability	1.42	2.92	3.33	2.92
Preferred	1.17	3.33	3.25	2.75

Note: Based on the following scale:
1 = most desired, 2 = very much desired, 3 = desired, 4 = somewhat desired, 5 = least desired

The open-ended discussions included issues such as the perceived usability, effectiveness, and advantages or disadvantages of each STOP/SLOW paddle. Responses to these discussions yielded similar results that flashing STOP/SLOW paddles were perceived to help increase the conspicuity of flaggers. However, the basic usability and durability issues such as inadequate weight and fragile electronic components were concerns mentioned by focus group participants. A few participants in each focus group explained that during strong wind and high-volume conditions, the weight and "top-heaviness" of some of the paddles have a direct effect on their productivity and worker fatigue. While heavy paddles may hinder the workability of flaggers, one participant pointed out light-weight paddles are not perfect either. The participant stated that during strong wind conditions, “wobbly” paddles may be hard to control.

In one device-building session, one flagger suggested the STOP/SLOW paddle be equipped with a built-in radio. The participant revealed that normally they would hold the STOP sign in one hand and the radio on another, while looking out for the opposing traffic. By incorporating the radio into the paddle, they can communicate effortlessly with the flagger on the other end, while control the traffic with this new technology.

Following the question-answer and discussion for question five, researchers addressed the standard and technology-enhanced safety vest usability and effectiveness in work zones. As shown in Table 3, the standard fluorescent yellow vest with reflective and orange striping (vest B) was ranked the highest in all four categories of usability (1.50), conspicuity (1.71), durability (1.50), and preferred (1.43). Participant comments revealed that blinking LEDs safety vests did not obtain the popularity as expected from the subjects. Participants pointed out that the battery-powered safety vests did not seem to have the usability and durability advantages over the conventional safety vests. Additionally, the LEDs and wiring components of the safety vests triggered a few participants to inquire about the washability and water resistance capability of these technology-enhanced vests. These comments were the impetus for the research team to actually wash the vests. Observations showed that many of the LEDs were rendered unusable after three washes.

Table 3. Results of question six: “How would you rank these four safety vests?”

	Vest A	Vest B	Vest C	Vest D
Usability	2.07	1.50	2.43	2.57
Conspicuity	2.57	1.71	2.07	2.79
Durability	1.93	1.50	2.57	2.00
Preferred	2.43	1.43	2.29	2.93

Note: Based on the following scale:
1 = most desired, 2 = very much desired, 3 = desired, 4 = somewhat desired, 5 = least desired

Interestingly, one focus group participant revealed that safety vest with zipper (or vest D) seems to be a better design than the traditional Velcro option as it helps mitigate the safety vest readjustment issue for flaggers. In one of the focus group discussions, one veteran flagger explained that the standard fluorescent yellow (or lime color) safety vest, i.e., vest A, is the most preferred color as it prevents the color from fading. However, most of the participants agreed that the combination of the fluorescent yellow vests with reflective and orange striping (vest B) was perceived to be the best among all of the safety vests presented.

Prior to the device-building session, participants were asked to answer their “intent to use” for three other technology-enhanced devices: a 4 in. red/amber light, a 2 in. red light, and a flashing headlight device that could be worn on the head, much like a miner’s light. Due to the functionality differences in each of these devices, participants were instructed to explain their “intent to use” instead of comparing and ranking them. The 4 in. red/amber light was a popular device in all three focus groups with 72% of participants indicating that they would use it in a flagging operation, whereas 22% of the participants revealed that they either had no opinions regarding this device or found it not applicable to them. The response from the discussion yielded interesting results.

Overall, participants perceived that the 4 in. red/amber light can help alert drivers sooner, while increasing the conspicuity of flaggers in work zones. Interestingly, one subject revealed that the 4 in. red/amber light that flashes in red was more effective in getting drivers to stop than those in amber color. Additionally, the same participant pointed out that the device can be more effective in alerting drivers by flashing the word "STOP" instead of just flashing in red or amber color. While the 4 in. red/amber light received positive results, the open-ended discussions from another focus group revealed that this device may be useful in isolated areas to increase conspicuity of workers.

Although the 2 in. red light did not receive the positive response (33%) as the 4 in. red/amber light, discussions showed that this LED light may be a good supplementary device to improve flaggers' safety. One veteran flagger demonstrated that by clipping the flashing light to a standard 24 in. STOP/SLOW paddle, they could create a possible low-cost alternative to the higher-priced LED STOP/SLOW paddle devices evaluated. While it increases the conspicuity of flaggers, this $20 flashing light, which is water resistant, can be used in many different working conditions. The flashing headlight in both questionnaire and discussion sessions received poor response with no subject indicating that they will use the device and 56% of participants stated that they will not use the light in a flagging operation. Table 4 shows the results described.

Table 4. Results of question seven: "How could the supplemental devices (4 in. red/amber light, 2 in. red light, and flashing headlight) improve flagger effectiveness/safety?"

Device	Would Use	Would Not Use	No Response/ No Opinion	Total
4 in. red/amber light	72%	6%	22%	100%
2 in. red light	33%	28%	39%	100%
Flashing headlight	0%	56%	44%	100%

Although portable, changeable message signs (CMS) were not presented at the focus group discussions, one focus group revealed that this CMS may reduce flagger conspicuity if used in the vicinity of the flagger even though it is useful in getting drivers' attention. One veteran flagger explained that they do not like any devices or vehicles around them that are distracting, except for traffic cones. While this portable CMS seemed durable, a few participants suggested that the device might not be able to withstand strong winds. One focus group participant suggested that the police vehicle is most effective in getting drivers to slow down in work zones. The flagger elaborated that the presence of police vehicle alone may be suffice to alert drivers to decrease their speeds on sections of highways where needed.

In the focus group with emergency responders (e.g., policemen and firefighters), participants revealed that firefighters are not expected to direct traffic at crash sites, even though they may arrive on the scene earlier than police officers. So, technology-enhanced devices generally do not apply to them. Nonetheless, they will assign one firefighter to block the traffic or use the flashing light wand at night to direct the traffic, if needed. While firefighters indicated they may still use certain technology-enhanced devices, police officers indicated otherwise. One police officer explained that police officers perceive that the red flashing lights (e.g., the 4 in. and 2 in. lights, as well as the flashing vests) may turn into drivers' targets, especially drivers that are operating while intoxicated. Additionally, the police in the focus group believed that any blinking lights on their uniform would detract from their desired serious demeanor, which they use to command respect of the drivers. Therefore, police officers tend to use flares, flashlights, and hand signals to get drivers' attention in emergency situations rather than flashing lights. While police officers revealed the disadvantages of red flashes; one firefighter pointed out that red is not as visible as amber.

Overall, participants understood that the flashing lights were incorporated to enhance the conspicuity of their presence in the work areas. However, electronic components, usability, and durability of these flagger devices appeared to be concerns that the subjects had when asked to relate the devices to their duties. Conventional equipment, such as the standard STOP/SLOW paddle and safety vest, seemed to have advantages over the technology-enhanced flagger devices (flashing STOP/SLOW paddles and blinking safety vests) presented. Nonetheless, participants in the first all-flagger focus groups revealed that they may purchase the flashing/blinking STOP/SLOW paddles and safety vests just for some night or evening work projects they occasionally worked. In addition, participants in all focus groups summarized (with the exception of the police) that supplementary devices such as the 4 in. red/amber light or 2 in. red light seemed to provide the flexibility, assurance, and budget they need to perform their duties.

FIELD SURVEYS

The final phase of this study was to conduct field surveys to examine the effectiveness of these technology-enhanced flagger devices on the basis of motorists' responses. Field survey locations were searched within the 100-mile radius from the city of Lawrence. With the assistance of the Kansas Department of Transportation (KDOT) and various county engineers' offices, the following were the three locations selected for field surveys:

- US-169 between Iola and Colony (Iola, Kansas)
- N 700th Road and E 1900th Road (Douglas County, Kansas)
- 31st Street between Louisiana and N 1275 Road (Lawrence, Kansas)

The setup of the flagging operation was also an important criterion that was considered when planning the survey instrument. Depending on the work to be completed by the workers, an ideal work zone for this research should be one mile long or longer, coordinated by two flaggers and a pilot car. However, field testing on 31st street revealed that the mobile work zone that moved intermittently could not provide the sufficient leeway needed to conduct the motorist surveys, and thus, this location was removed from further consideration.

Observations showed that flaggers on US-169 and 31st Street were equipped with similar safety gear, which included safety vests and hats. These workers explained that the clothing and equipment can help alert the drivers sooner and increase their conspicuity. The superintendent of the work zone on US-169 elaborated that while a queue of vehicles was instructed to stop, the traffic released from the opposite direction can better recognize the dress of the flagger stopping the opposing traffic and drive with caution. In order to prevent unnecessary procedures that may disturb the flagging operation, only the standard and flashing STOP/SLOW paddles were tested in these work zones.

Due to the nature of the construction work, the field surveys were conducted during clear daylight conditions. Contractors and authorities in KDOT revealed that nighttime flagging operation are infrequent and are only allowed when construction workers need to complete certain work at night; additionally, these night work locations are often police-controlled. Due to this issue, the selected technology-enhanced devices were only tested during daytime. By consolidating the surveys, a total of 99 motorists' responses were collected for subsequent analysis. The survey questions used for this research were consistently performed throughout the study period unless otherwise indicated in the report.

Prior to initiating the surveys, vehicle type was recorded to classify each automobile sampled. On the basis of the results, 73 vehicles were recorded as passenger cars and the remaining 26 vehicles were classified as heavy trucks. Information such as gender, age, and education level were not collected for this study. As aforementioned, four different STOP/SLOW paddles, including a standard paddle, were tested on both ends of the flagger-controlled work zone. Additionally, only the first and second vehicles in a queue on both ends were selected to participate in this motorist survey. This was to ensure that the selected respondents reacted based on the presence of flagger or the STOP sign, and not because of the vehicles that stopped or slowed down in front of them. Preliminary trials revealed that each questionnaire needed to be completed within the 7-8 minutes time frame for the first and second vehicle in a queue before the flagger released the traffic. Secondary vehicles in queues that stopped behind a large truck were not surveyed. This measure was to ensure that the views of the selected drivers second in order were not obstructed by the first vehicle.

The survey questionnaire was conducted in four parts. The first section was to estimate whether drivers were local, same county residents, or other state residents. In addition, the questions were designed to ensure that there were no repeat respondents and each respondent was independent of the others. Anyone who had taken the survey earlier and was returning back through the work zone was not surveyed a second time. The second part was to examine the effects of the STOP/SLOW paddles along with other existing traffic control devices. In this section, survey respondents were asked to name all of the things they saw from the time they entered the work zone until they stopped by the flagger. For the third section, drivers were asked to state their opinions regarding the (flashing or standard) STOP/SLOW paddles handled by the flaggers. In the final sections of the questionnaire, drivers were instructed to state their responses and reactions to the flashing STOP/SLOW paddles displayed. For the second, third, and final sections, respondents can reply multiple answers.

Figure 1 shows the results of the distribution for question one: “Have you driven this work zone before?”, question two: “Is this your first time today?”, and question three: “Did you see construction activities before this?”. The first question was designed to separate two major drivers: local and different state. The second question was created to further divide the drivers into specific groups. It can be observed from this chart that advance warning signs were the leading choice in each of the categories, except for the group of first-time drivers who have not driven the work zone before. While the advance warning signs received attention from the drivers, the results of the STOP sign and flaggers indicated otherwise, with less than 27% of respondents in these categories stating that they noticed them.

For question three: “What did you see?”, most of the drivers (70%) responded that they paid attention to the advance warning signs, whereas 36% of respondents replied that they saw the construction cones and new pavement. For the STOP sign and flagger, each of these categories received 15% and 12%, respectively. Overall, the conspicuity of STOP sign and flagger when combined together collected a total of 27% of responses. Additionally, 13 drivers stated that they did not see the STOP sign until about 100 feet away or just noticed it when instructed to stop. The lack of conspicuity effect is one evident finding that can be observed from the results; however, the safety of the flaggers is the bigger concern when these workers are out performing the flagging operation in work zones.

Four different STOP/SLOW paddles, including a standard paddle, were tested in three work zones. Overall, the results of the computations revealed that there were few differences between the standard and flashing STOP/SLOW paddles when tested in work zones. The results were considered reasonable and conformed to expectations.

Figure 2 shows the overall distribution of question five. Overall, most drivers (65%) stated that they liked the displayed STOP sign they saw, whereas 14% of drivers indicated that they did not like the paddle and

21% had no preference. When asked about their opinions regarding the displayed STOP sign, 59% of in favor drivers stated that it commanded their attention, whereas the next in order 14% of in favor drivers indicated that it fulfilled a need. While most respondents replied that they were in favor of the displayed STOP sign, 86% of those that were not in favor indicated that they either did not see it or thought it was hard to see. Overall, most drivers liked the six foot STOP signs that were displayed to them. The flaggers revealed that while these flashing STOP/SLOW paddles were new, striking, and clean, the bigger size (24 in. standard or flashing STOP/SLOW paddle) seemed to provide the respect and attention they needed in order to direct the traffic compared to the 18” STOP/SLOW paddle they normally used.

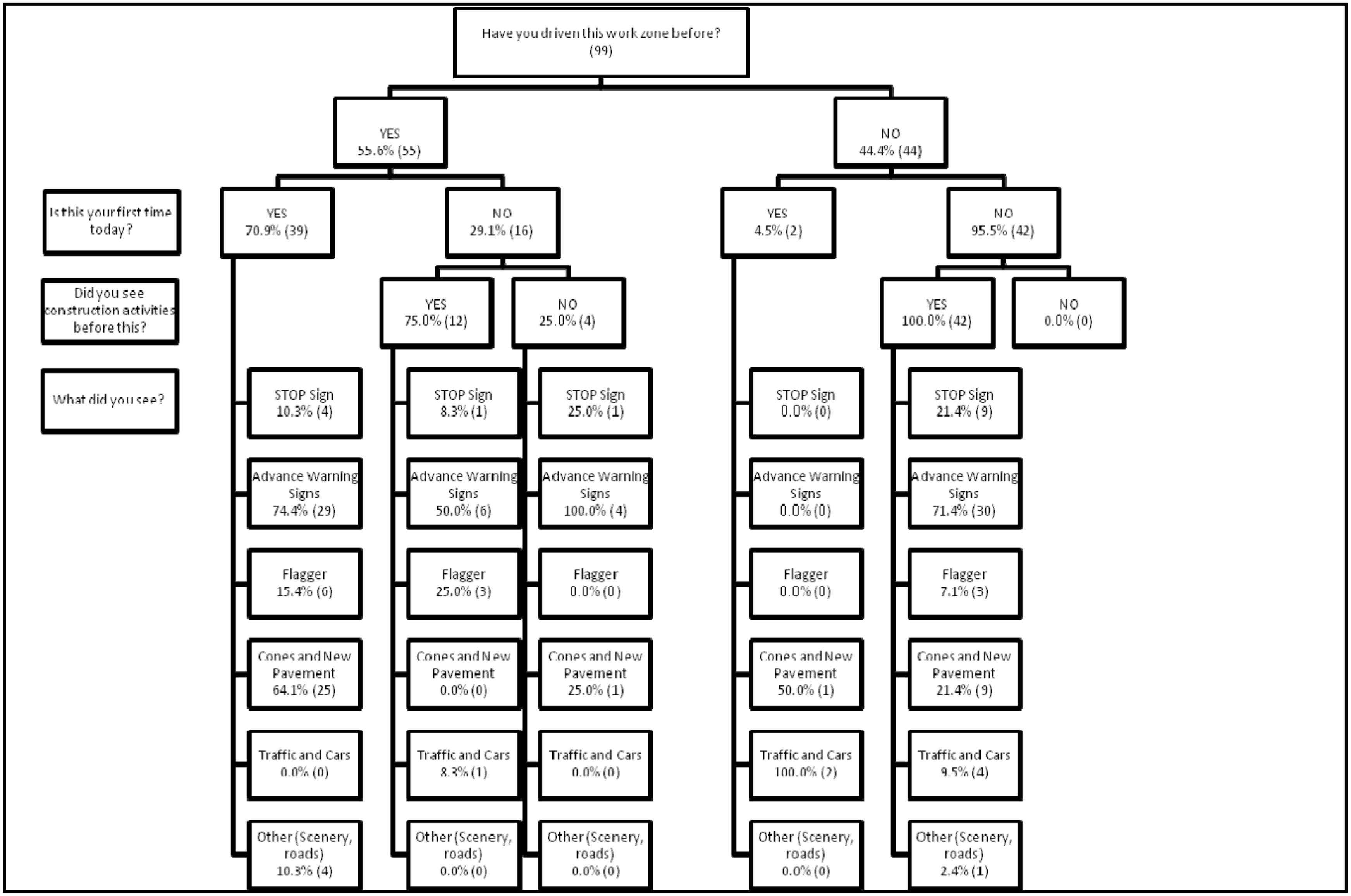
Have you driven this work zone before? (99)
YES 55.6% (55)
NO 44.4% (44)
Is this your first time today?
Did you see construction activities before this?
What did you see?
YES 70.9% (39)
NO 29.1% (16)
YES 4.5% (2)
NO 95.5% (42)
YES 75.0% (12)
NO 25.0% (4)
YES 100.0% (42)
NO 0.0% (0)
STOP Sign 10.3% (4)
Advance Warning Signs 74.4% (29)
Flagger 15.4% (6)
Cones and New Pavement 64.1% (25)
Traffic and Cars 0.0% (0)
Other (Scenery, roads) 10.3% (4)
STOP Sign 8.3% (1)
Advance Warning Signs 50.0% (6)
Flagger 25.0% (3)
Cones and New Pavement 0.0% (0)
Traffic and Cars 8.3% (1)
Other (Scenery, roads) 0.0% (0)
STOP Sign 25.0% (1)
Advance Warning Signs 100.0% (4)
Flagger 0.0% (0)
Cones and New Pavement 25.0% (1)
Traffic and Cars 0.0% (0)
Other (Scenery, roads) 0.0% (0)
STOP Sign 0.0% (0)
Advance Warning Signs 0.0% (0)
Flagger 0.0% (0)
Cones and New Pavement 50.0% (1)
Traffic and Cars 100.0% (2)
Other (Scenery, roads) 0.0% (0)
STOP Sign 21.4% (9)
Advance Warning Signs 71.4% (30)
Flagger 7.1% (3)
Cones and New Pavement 21.4% (9)
Traffic and Cars 9.5% (4)
Other (Scenery, roads) 2.4% (1)

Figure 1. Response to question one: "Have you driven this work zone before?" two: "Is this your first time today?" three: "Did you see construction activities before this?" and four: "What did you see?"

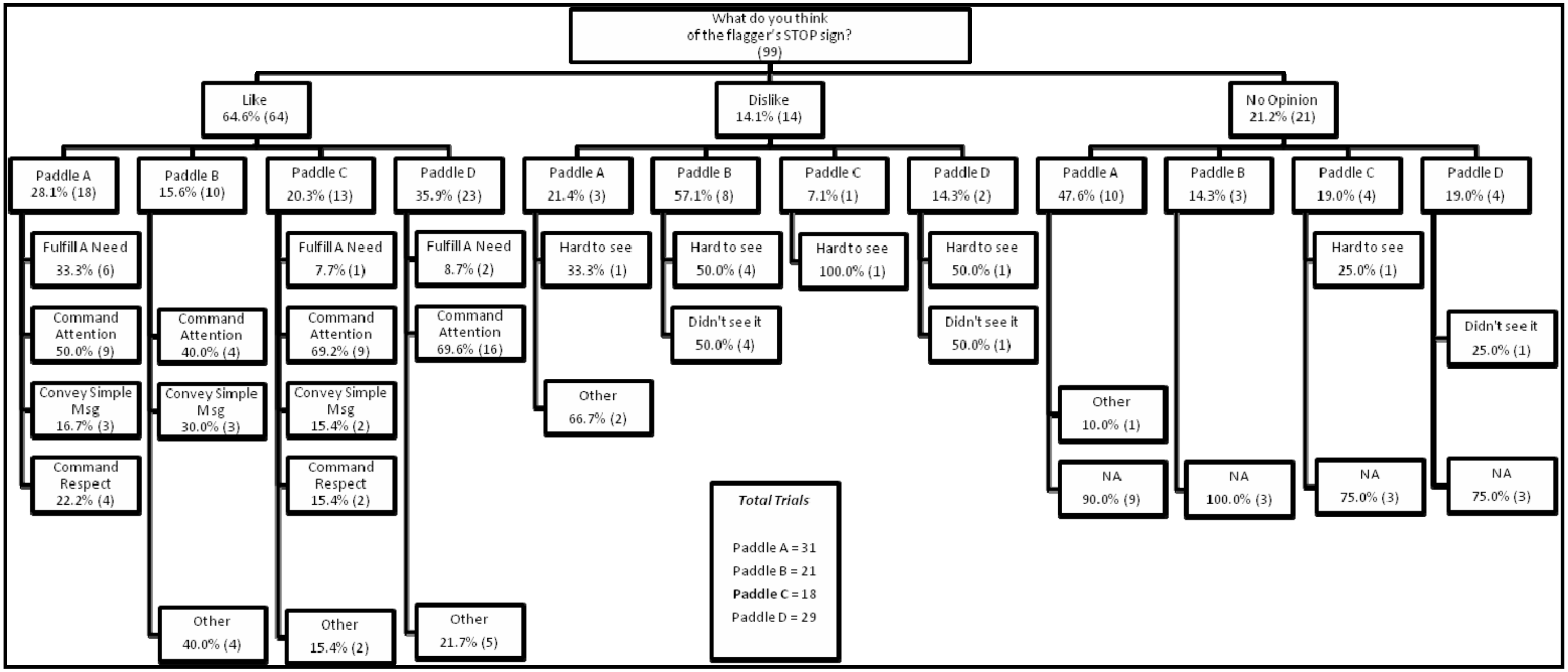
What do you think of the flagger's STOP sign? (99)
Like 64.6% (64)
Dislike 14.1% (14)
No Opinion 21.2% (21)
Paddle A 28.1% (18)
Paddle B 15.6% (10)
Paddle C 20.3% (13)
Paddle D 35.9% (23)
Paddle A 21.4% (3)
Paddle B 57.1% (8)
Paddle C 7.1% (1)
Paddle D 14.3% (2)
Paddle A 47.6% (10)
Paddle B 14.3% (3)
Paddle C 19.0% (4)
Paddle D 19.0% (4)
Fulfill A Need 33.3% (6)
Command Attention 50.0% (9)
Convey Simple Msg 16.7% (3)
Command Respect 22.2% (4)
Command Attention 40.0% (4)
Convey Simple Msg 30.0% (3)
Other 40.0% (4)
Fulfill A Need 7.7% (1)
Command Attention 69.2% (9)
Convey Simple Msg 15.4% (2)
Command Respect 15.4% (2)
Other 15.4% (2)
Fulfill A Need 8.7% (2)
Command Attention 69.6% (16)
Other 21.7% (5)
Hard to see 33.3% (1)
Other 66.7% (2)
Hard to see 50.0% (4)
Didn't see it 50.0% (4)
Hard to see 100.0% (1)
Hard to see 50.0% (1)
Didn't see it 50.0% (1)
Other 10.0% (1)
NA 90.0% (9)
NA 100.0% (3)
Hard to see 25.0% (1)
NA 75.0% (3)
Didn't see it 25.0% (1)
NA 75.0% (3)
Total Trials
Paddle A = 31
Paddle B = 21
Paddle C = 18
Paddle D = 29

Figure 2. Distribution of responses to question five: "What do you think of the flagger's STOP sign?" with respect to the STOP sign used.

Figure 3 shows the responses to question five: “What do you think of the flagger’s STOP sign?” with respect to the STOP signs used. From this chart, it can be observed that the overall responses for flashing STOP paddles were not significantly different than the STOP sign that flashes alternately above and below the STOP/SLOW words, which received the highest response. When asked about the indication of the flashing light(s), only 13% of the drivers who noticed the signs replied that the flash light(s) signified a more important situation. Interestingly, more than half (54%) of the surveyed drivers did not think that the flashing STOP/SLOW paddles indicated a more important situation than if the paddle did not flash.

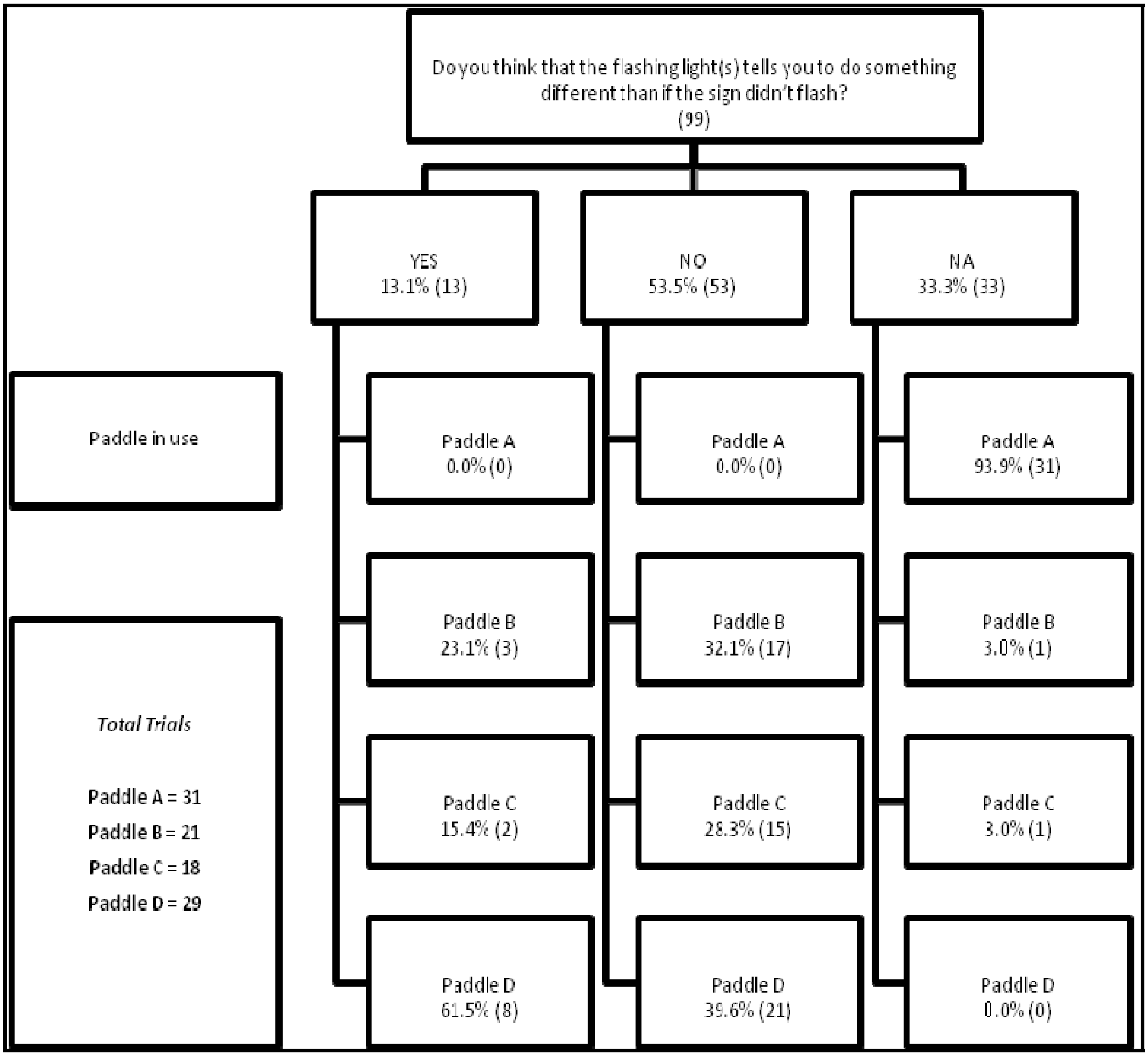
Do you think that the flashing light(s) tells you to do something different than if the sign didn't flash?
(99)
YES
13.1% (13)
NO
53.5% (53)
NA
33.3% (33)
Paddle in use
Total Trials
Paddle A = 31
Paddle B = 21
Paddle C = 18
Paddle D = 29
Paddle A
0.0% (0)
Paddle B
23.1% (3)
Paddle C
15.4% (2)
Paddle D
61.5% (8)
Paddle A
0.0% (0)
Paddle B
32.1% (17)
Paddle C
28.3% (15)
Paddle D
39.6% (21)
Paddle A
93.9% (31)
Paddle B
3.0% (1)
Paddle C
3.0% (1)
Paddle D
0.0% (0)

Figure 3. Distribution of overall responses to question five: “What do you think of the flagger’s STOP sign?”

In the last question, drivers were asked to state the precaution measures that they took when they observed the flashing STOP/SLOW paddles. Figure 4 shows the response to question six described. About one-quarter (26%) of the drivers indicated that they drove differently because of the flashing STOP signs, while 41% of the drivers replied that they did not adjust their driving. Surveyed drivers (26%) who responded to the presence of the flashing STOP/SLOW paddles indicated they either slowed down earlier, drove slower, or drove more cautiously with the flashing STOP sign that flashes alternately above and below the STOP/SLOW words, and the blinking STOP/SLOW paddle received the highest response.

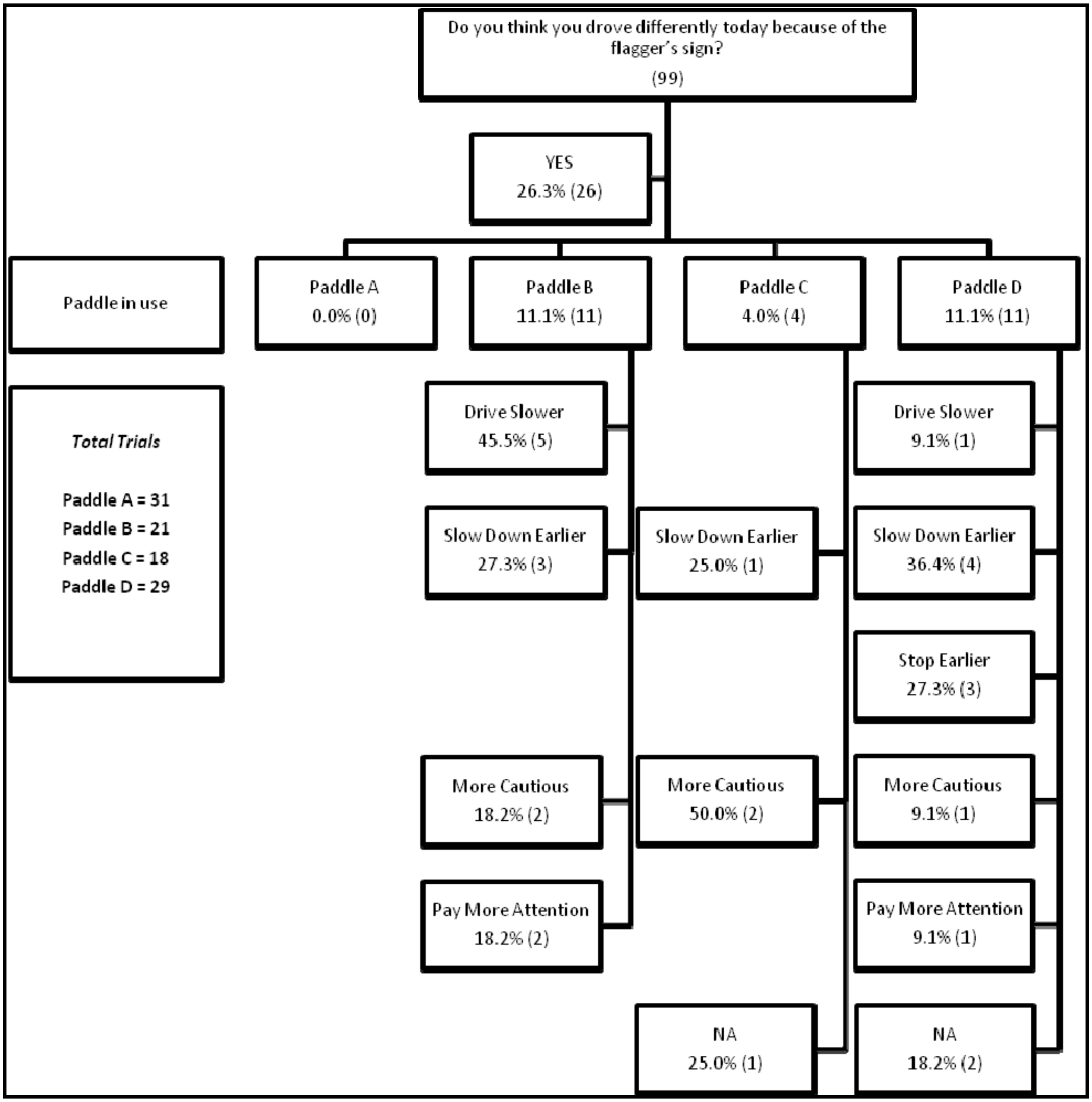
Do you think you drove differently today because of the flagger's sign?
(99)
YES
26.3% (26)
Paddle in use
Paddle A
0.0% (0)
Paddle B
11.1% (11)
Paddle C
4.0% (4)
Paddle D
11.1% (11)
Total Trials
Paddle A = 31
Paddle B = 21
Paddle C = 18
Paddle D = 29
Drive Slower
45.5% (5)
Slow Down Earlier
27.3% (3)
More Cautious
18.2% (2)
Pay More Attention
18.2% (2)
Slow Down Earlier
25.0% (1)
More Cautious
50.0% (2)
NA
25.0% (1)
Drive Slower
9.1% (1)
Slow Down Earlier
36.4% (4)
Stop Earlier
27.3% (3)
More Cautious
9.1% (1)
Pay More Attention
9.1% (1)
NA
18.2% (2)

Figure 4. Do you think you drove differently because of the flagger's sign?

FINDINGS AND DISCUSSION OF FUTURE RESEARCH

The findings of the analyses were organized by (1) focus groups evaluations and (2) field surveys. The following are the key findings from this synthesis effort:

Focus Group Evaluations

- The standard 24 in. STOP/SLOW paddle and the standard fluorescent yellow safety vest with reflective and orange striping emerged to be favorites among panel evaluation participants over other technology-enhanced equipment displayed.
- The results of the focus groups revealed that weight of devices, conspicuity of flaggers, and awareness of drivers were among the influential criteria for field personnel to opt for a flashing STOP/SLOW paddle over a standard paddle.
- Seventy-two percent of participants agreed that the 4 in. red/amber light appeared to have the best potential for large visibility gains, versatility of applications, and ease of use.
- The 2 in. red light did not receive 33% positive responses. Discussions showed that this LED light may be a good supplementary device to improve flaggers' safety.
- Focus group participants generally understood that the flashing lights were incorporated to enhance their conspicuity in work areas; however, electronic components, usability, and durability of these technology-enhanced devices appeared to be concerns that the participants had when asked to relate the devices to their duties.

Field Surveys

- Twenty-eight percent of drivers indicated that they saw the STOP sign or flagger in work zones when enquired about the things that they observed.
- When asked about their opinions regarding the displayed STOP sign, 74% of in favor drivers stated that it commanded their attention or fulfilled a need, whereas 86% of those not in favor indicated that they either did not see it or thought it was hard to see.
- More than half (54%) of the surveyed drivers did not think that the flashing STOP/SLOW paddles indicated a more important situation than if the paddle did not flash.
- Twenty-six percent of drivers stated that they drove differently (e.g., more cautiously) because of the flashing STOP/SLOW paddles, while 41% of the drivers replied that they did not adjust their driving. So the message was interpreted as the same.

In conclusion, the use of focus groups of flaggers and emergency services personnel, combined with field surveys of drivers has revealed that technology-enhanced STOP/SLOW paddles have potential for increasing the conspicuity of flaggers, even under bright daylight conditions. Additionally, flaggers and emergency services personnel liked the potential of the 4 in. lights as a way to warn drivers that they are approaching workers in or near the roadway.

ACKNOWLEDGEMENTS

The authors would like to thank the Smart Work Zone Deployment Initiative (SWZDI) for their support of this project. We also thank the following individuals for their help in recruiting subjects for this research: Mr. Steve Glass of LRM Industries, Inc., Capt. James Saladin of Lawrence Fire Department, and Mr. Keith Browning and Mr. Mike Perkins of the Douglas County Public Works. Also, the authors thank the data collection efforts of graduate research assistant Mr. Robert Rescot and undergraduate research assistant Samuel Klein.

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Assessing Transportation Impediments to Medicaid Members in the State of Iowa

Nikhil Sikka
Department of Civil and Environmental Engineering and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
nikhil-sikka@uiowa.edu

Paul Hanley
Department of Civil and Environmental Engineering and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
paul-hanley@uiowa.edu

ABSTRACT

Medicaid, an entitlement health insurance program funded jointly by states and the federal government, is one of the largest funding resources for transportation to the elderly, the financially constrained (low income), the disabled, and children across the country. To keep these subgroups of the population mobile and efficiently meet their accessibility needs is a challenge for Medicaid. It is a challenge that is increasingly difficult because of the profound demographic change in the United States—age is increasing and incomes are decreasing. These trends have been forecasted to continue for much of the first half of the twenty-first century.

The ensuing question is whether the available transportation services are adequate for Medicaid members in the state of Iowa. Iowa has approximately 250,000 individuals who receive coverage through the Medicaid program. Therefore, the goal of this study is to identify the current gaps in transportation services available to Medicaid consumers based on the information collected from Medicaid consumers, Medicaid workers, and transportation providers. To capture this, several surveys (including mail-out/mail-back surveys, Internet-based surveys, and telephone interviews) have been completed that capture transportation demand and supply within the state of Iowa.

Data analysis is done at various levels. Medicaid members are divided into three subpopulations: adults with disabilities, the elderly (65 years and older), and remaining adult members. In addition, the subpopulations are further identified by geographical area (urban area with fixed-route transit, urban area without fixed-route transit, and rural area without fixed-route transit). The initial descriptive analysis shows that transportation services are not a major problem for most Medicaid members. Nearly 89% of all members reported they have not missed a desired activity or medical service due to lack of transportation. As a whole, those who rely on fixed-route transportation report that it is almost always available, and those who do not drive themselves arrange rides with family and friends. However, the results are not uniform across the study groups. People with disabilities miss more medical services and desired activities than other Medicaid members. Strikingly, people who live within urban areas with access to fixed-route transportation missed the most medical visits and other activities.

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Logistic regression models are used to assess various transportation and socio-economic factors related to utilization of Medicaid transportation services. The transportation factors include car ownership or its immediate availability, availability of transit facilities, travel time and cost, weather conditions, understanding of Medicaid transportation resources, and reimbursement eligibility. The socio-economic factors include sex, age, and employment conditions. This multivariate analysis helps in understanding how various transportation barriers as a whole contribute to the problem of missing transportation trips, not only in various geographical areas, but also among the above-stated subgroups. Results of this analysis identify the groups with the greatest transportation needs.

Key words: fixed-route transportation—logistic regression models

Explaining the Recent Shift in Travel Behavior and Vehicle Miles Traveled: A Vector Error Correction Approach

Nikhil Sikka
Department of Civil and Environmental Engineering and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
nikhil-sikka@uiowa.edu

Paul Hanley
Department of Civil and Environmental Engineering and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
paul-hanley@uiowa.edu

Jielin Sun
Department of Urban and Regional Planning and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
jielin-sun@uiowa.edu

ABSTRACT

Given the recent economic downturn and increase in energy costs, forecasting travel behavior (and vehicle miles traveled [VMT]) has increased in complexity over time. In this paper, we estimate the long-run elasticities of VMT. We advance the analytical framework typically used to model elasticities (ARIMA and ARMA) with a vector error correction (VEC) regression model, which is based on system-based co-integration techniques. A benefit of co-integration analysis is that the dynamic co-movement among variables and the adjustment process toward long-term equilibrium can be studied. Applying the VEC model, we find that long-run gasoline price elasticity to VMT ranges from -0.31 to -0.88, and income elasticity ranges from 0.18 to 0.49 for the period 2000–2008. Comparing these results to previous studies, they show consumers have become more sensitive to gas price from earlier decades.

The recent changes in elasticities can be attributed to price spikes, but we posit that economic uncertainty has also played a significant role in altering people’s travel behavior. Therefore, we expanded the analysis by including the Consumer Sentiment Index (CSI) and disposable income. Results show that the CSI is a significant confounding variable that should not be excluded when explaining the cyclical shifts in travel behavior.

Key words: co-integration—elasticities—vector error correction—vehicle miles traveled

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Iowa's Roundabout Experience

Tim Simodynes
Office of Traffic and Safety
Iowa Department of Transportation
800 Lincoln Way
Ames, IA, 50010
Tim.Simodynes@dot.iowa.gov

ABSTRACT

As of the beginning of 2009, Iowa had approximately 32 modern roundabouts on public roads, including three on primary highways. The purpose of this report will be to analyze and present the safety performance and operations of the roundabouts in Iowa.

Key words: crash reduction—roundabouts—traffic operations—traffic safety

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Preliminary Results from Evaluation of Dynamic Speed Feedback Signs on High-Speed Curves

Shauna Hallmark
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
shallmar@iastate.edu

Neal Hawkins
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
hawkins@iastate.edu

Omar Smadi
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
smadi@iastate.edu

ABSTRACT

Crash rates on horizontal curves are often higher than those on tangent sections. Frequency and severity are related to factors including radius, degree of curve, length of curve, type of curve transition, lane and shoulder widths, preceding tangent length, and required speed reduction (Luediger et al. 1988; Miaou and Lum 1993; Mohamedshah et al. 1993; Vogt and Bared 1998; Shankar et al. 1998). “A Guide for Reducing Collisions on Horizontal Curves (2004)” reports that the crash rate for horizontal curves is around three times that of tangent sections. It also indicates that about 76% of curve-related fatal crashes involve single-vehicle run-off-road crashes, and 11% are head-on with an oncoming vehicle. In Iowa (2001–2005), 12% of all fatal crashes and 15% of all major injury crashes occur on curves; 14% of all urban fatal crashes and 11% of all urban major injury crashes occur on curves; 11% of all rural fatal crashes and 19% of all rural major injury crashes occur on curves.

Curve-related crashes result from a number of causes, including driver workload, driver expectancy, and speeding. Approximately 56% of run-off-road fatal crashes on curves are speed related. Studies have suggested that geometric improvements can reduce crashes. Zegeer et al. (1992) suggested that curve flattening could reduce crashes by as much as 80%, while widening lanes and paving shoulders on horizontal curves could reduce crashes by 21% and 33%, respectively. Costs for geometric improvements, however, are prohibitive, especially for counties with a large number of rural two-lane roads to maintain. Geometric improvements also require programming and can take some time to implement.

Reducing speed on curves can be done in the short term and at significantly lower costs than making geometric improvements. Dynamic curve warning systems (DCWS) are one method that has been tried in limited applications to reduce vehicle speeds and, subsequently, crashes. A DCWS consists of a speed measuring device, which may include loop detectors or radar, and a variable message sign, which provides warnings to speeding drivers to slow down.

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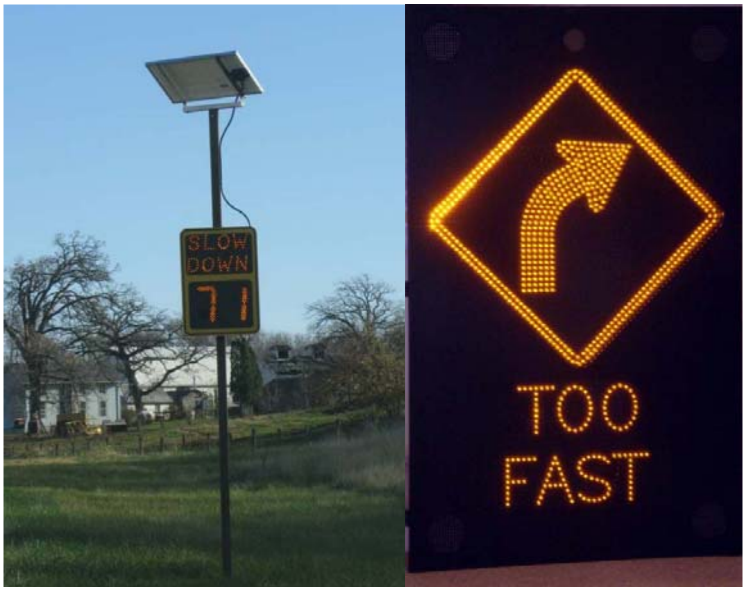
This presentation will discuss results of a project funded by the Federal Highway Administration (FHWA), the Iowa Highway Research Board, and the Iowa Department of Transportation. This project is conducting a national field evaluation of low-cost dynamic speed signs on rural roadways. The objective was to provide traffic safety and county engineers and other professionals with additional tools to more effectively manage speeds and decrease crashes on rural horizontal curves. The project installed two different types of signs on 22 curves in Washington, Oregon, Iowa, Florida, Pennsylvania, Arizona, and Texas. One sign displays the drivers speed when they exceed the 50th percentile speed, as shown in Figure 1a. The other displays the appropriate curve warning sign when a driver exceeds the 50th percentile speed, as shown in Figure 1b.



(a) **(b)**

Figure 1. Dynamic speed feedback signs

Speed studies were conducted before the signs were installed and will be collected 1 month, 12 months, and 24 months after installation. A before and after crash analysis will be conducted as well.

The first signs were installed in July 2008. This presentation will provide initial speed results.

Key words: curves—dynamic speed feedback signs—run-off-road crashes

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Plug-In Hybrid Electric Vehicles: Assessing Readiness for the Electrification of Personal Vehicle Transportation

Anthony Smick
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1415 Engineering Drive
Madison, WI 53706
smick@wisc.edu

Jessica Guo
Department of Civil and Environmental Engineering
University of Wisconsin, Madison
1415 Engineering Drive
Madison, WI 53706
jyguo@wisc.edu

Megan Mallette
Department of Electrical and Computer Engineering
University of Wisconsin, Madison
1415 Engineering Drive
Madison, WI 53706
megan.mallette@gmail.com

Giri Venkataramanan
Department of Electrical and Computer Engineering
University of Wisconsin, Madison
1415 Engineering Drive
Madison, WI 53706
giri@engr.wisc.edu

Chris Getter
Department of Electrical and Computer Engineering
University of Wisconsin, Madison
1415 Engineering Drive
Madison, WI 53706
getter.chris@gmail.com

ABSTRACT

This paper presents an analysis of plug-in hybrid electric vehicle (PHEV) readiness at the community level using parcel level, Tax Assessment data for the City of Madison, WI. Based on results of the readiness analysis, a scenario analysis of electrical grid impact due to varying levels of PHEV adoption is also described.

Key words: plug-in hybrid electric vehicles—infrastructure readiness—electricity accessibility—grid impact

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INTRODUCTION

Plug-in hybrid electric vehicles (PHEVs) extend the vehicle technology of hybrid cars to replace a portion of petroleum-powered personal vehicle travel with electric power. What are currently known as hybrid cars or conventional hybrid vehicles are technically named hybrid electric vehicles (HEVs). As the word “plug-in” and the letter “P” is included in its names, plug-in hybrid electric vehicles are designed for external electrical recharge, while HEVs are not. PHEVs render the energy of electricity, which while ubiquitous, an alternative fuel for personal transportation.

Both HEV and PHEV technologies focus on *hybridizing* the energies of petroleum fuel and electricity to deliver power to vehicle drive trains. The technological extension to PHEV begins with the propulsion and drive power designs of HEV. HEV designs differ among manufacturers and models, but they are alike in that they use petroleum fuel and internal combustion engines to generate electricity, which in turn sustains the power of electric drive systems. Note then, that electric vehicles (EVs), also known as battery electric vehicles (BEVs), have no such hybridization with petroleum energy and would always rely on electricity and, subsequently, recharge connections for it. When HEVs are operated at higher loads, the internal combustion engine power systems supplement the electric drive power. When HEVs are not commanded to be propelled, i.e., during coasting and braking, kinetic energy regenerates electricity for battery storage. While the charge of the HEV battery packs may fluctuate slightly, the HEV power control systems ensure that the internal combustion engines sustain relatively constant battery charges. The current HEV battery packs require stable charges because cyclic and deep depletions and recharges jeopardize their performances and life cycles.

Designs of PHEV propulsion and drive trains also differ among each other. PHEV hybridization of petroleum and electricity functions similar to HEVs. However, the ability to plug these vehicles into external sources of electricity indicates that the battery packs of PHEVs are rechargeable and more able to be depleted with greater fluctuations than the HEV battery packs.

PHEVs are typically classified among themselves according to the distance in miles a fully charged battery can propel the vehicle under standard conditions while operating completely with electricity, or in 100% *charge depleting* mode. For instance, when fully charged, a PHEV-40 will travel about 40 miles on level terrain with mild acceleration without needing petroleum fuel and the internal combustion engine. When in *charge sustaining* mode, PHEVs operate much like HEVs—nearly exclusively with the internal combustion engine sustaining the non-depleting battery portion and supplemented only by regenerated electricity. A blend of the two energies propels a PHEV that has a fully charged battery greater distances or with harsher operation by depleting the battery more slowly while the internal combustion engine is selected to co-operate with the battery pack. Particularly, a PHEV-0 operates like this all of the time. It is not an HEV since its battery pack can be recharged, but it also will not travel exclusively electrically with its fully charged battery pack. Once the rechargeable portion of a PHEV battery back is depleted, the internal combustion engine alone energizes the drive system much like an HEV.

While PHEVs are recognized as a promising technology for reducing some of the economic, environmental, geo-political, and energy disadvantages associated with the conventional internal combustion engine (ICE) automobile, the adoption of PHEVs into our communities is not without challenges. The net societal benefit of PHEVs over the ICE vehicles also depends on an array of factors such as recharging infrastructure, energy production life cycle, and vehicle and energy costs.

One particular challenge associated with PHEV adoption is the infrastructure that distributes electrical energy and the necessary connections to that infrastructure for charging PHEV battery packs as more PHEVs penetrate that national personal vehicle fleet. Currently, only a fractional percentage of the current

U.S. personal automobile fleet consists of PHEVs in forms of manufacturer prototypes, eager early-adopter personal vehicles, and retrofitted conventional hybrid vehicles. The future market penetration of PHEVs in the United States is predicted to range from 100,000 vehicles by 2011 (1) to 25% of the light-duty vehicle fleet by 2050 (2). Policy makers' desire to encourage PHEV adoption and the present uncertainty of PHEVs' eventual level of market penetration raise the question of whether, and to what extent, our current infrastructure is ready to support communities' adoption of PHEV.

This study sets out to evaluate communities' PHEV readiness, defined here as the ability of the current electricity infrastructure to meet the demand of additional new "fuel" utilized by PHEV. In particular, we consider a household as being "PHEV ready" if it has the ability of at-home charging. PHEV readiness is also considered as the base criterion for a household to qualify as part of the market pool for early PHEV adoption.

PAST STUDIES OF PHEV READINESS

The use of home charging/refueling ability to define the upper bound of early PHEV market is not new and is drawn from past observations of ICE vehicle adoption. At the advent of the ICE vehicle introduction, gas stations as we know them were not prevalent. Motorists purchased vehicle fuel in bulk for storage at home. Doing so among these early ICE adopters sustained ICE vehicle growth, until stations became profitably operable. The same may be for PHEV. After a period when motorists who elect to own PHEVs do so with the exclusive home charging, a PHEV fleet may grow to demand secure, public recharging or battery swap stations that resemble, replace, or replicate the current gas station infrastructure. Such stations, thereby, may preclude the need or desire for additional home recharge stations and may induce a market to those without home structural capacity for recharge stations.

The correlation between PHEV adoption and home charging is evaluated to some degree by (3) noting that 120 VAC outlet accessibility defines "prospective owners" as what we would call a pool market for potential PHEV adoption (4). This indicates that while garages can define the pool market, they too may need modification before motorists bring PHEVs home intending to recharge at home. In fact (5), in writing about home 120 volt and 240 volt charging only briefly mentions the use of a standard wall outlet for charging a PHEV. It is conjectured in the study that existing circuits currently in garages could be used, but a switch may be required that deactivates the wall socket in favor of PHEV charging socket and vice versa. In such cases, homeowners, electrical codes, and installers will need to assess continuous power to other garage appliances.

Two studies conducted surveys, with which access to home charging locations was quantified. In (6) 2,372 U.S. nationwide household respondents, 52.4% "identified a electrical outlet within 25 feet of their vehicle parking spot at their home location at some time during their 24-hour day." The precision of that question does account for vehicle owners who may not park in single-home garages yet still have access to an outlet. In (7), 400 consumers in Boston, Atlanta, Phoenix, and Los Angeles were sampled for other PHEV-related items, such as "relatively easy access to a plug, with 120 volt systems being relatively hassle free." Unfortunately, the phrasing of the question is not evident and with 86% of households reporting such access, which seems intuitively excessive, by no means matches the rate of (6). Nonetheless, the precision of not necessarily relying on single-home garage information is noted.

For a larger sample size, (4) uses the 1% Public Use Microdata Sample (PUMS) of the 2000 Census of the State of California. These data were used for the purpose of analyzing not just capabilities for only electric home refueling but for future hydrogen refueling as well. With it, housing stock built after 1974 was used as a proxy for "home connection hardware." Buildings built since 1974 have been wired according to the National Electric Code revision of that year, and those authors intimate that code as sufficient for the purpose of their study. This limits the precision of the PHEV pool market by underestimating older homes and, specifically, their garages that could easily have been wired for PHEV charging and overestimating the number of buildings, which could include apartments and condominiums. The result of that assessment was a pool market of 15% of households. Similarly, the publicly available American Housing Survey is used by (8) to identify the existence of a “garage or carport” amenity at a single-unit house is a proxy for “likely PHEV buyers.” The pool market assessment in this case yields 38.7% of households. Extending electricity to carports may be more code cost prohibitive than even doing so with detached garages, since electrical raceways exposed to the elements do require additional code adherence. So, assuming that builders included costs to serve carports with electricity may have overestimated the pool market in this case.

The four aforementioned studies have delivered significantly different assessments of pool markets. As described above, pool markets among households are either 52.4%, 86%, 15%, or 38.7%. Intuitively, the values, 86% and 15% seem too large and too small, respectively. The values, 52.4% and 38.7% both seem reasonable, yet where 38.7% of single homes have garages or carports might overestimate the pool market on behalf of carports; that value is still less than the survey respondents’ 52.4% pool market. Perhaps opening the pool market of that survey to include any outlets within 25 feet subsequently overestimated it.

A report on compressed natural gas (CNG) infrastructure of the 1980s shares lessons about alternative fuel transportation (9). That report does call exception to the electricity utility infrastructure, “which is already available in homes, (…and…) can avoid this issue (inadequate infrastructure) through affordable repowering.” The report also stated that “the main barrier was a lack of infrastructure to support the converted vehicles” and “utilities or energy suppliers can be allies, but need to ensure that their actions are strategic in building the market.” These comments are the basis for this paper.

A SPATIAL ASSESSMENT OF PHEV READINESS

In order to accurately assess current and future infrastructure needs, it is imperative that the electrical utility organizations and agencies know both how much PHEV load will be added to their infrastructures and the locations where that PHEV load will be added. For that, local utilities will need quantitative and geospatial insight into assessments of the PHEV pool market. The work described is intended as a tool for this purpose. Specifically, the authors present a method of geospatially determining PHEV electricity demand for distribution planners. This may be one of only a few, if any, methods that will assist the assessment of PHEV Readiness at the community level.

The ability for motorists to refuel their personal vehicles at home with connection to an electric outlet is one of the touted advantages of PHEVs. Focusing on the present for a first assessment of PHEV infrastructure support directs consideration of detached housing units that are known to have the secure and convenient personal-vehicle parking location of a garage. A survey conducted by the research team confirms that of the 267 sampled households residing in single detached houses with attached garages in the City of Madison, WI, 98.5% have access to 120 volts in their garages. Of the 43 sampled households residing in single detached houses with detached garages, 100% have access to 120 volts in their garages. Acknowledged are potential early or present PHEV adopters who have no access to home charging sources yet know they can rely on electrical outlets at their employers’ parking locations or their other activities’ parking locations. Similarly acknowledged are potential PHEV adopters who live in condominiums or apartment complexes yet can still arrange vehicle charging connections. While such dwelling facilities may or may not, and moreover infrequently, have secure charging amenities, the authors are certain that most garages associated with detached homes do.

Based on the above reasoning, the authors' analysis of PHEV readiness in the City of Madison, WI, is based on the assumption that all single detached houses with garages in the City of Madison, WI, have electric service in their garages for recharging. Differing from previous studies that used the Census data to identify the aggregate distribution of such housing units across census reporting units, the authors opted for the City's Property Tax Assessment data, which provide housing information at the land parcel level. This allows the authors to pinpoint the exact locations of housing units that are PHEV ready or not.

Specifically, the City of Madison Tax Assessment data were geo-referenced using the Madison Area Transportation Planning Board's land parcel boundary data in ArcGIS format. As such, each land parcel in the city is characterized by its property class, property use, and garage type, among other attributes. Parcels with the following properties are then classified as PHEV ready:

- Property Class = "residential"
- Property Use Code = "single family"
- Garage Type = "detached" or "attached"

When these parcels are displayed on a map as shown in Figure 1, distinct clusters and sectors of these parcels are evident, thus displaying the PHEV Pool market.

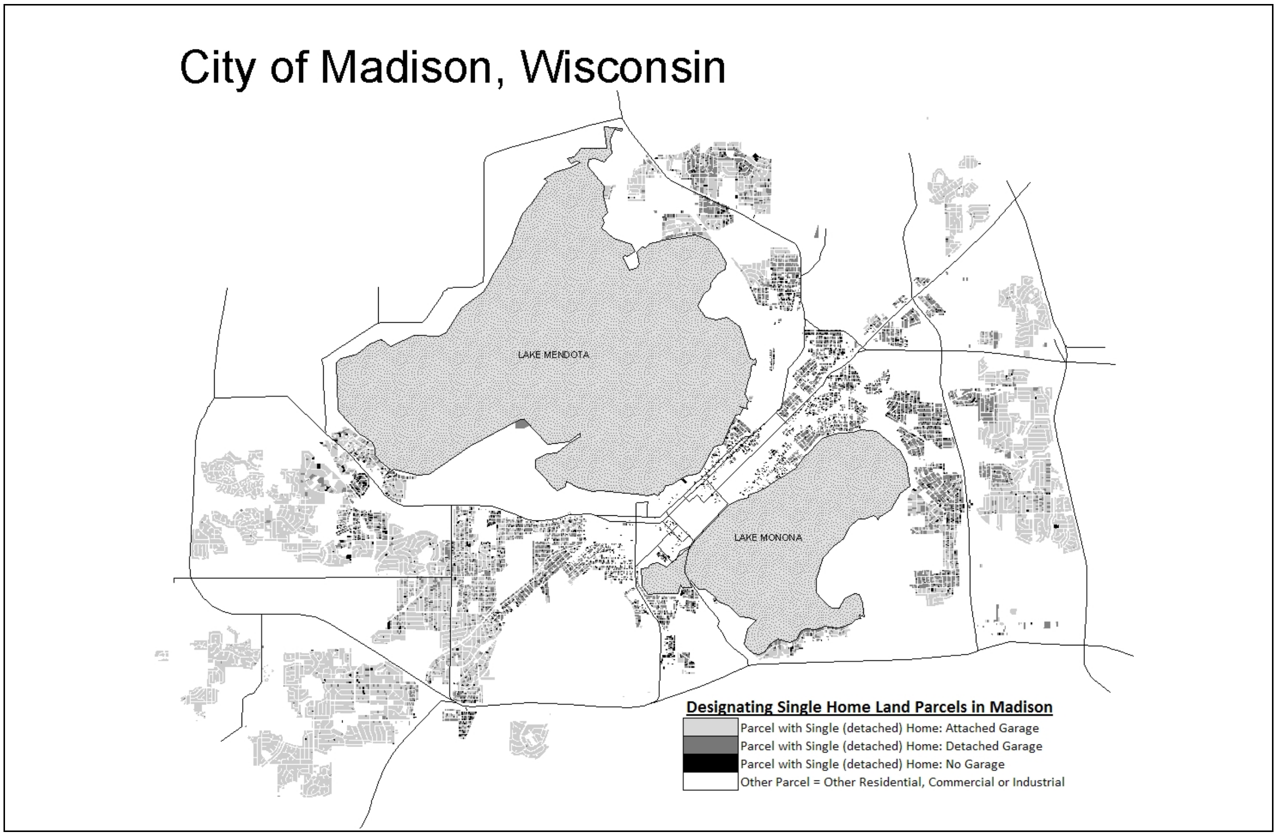
City of Madison, Wisconsin
LAKE MENDOTA
LAKE MONONA
Designating Single Home Land Parcels in Madison
Parcel with Single (detached) Home: Attached Garage
Parcel with Single (detached) Home: Detached Garage
Parcel with Single (detached) Home: No Garage
Other Parcel = Other Residential, Commercial or Industrial

Figure 1. Distribution of single-family residential units by garage type

AN ASSESSMENT OF GRID READINESS

Charging PHEVs in household garages substantiates a notable change in household electricity use, which in turn impacts the temporal and spatial load distribution seen by utilities. For instance, charging a fully depleted PHEV-40 from a standard 120 volt outlet would be on the order of operating a small hair dryer or one-third of a central air conditioning unit (different voltages aside), non-stop for two hours every other day (assuming full depletion of the PHEV-40 is a result of 40 miles all-electric driving; 20 miles/day x 2 days) all year. This could potentially be the second largest energy load in the cooling months and, in the course of a year, perhaps the single largest energy load, at the household level.

Fundamentally, all electrified homes, and subsequently their garages, are connected to electric power utilities and the generators that serve them through a network of transmission equipment and distribution lines. With the aforementioned substantial load of one household's PHEV in mind, the aggregate load of many households with PHEVs will merit study.

Following from the PHEV readiness analysis, the authors present a preliminary analysis performed to gauge the impact of adding significant PHEV load at the transformer level. According to a study performed by Duke Energy, the most significant impacts of PHEV market penetration will likely be due to geographic clustering of vehicles (1). Such demand clustering suggests the need for analyzing areas with high concentration of PHEV-ready households.

The grid impact analysis is based on data provided by Madison Gas & Electric (MG&E), the local utility serving the City of Madison, WI. MG&E provided a typical peak-day load curve for the daily consumption of eight customers fed from a single 50 kVA transformer. The data obtained are an approximation based on the load curve for a primarily residential feeder within MG&E's service territory. This approximation was used because MG&E does not record hourly load data at the transformer level.

As a case study, a small neighborhood of eight PHEV-ready households in Madison who share a selected transformer is the focus for the grid impact analysis. The scenarios of one, two, three, …, and up to all eight of these households becoming owners of PHEV are examined. These households are assumed to charge their vehicle from a standard 120 VAC 15 A outlet available in their garage. Although these outlets are rated to provide power up to 1.44 kW, preliminary studies on actual charging patterns from converted Hymotion Prius PHEVs show that the average power drawn by the vehicle when charging is approximately 0.77 kW (1).

Combining the transformer data and the vehicle data, the authors were able to determine the energy impact of adding discrete PHEV load to the selected transformer. Given that the transformer is rated at 50 kVA and assuming an average residential power factor of 0.8, it is possible to determine how adding PHEV load will impact the percent loading of the transformer. Utilities will likely attempt to minimize PHEV contribution to peak load by implementing certain rate structures. However, as a worst-case scenario, we can assume that customers will elect convenience over cost and charge their vehicles in the late afternoon. This unfortunately coincides with the typical residential peak load. Figure 2 illustrates the impact of adding PHEVs to the selected 50 kVA transformer, assuming each adopter owns a single PHEV.

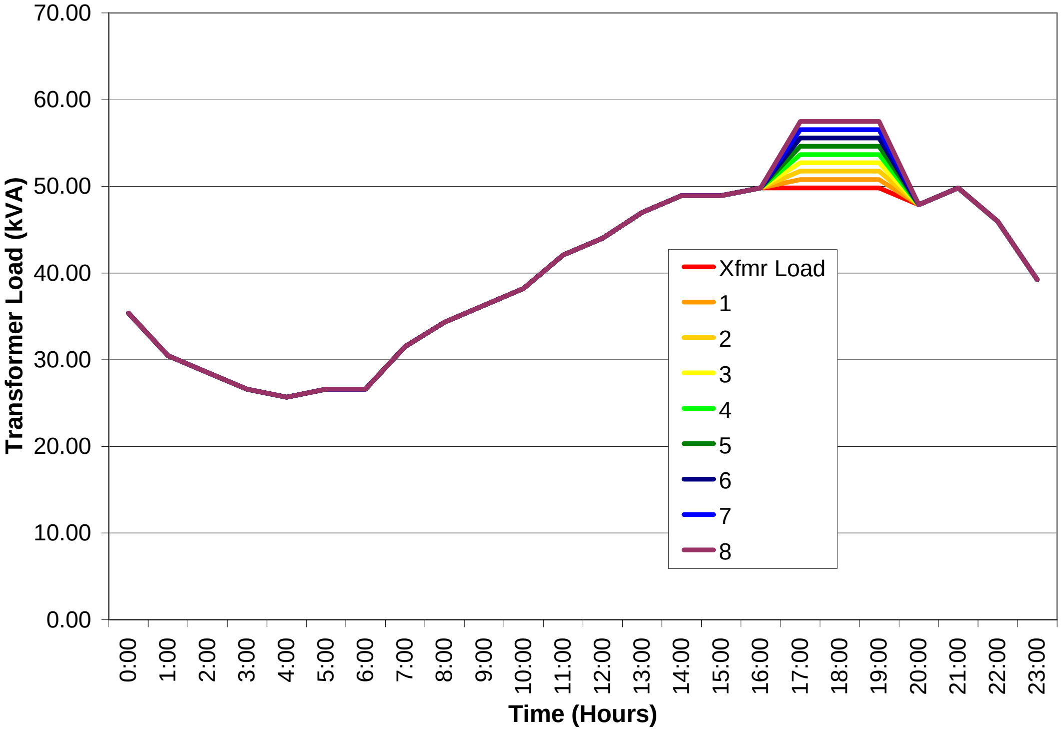

Figure 2. Transformer load profiles for the base and the eight levels of PHEV adoption scenarios

It should be noted that prior to adding any PHEV load to the system, this selected transformer is already approaching its rated value. Recall that this is not the typical load seen by the transformer,but rather the load seen on a particularly warm day within the last five years. However, adding four PHEVs to this transformer does increase the percent loading to over 107%, and adding eight PHEVs to this transformer increases the percent loading to nearly 115%. It will be important of utility planners to understand when, where, and how much the PHEV load will add to the system in order to adequately plan maintenance, upgrades, and additions to the distribution system.

DISCUSSION

The authors should consider that more than one PHEV could be associated with any parcel, where there may be circuitry in multi-car garages to simultaneously charge multiple PHEVs or where staggered charging times are used. In fact, the City of Madison Tax Assessor data do include variables for multiple garages on parcels and for their quantities of parking stalls. For the demonstration herein, the envelope of demand and pool market was assumed as only one PHEV per single home parcel with garage.

Focusing on just the present did eliminate the future considerations of battery transfer stations for drivers who may have elected to own a PHEV yet have no periodic access to electrical charging sources. Moreover is the potential for retail charging stations that resemble gas stations, where PHEVs can be charged in time frames of minutes instead of hours. Even though most home charging is likely to occur during sleeping hours, which fortunately corresponds to off-peak electricity demand, some charging may necessarily occur during daytime hours. So far in this work, only residential charging durations on the order of two hours have been discussed.

A range of other PHEV topics extends from the vehicle-operator level to the energy policy and national market demand level. At the vehicle-operator level, for instance, fuel-related performances perceived and recognized by PHEV operators and battery size with weight conditions and benefits are analyzed with respect to electric fuel balances, blends, and choices against, with, and over gasoline. At the policy national market level, researchers investigate attributes related to regional and national energy use and the same aspects of fuel types, and as well implore the engagement of electric utilities to utilize rate structures that encourage leveling overall electricity demand with off-peak versus peak charging.

Another analytical complication may arise when electric utilities consider another conjectured attribute of PHEV—that of "plugging out" with "Vehicle-to-Grid" electricity. In that scenario, PHEV motorists elect to charge their vehicles as lower, off-peak cost, then subsequently resell some of the vehicle energy back to the utility for a higher price during higher demand, whether still at home or at a secure municipal/utility station. This aspect of PHEV is not covered herein, but to do so in the future would require an analysis of commercial/industrial geographic areas where such opportunities are more likely to be installed. It is hoped that this method would assist with those expanded efforts.

Policy makers and transportation planners alike may involve themselves with the process of defining other PHEV pool and real markets, since PHEVs and charging them does present one particular equity issue. Notable is the alleged $10,000 premium. If this was simply a premium for a standard ICE car, there would be a set of the motorist population that would not afford it. However, there could be members of that lower income group who may elect to purchase PHEV on the merits of fuel economy. Such instances may require a geographic representation that defines sectors of population with less vehicle purchase power yet perhaps does not include the population with single detached homes. Furthermore, if those with less purchase power could purchase a PHEV, they still may be prohibited from doing so because they still cannot afford to purchase their own secure personal charging station, i.e., a detached dwelling with a garage—their own home. Omitting sectors where the primary charging locations of single detached homes exist may help reveal where municipal, commercial, or dwelling complexes should be encouraged to install charging stations.

Additionally, agencies that wish to adopt policies encouraging denser built environments with goals of fewer road miles may reveal a contradiction with PHEV home charging. If single-unit personal garages remain the universal primary charging locations, the inclusion of those garages will use more land. Furthermore, residents who choose to live in denser urban environments may be served well by PHEV if they must own personal vehicles but may not have adequate access to PHEV charging opportunities. It is anticipated that the availability of a geo-spatial reference would assist in these assessments.

ACKNOWLEDGMENTS

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Rural Safety Innovation Program: An Overview and Status Update

Theodore A. Smith
Noblis
600 Maryland Ave. SW, Suite 755
Washington, DC 20024
theodore.smith@noblis.org

Linda Dodge
ITS Joint Program Office, Research and Innovative Technology Administration
United States Department of Transportation
1200 New Jersey Avenue SE
Washington, DC 20590
linda.dodge@fhwa.dot.gov

ABSTRACT

Rural roads carry approximately 40 percent of the vehicle miles traveled in the United States, yet annually, they account for nearly 55 percent of the fatalities. In 2006, there were 23,339 vehicle fatalities in rural areas, compared with 18,359 that occurred in urban areas. This is alarming considering that only 23 percent of the U.S. population resides in rural areas. Further, according to the National Highway Safety Administration’s Fatality Analysis Reporting System, the fatality rate for rural crashes is more than twice the fatality rate in urban crashes.

To address this challenge, the U.S. Department of Transportation launched the Rural Safety Initiative in February 2008. The focus of the Rural Safety Initiative is to highlight available options to help reduce highway fatalities and injuries on the nation’s rural roads. The Rural Safety Innovation Program (RSIP) is one element of the Rural Safety Initiative. The goal of the RSIP is to improve rural road safety by assisting rural communities in addressing highway safety problems and by providing rural communities the opportunity to compete for project funding to address these problems. This paper provides an overview and status of the Intelligent Transportation System (ITS) projects funded through the RSIP.

Key words: rural—safety

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RURAL SAFETY CHALLENGE

Rural roads carry approximately 40 percent of the vehicle miles traveled in the United States, yet annually, they account for nearly 55 percent of the fatalities. In 2006, there were 23,339 vehicle fatalities in rural areas, compared with 18,359 that occurred in urban areas. This is even more alarming considering that only 23 percent of the U.S. population resides in rural areas.[i] Further, according to the National Highway Safety Administration's (NHTSA's) Fatality Analysis Reporting System (FARS), the fatality rate for rural crashes is more than twice the fatality rate in urban crashes. The fatality rate for rural areas in 2006 was 2.25 fatalities per 100 million vehicle miles traveled (VMT), compared with 0.93 fatalities per 100 million VMT in urban areas.[ii]

Rural areas in the United States face a number of unique highway safety challenges that contribute to the severity of and frequency of accidents, including

- Seat belt usage—In 2006, 57 percent of all the people who died on rural roads were not restrained, compared with 52 percent in urban areas. In addition, 68 percent of fatally injured pickup truck drivers were unrestrained; the restraint use rate among these drivers is the lowest of any vehicle type.
- Speed—In 2006, 12,190 drivers involved in fatal crashes were speeding; of these fatalities, 57 percent were drivers in rural areas.
- Impaired drivers—Of the passenger-vehicle occupant fatalities involving impaired-driving crashes (blood alcohol concentration 0.08+) in 2006, 58 percent were in rural areas. At most blood alcohol concentration levels, the percentage of rural drivers involved in fatal crashes exceeds the percentage of urban drivers involved at the same blood alcohol concentration.
- Post-crash—In 2006, 66 percent of rural drivers killed in crashes died at the scene, compared with 51 percent of urban drivers. Seventy-two percent of drivers who died en route to a hospital were in rural areas.
- Implementation challenges—Implementing countermeasures through an interdisciplinary approach that includes engineering, enforcement, education, and emergency medical services is more challenging in rural areas, where engineering teams vary widely in their ability to develop, implement, and operate safety strategies.
- Outdated roadway design and roadside hazards such as utility poles, sharp-edged pavement drop-offs, and trees close to the roadway also are major contributors to the severity of rural crashes.[iii]

RURAL SAFETY INNOVATION PROGRAM

To address the challenges of rural safety, the U.S. Department of Transportation (U.S. DOT) initiated the Rural Safety Initiative in February 2008. The focus of the Rural Safety Initiative is to highlight available options to help reduce highway fatalities and injuries on the nation's rural roads. This targeted national campaign is taking advantage of opportunities to raise awareness of the risks drivers face on America's rural roads and provide communities with tools and assistance to address these risks where the Department's resources can be leveraged quickly and effectively.[iv]

The Rural Safety Innovation Program (RSIP), published in the *Federal Register* on February 29, 2008, is one element of the Rural Safety Initiative. This one-time opportunity is using funds from the Intelligent Transportation Systems (ITS) Program and the Delta Region Transportation Development Program (DRTDP). The RSIP requested applications for funds from road owners for projects to improve safety on rural roads. Eighty applications were received for ITS Program funds and 16 applications were received for DRTDP funds. There were 96 initial applications received from state and local governments in 28 states by the April 14, 2008, due date. Twenty-four applications were selected to be developed into comprehensive proposals for ITS program funds. The comprehensive Phase II application, review, and selection process was completed, and the final project selections were announced on August 27, 2008.[v]

The U.S. DOT has awarded funding to 10 state and local transportation agencies through the ITS Program for 12 projects that aim to improve safety on local and rural roads. Funding recipients are

- Arizona Department of Transportation (ADOT)
- California Department of Transportation (Caltrans) (2 grants)
- Colorado Department of Transportation (CDOT) (2 grants)
- Illinois Department of Transportation (IDOT)
- Minnesota Department of Transportation (Mn/DOT)
- Iowa Department of Transportation (Iowa DOT)
- Kansas Department of Transportation (KDOT)
- South Carolina Department of Transportation (SCDOT)
- King County, Washington Department of Transportation (KCDOT)
- Wisconsin Department of Transportation (WisDOT)

The goal of the RSIP is to improve rural road safety by assisting rural communities in addressing highway safety problems and by providing rural communities the opportunity to compete for project funding to address these problems. The primary objectives of the RSIP are to

- Improve safety on local and rural roads with innovative approaches in which rural communities develop and design local solutions to their roadway safety problems
- Provide best practices and lessons learned on innovative safety technologies to assist local and rural road owners and operators in developing and implementing infrastructure-based safety countermeasures that complement behavioral safety efforts
- Promote national awareness and interest in addressing rural safety issues,
- Promote the use of ITS technologies to improve safety on rural roads
- Implement and test ITS technologies in the rural environment that have been successfully deployed and operated in an urban environment

RURAL SAFETY INNOVATION PROGRAM PROJECTS

Minnesota Department of Transportation

Approximately 27 percent of all the crash fatalities reported in Minnesota between 2001 and 2005 were on curves in rural areas. To address this challenge, the Mn/DOT in partnership with the University of Minnesota is developing a low-cost technology that may help drivers select an appropriate speed, thereby enhancing safety when approaching a horizontal curve. The system currently under development is the dynamic curve warning system (DCWS) that may help drivers select an appropriate speed when approaching a horizontal curve. The DCWS consists of a warning sign combined with a speed measuring device (e.g., radar) that activates a variable message sign (e.g., slow down) when vehicles are traveling above a set specified threshold. The goal of the proposed DCWS is to evaluate the speed and actual or possible predicted (based on speed changes) of DCWS installations at three rural roadway horizontal curve locations (with speed-related safety concerns). Through the RSIP, Mn/DOT will evaluate the potential effects on speed and crash impacts of three permanently installed DCWSs in three Minnesota counties. If this system proves feasible in enhancing safety, it will be implemented elsewhere in the state of Wisconsin. The development of DCWS will begin the summer of 2009.

Wisconsin Department of Transportation

Using RSIP funds, the WisDOT will implement, demonstrate, and validate a new Rural Intersection Collision Avoidance System (RICAS). This new intersection collision avoidance system will use emerging sensing, computation, and display technology to provide real-time warnings to drivers before the conditions that lead to a crash can develop. RICAS is being developed to specifically address crashes that result from gap selection errors.

The intersection of US 53 and State Trunk Highway (STH) 77 will serve as the RICAS test site. RICAS comprises three components: sensing, computation, and an infrastructure-based Driver Infrastructure Interface (DII), which is an active variable message sign.

Sensors are used on US 53 (mainline road) to determine the position, speed, and lane of travel for vehicles approaching the intersection crossroads. Automotive radar was selected for this application, as it is accurate, durable, reliable, available, relatively inexpensive, and works in all weather conditions. Loop detectors are installed in the median and minor road approaches to sense vehicle presence. If a vehicle has been detected, the system will activate the DII. The DII relays alerts and warnings to drivers as determined by the computational system. If no vehicle is sensed, the DII will remain inactive, thereby limiting unnecessary distraction to the driver.

The DII that will be used in the project has been tested in driving simulators and was on-road tested during the summer of 2008 under the Cooperative Intersection Collision Avoidance Systems—Stop Sign Assist (CICAS-SSA) program.[vi] Figure 1 illustrates the layout of the equipment that RICAS will use at the US 53/STH 77 test site. It is anticipated that RICAS will be operational by October 2009. RICAS will be implemented at other critical intersections along US 53 and other high-risk rural roads throughout Wisconsin if it proves successful at the US 53/STH 77 site.

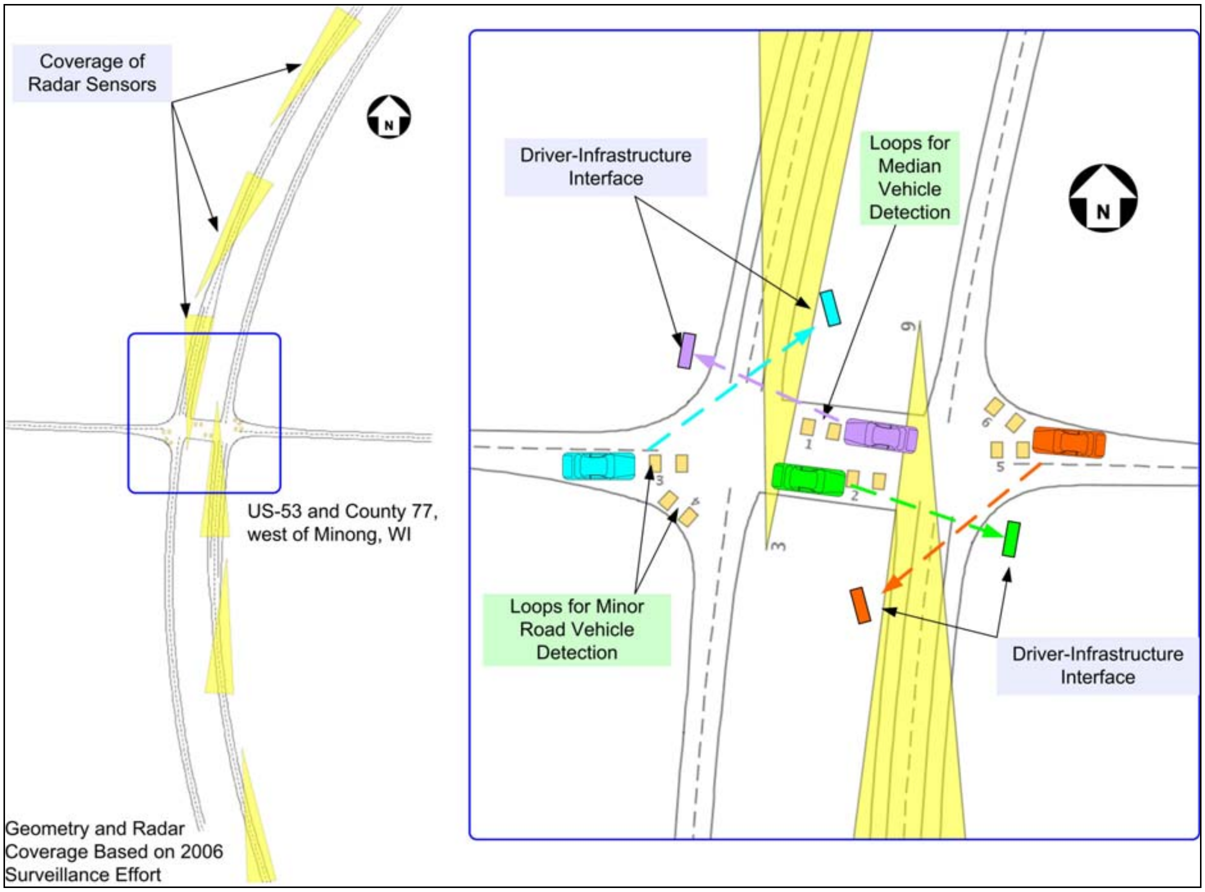
Coverage of Radar Sensors
US-53 and County 77, west of Minong, WI
Geometry and Radar Coverage Based on 2006 Surveillance Effort
Driver-Infrastructure Interface
Loops for Median Vehicle Detection
Loops for Minor Road Vehicle Detection
Driver-Infrastructure Interface
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Figure 1. Placement of equipment for RICAS test site in Minong, WI

Colorado Department of Transportation—Wolf Creek Pass

Using RSIP funding, CDOT is in the process of designing a system that consists of in-road, light-emitting diode (LED) lighting and dynamic speed messaging signs (DMS). The system is being developed to address collisions involving vehicles crossing over the centerline of the roadway in Wolf Creek Avalanche Shed. DMSs will be implemented in advance of both entrances to the Wolf Creek Pass Snow Shed that is located along a curve of US 160 in Mineral County, Colorado. Trucks transporting freight, ski travelers, tourists, and recreational vehicles heavily use this route. The in-road LED lighting system will illuminate/delineate the centerline of the roadway and reduce wall hits and crossover accidents in the snow shed.

The LED lighting will be augmented with speed messaging signs to warn drivers to reduce travel speeds to decrease the likelihood of over-driving the curve in the snow shed, resulting in lane departures. By combining speed warning signs and the in-road light delineation, CDOT anticipates that this system will lower vehicle accident rates and increase vehicle compliance for the posted speeds approaching the curves.

The uniqueness of this project is that CDOT will use the LED in-pavement lighting system to delineate a centerline, no-passing zone within the snow shed where lighting conditions are less than desirable and snow removal operations tend to obliterate conventional stripe delineation. LED in-road marking is also relatively new to the market, and no standards or design specifications exist. Consequently, CDOT is currently in the process of developing design specifications relevant for the project. The construction of the system is to begin in late 2009.

Colorado Department of Transportation—US 50

CDOT was also selected through the RSIP to develop and implement a truck tip-over warning system on US 50, a rural, low-volume roadway with low-speed curves. The system will warn all motorists of their speeds prior to the curves. Most of the accidents occurring on US 50 between mileposts 230 and 231 are fixed-object crashes (primarily involving guardrail) and overturning. These accidents are frequently due to drivers approaching the tight curves on the highway at unsafe speeds.

To address this problem, CDOT will develop an innovative stand-alone ITS application that is independent of a fiber backbone for management and operation. This application will include dynamic speed warning devices and speed-actuated variable message signs (VMS) that flash warning messages to drivers who travel too fast in advance of each horizontal curve. Due to lack of power at the location, two of the three signs must be operated with solar power. To do this, CDOT will use low-power LED blank-out signs, rather than the VMS boards used elsewhere in the state, with separate battery packs and solar arrays to power the blank-out signs and radar devices. CDOT anticipates this system will be operational in spring of 2010.

California Department of Transportation

A team consisting of (Caltrans) and Western Transportation Institute at Montana State University is using RSIP funding to research whether the deployment of an augmented Speed Enforcement (aSE) system on State Route 12 (SR 12) in San Joaquin County, California, will help to change driver behavior and reduce crash rates in work zones. The primary function of this system is to communicate relevant speed, violation, and hazard information to the stakeholders in this work zone context. Stakeholders are the driver, California Highway Patrol (CHP) officers, and the workers. The aSE includes the following functional components that are illustrated in Figure 2:

A. Portable radar stations (sensors) that track the speed of vehicles exceeding the advanced work zone speed limit sign.
B. Violators identified by their license plate will receive a speed warning on a changeable message sign (CMS) at the entrance to the work zone.
C. Once entering the work zone, a series of “smart cones” that are each fitted with a light display (beacon) and with non-radar sensors (e.g., sonar, light) track individual vehicle speed and synchronize the cone light display to “highlight” and follow any violating vehicle. These lights automatically cancel when the violation is corrected by reducing speed. This is intended to provide a visual warning to drivers that they are violating the speed limit.
D. A local pager network will be configured to automatically alert (vibration mode) only those workers in direct proximity to the detected hazard. This pager system will also incorporate a “panic mode” that any worker can trigger in the case of an injury to automatically contact the site supervisor, who can request public safety assistance to the work zone. This panic mode may also trigger a unique and conspicuous sequence of cone lights to alert all workers to the potential injury event.
E. Those vehicles that do not adhere or adjust to the posted speed limit for the work zone will be notified that they may be subject to a speed citation with an additional CMS at the exit of the work zone.
F. Relevant information about the violating vehicle (e.g., duration of violation, maximum speed, average speed, license plate, vehicle photograph, etc.) will be communicated and displayed to

downstream CHP officers, who can then use their judgment to locate the vehicle and cite the driver based on the information documented by the aSE

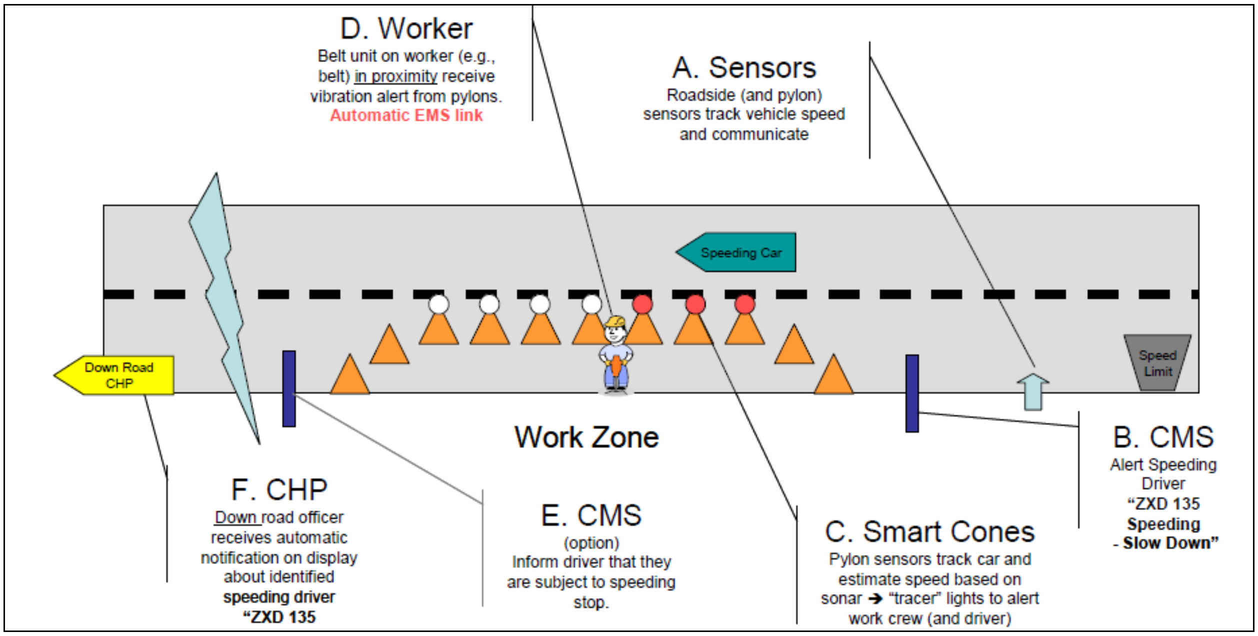
D. Worker
Belt unit on worker (e.g., belt) in proximity receive vibration alert from pylons.
Automatic EMS link
A. Sensors
Roadside (and pylon) sensors track vehicle speed and communicate
Speeding Car
Down Road CHP
Speed Limit
Work Zone
B. CMS
Alert Speeding Driver
"ZXD 135 Speeding - Slow Down"
F. CHP
Down road officer receives automatic notification on display about identified speeding driver "ZXD 135
E. CMS
(option)
Inform driver that they are subject to speeding stop.
C. Smart Cones
Pylon sensors track car and estimate speed based on sonar → "tracer" lights to alert work crew (and driver)

Figure 2. Functional components of aSE

Development of aSE is scheduled to begin during the summer 2009.

South Carolina Department of Transportation

SCDOT is using RSIP funds to implement a number of innovative technology-oriented solutions to improve safety by reducing speed-related, wet weather-related, and roadway departure crashes on a two-mile segment of rural US 25 immediately south of the North Carolina border in Greenville County. The roadway segment is located in an isolated mountainous area of the county. Some of the characteristics that make this an ideal location to deploy and test technologies are

- The area is subject to unusual weather patterns with frequent fog that dissipates slowly
- Roadway geometry changes dramatically from a long, straight parkway with grass shoulders in North Carolina to a curved grade with no shoulders in South Carolina
- There is insufficient signage for the grade and curve after entering South Carolina
- There are no retroreflective lane delineators along this area of US 25

With 87 percent of the crashes on this segment related to speeding, the use of variable speed limit (VSL) signing is a particularly important component of the system. The use of VSL, specifically during wet conditions, will enforce the need for reduced regulatory speeds during varying weather conditions. VSL may be extended along US 25 beyond project limits to address safety issues where weather and speed are significant contributing factors if proven useful at this site. This project will be the first application of VSL in South Carolina.

SCDOT will also construct overhead CMSs at the beginning of the northbound and southbound segments of the project. The proposed overhead signs will provide multiple safety benefits. The signs will be connected to both speed and weather sensors and display information as conditions warrant. This capability is particularly important as 62 of the 71 crashes on this short segment of roadway between 2003 and 2007 were listed “Driving Too Fast for Conditions” as a contributing factor, and over 84 percent of the crashes occurred during “Wet” conditions.

This section of roadway has also been identified as a location where cameras will prove both cost-effective and beneficial in determining the level of emergency response. The District Traffic Management Center in Greenville will receive live feed from these cameras to assist in managing any potential congestion and safety issues

The goal of this project is to reduce speed-related and hydroplaning crashes by 50 percent within one year of project implementation on the two-mile segment of the project through the implementation of multiple ITS components. Development of the system will begin in the fall of 2009.

Arizona Department of Transportation

ADOT is using RSIP funds to develop the Dual Use Safety Technology (DUST) Warning System to help reduce the loss of life, injury, and property damage on rural Interstate 10 in Cochise County. The proposed system has been designed to focus on two primary challenges:

- Visibility hazards caused by blowing dust on a 60-mile segment of Interstate 10 between Bowie and the New Mexico State Line
- Unexpected snow and ice in the Texas Canyon area of Interstate 10

The project will also provide early warning and detection for icy conditions in Texas Canyon as well as wind borne dust along Interstate 10 using several Environmental Sensor Stations (ESS) with a comprehensive sensor array. Each ESS site will also be equipped with a snapshot CCTV camera to confirm any potential low-visibility conditions. The enabling technologies that will be integrated to form the DUST Warning System include

- Wireless Ethernet Networks—Based on the WIMAX IEEE 802.16 standard, the wireless network solution will be integrated to serve as a cost-effective and reliable long-range communications backbone for the DUST Warning System.
- Photovoltaic Cells—Power for the remote telemetry sites will be derived from renewable solar energy generated using photovoltaic cells. Initially developed to power satellites, the technology has gained recent widespread acceptance for solar-powered remote telemetry and warning applications.
- Anemometers—These devices will be applied to measure wind speed to predict the potential for onset of high-wind conditions that may lead to reduced-visibility conditions.
- Forward Scatter Visibility Sensors technology—This uses the forward scatter principle of light in the presence of atmospheric particles to measure the coefficient and visibility. A high-intensity, infra-red LED transmitter is used to illuminate the sensor’s scatter volume. This results in a high signal-to-noise ratio and reduces the effects of background light variations. Visibility measurements are possible over a standard range of more than 10 miles.
- Light Emitting Diodes—LEDs have been in use as indicators for decades. As the reliability, heat tolerance, brightness, and efficiency have increased, LED technology has gained widespread acceptance for application as traffic signal or warning beacon indications.

The proposed system has been designed to reduce the loss of life, injury, and property damage by focusing on visibility hazards caused by blowing dust on a 60-mile segment between Bowie and the New Mexico State Line and unexpected snow and ice in the Texas Canyon area of Interstate 10.

Illinois Department of Transportation

IDOT is using RSIP funds to develop a countermeasure for two sections of roadway that have serious injury and fatal crash histories. At one location, a system alerts drivers of changing conditions by detecting any approaching vehicles. This will activate an LED-flashing beacon that is mounted over advanced curve warning signs.

At the second location, a countermeasure will be implemented to provide advanced warning of a two-way stop. The countermeasure will also use a vehicle-actuated LED to highlight the stop condition for motorists on the lower-volume minor route, as well as warn the driver on the major route of an upcoming intersection. A total of four beacons will be used at the intersection—one for each approaching leg of the intersection. It is anticipated that these systems will be operational in late 2009.

King County, Washington Department of Transportation

KCDOT is using RSIP funds to develop and implement two driver feedback signs that activate when a vehicle is detected and display the vehicle’s speed at two sites. The system will use radar to measure the vehicle’s speed. The display will flash when the measured speed is greater than the advised speed.

Warning signs will use radar to measure the speed of approaching vehicles. The display will then flash when the speed of the vehicle exceeds the advisory speeds. A variable message sign (VMS) may also be implemented, as part of the project. If implemented, signs will display a message (e.g., slow down) when the measured speed of the approaching vehicle exceeds the advised speed. It is anticipated that this system will be operational in late 2009.

California Department of Transportation

Caltrans in partnership with El Dorado County (California) Department of Transportation is developing a collision countermeasure system (CCS) on US 50 near the community of Camino in El Dorado County. The system consists of two types of actively illuminated warning signs located on the eastbound and westbound lanes of US 50 and loop detectors on an intersecting road. When a vehicle on the minor road is detected approaching the intersection, the illuminated signs will warn drivers on US 50 the presence of vehicles approaching the roadway. It is anticipated that the system will be operational in late 2009.

Iowa Department of Transportation

The Iowa DOT is developing a web-based version of the Traffic and Criminal Software (TraCS).[vii] TraCS provides rural law enforcement agencies the capability to improve the accuracy and completeness of crash data, make use of the crash data locally, and improve the timeliness of the crash data submission for inclusion in a statewide database for use by analysts and decision makers.

The goal of the web-based TraCS system is to electronically transmit and load the data into the state’s statewide crash database, with local agencies able to print a copy and query their crash reports to generate reports and create pin maps. This project, when completed, will assist Iowa, along with the 15 other

TraCS states and hundreds of rural areas within those states, to collect the crash data needed to make data-driven decisions about allocation of scarce resources, including safety improvements.

Web-based TraCS will allow states to deploy the data collection software to rural law enforcement agencies. Iowa has approximately 450 law enforcement agencies and has deployed TraCS to 190 of those agencies. Iowa DOT does not have the staff resources to deploy to the other agencies, which are the small rural agencies. If the data collection software is made available over the internet, the Iowa DOT will be able to provide password-protected access to all Iowa agencies. This will result in higher-quality data (edits and validations preclude many errors) in a timelier manner. This can and will be replicated by the 15 other states using TraCS. Development of TraCS is currently underway and is anticipated to be completed during the summer 2010.

Kansas Department of Transportation

Using RSIP funds, KDOT has entered into a unique partnership with the Prairie Band Potawatomi Nation to deploy ITS at three intersections along US 75 that provide access to the reservation’s housing population and medical and Tribal Government operations. ITS technologies that will deployed to enhance safety and improve winter road maintenance activities include one roadway weather information system station, one closed-circuit television camera, two portable DMSs, and flashing beacons and queue detection systems at two other locations along US 75.

PROJECT EVALUATIONS

Recipients of RSIP funds have agreed to collaborate with an independent entity in the evaluation of their projects. Projects funded through the ITS Program will be evaluated through a combination of evaluation studies that examine system component performance and the systems impact on enhancing safety on local and rural roads. As part of the evaluation process, institutional and technical challenges, as well as lessons learned and best practices, will be addressed and documented to assist other state and local transportation agencies. The independent evaluator will engage each funding recipient early in the development process to ensure that the results of the evaluations are as useful as possible to others considering similar projects.

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[ii] http://www.dot.gov/affairs/ruralsafety/ruralsafetyinitiativeplan.htm. Viewed 6 January 2009.

[iii] http://www.dot.gov/affairs/ruralsafety/ruralsafetyinitiativeplan.htm. Viewed 6 January 2009.

[iv] http://www.dot.gov/affairs/ruralsafety/ruralsafetyinitiativeplan.htm. Viewed 6 January 2008.

[v] http://safety.fhwa.dot.gov/local_program/. Viewed 6 January 2008.

[vi] Led by the Minnesota Department of Transportation (Mn/DOT) and University of Minnesota, Cooperative Intersection Collision Avoidance Systems—Stop Sign Assist (CICAS-SSA) targets the national problem of crashes at rural through-stop intersections, particularly those where low-speed, low-volume roads intersect high-speed, high-volume expressways.

[vii] TraCS is a sophisticated data collection and reporting software application for the public safety community. It provides organizations with a state-of-the-art information management tool to streamline and automate the capture and transfer of incident data in the field. Using the latest mobile computing technologies to capture and report incident data where it occurs, TraCS improves the accuracy, completeness, and timeliness of incident data and reduces user’s administrative duties and paperwork. TraCS was developed by the Iowa DOT with funding assistance from several federal agencies. From its conception, TraCS was designed and developed using a flexible architecture that, with minor modifications, could be transferable and easily adapted and customized for use by agencies in states/provinces other than Iowa.

Meeting Iowa’s Transportation Agencies’ Leadership Development Challenges

Duane Smith
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, Iowa, 50010-8664
desmith@iastate.edu

Bob Sperry
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, Iowa, 50010-8664
rsperry@iastate.edu

ABSTRACT

The fiscal environment for today’s public agencies includes reduced staffing levels, reduced capital resources, increased expectations for services provided, and increased restrictions for travel and educational sessions. In addition, there is a greater dependence on cross-training and on employees being proficient in many and varied job assignments.

The Iowa Public Employees Leadership Academy is a training program designed to develop better (and/or new) leaders and supervisors. It provides a curriculum to train the next generation of leaders replacing existing leaders when retirements occur. In addition, the Academy provides cross-training, allowing management the full use of the new leaders’ resources.

The development and implementation of the Academy has been conducted as a research project of the Iowa Highway Research Board (IHRB). The IHRB is composed of representatives from state, county, and local transportation agencies. A Technical Advisory Committee (TAC) has been assembled to oversee the development and implementation of the Academy. The TAC provided a vision and identified 10 core modules to be developed, presented online, and incorporated into professional association meetings and conferences.

As the TAC worked through the development of the first Academy modules, a process started to develop. The process includes 10 steps, starting with appointing a TAC and ending by recognizing the volunteers that contributed their time and efforts making presentations.

The products that came from this research project are an identity to be used in the production of module content and promotional materials (a logo), a marketing plan, integration into conferences, workshops and training activities for professional organizations and agencies, identification of measures of success, and a process to conduct peer reviews.

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The TAC and the development teams have learned many things and have smoothed out the process. Throughout the development of the Academy, many adjustments were made to facilitate the content development, the recording activities, and the posting online. Those lessons are provided to assist others developing similar activities.

Key words: academy—curriculum—Iowa Highway Research Board—Iowa Local Technical

Assistance Program (LTAP)—leadership—lessons learned

INTRODUCTION

The fiscal environment for today's public agencies includes reduced staffing levels, reduced capital resources, increased expectations for services provided, and increased restrictions for travel and educational sessions. In addition, there is a greater dependence on cross-training and on employees being proficient in many and varied job assignments. Responding to these issues involves a well-informed and coordinated work team that includes professionals, supervisors, technicians, lead workers, and workers. Becoming a coordinated work team demands training and interaction that produces a common knowledge foundation for agencies and employees to draw upon.

The Iowa Local Technical Assistance Program (Iowa LTAP) mission is to disseminate the results of research activities and to provide training for transportation agencies personnel. As an example, research such as that completed by the Corps of Engineers Cold Regions Research and Engineering Lab is used for solving problems associated with frost heaves on Iowa's roadway system. The National Research Council's Strategic Highway Research Program (SHRP) snowplow research has resulted in more efficient snowplowing equipment and techniques. Research conducted on pavements has been disseminated by the Iowa LTAP in the form of printed materials, technical articles, and presentations at workshops and conferences. Training conducted by the Iowa LTAP is focused for transportation professionals and workers.

In Iowa, the training and educational constraints are being addressed to some extent by providing non-credit, online training sessions. The Iowa LTAP, in conjunction with Iowa's public agency representatives, is providing that common knowledge foundation. An example is being provided to illustrate how the delivery of a typical training session is being adjusted for the benefit of Iowa's public agencies. The example presented is the Iowa Public Employees Leadership Academy (the Academy). A coordinated, non-credit training program does not currently exist in Iowa to provide training for the existing or upcoming managers and leaders in public agencies. Through the Academy, the Iowa LTAP is providing a coordinated, structured, non-credit educational program for that purpose.

The Academy is a training program designed to develop better (and/or new) leaders and supervisors. It provides a curriculum to train the next generation of leaders who will replace existing leaders when retirements occur. In addition, the Academy will provide cross-training relating to agency types, which allows management the full use of the new leaders' resources. It must be noted that the techniques and skills offered in the Academy can apply to all who wish to develop or sharpen their leadership and management abilities. This will be true whether the participants are employed in the private or public sectors.

BACKGROUND

Disseminating the results of research and making training available and affordable is the challenge when developing a competent workforce team. In the past, the Iowa LTAP has partnered with the Iowa chapter of American Public Works Association (APWA) to provide training across the state of Iowa. Many times, the attendees were the only ones in their agency to keep the work activities organized and on schedule. They needed to be at their work location first thing in the morning and facilitate the beginning of the work day. By the same token, they needed to be back to the work location to finish out the business day. In order to meet these requirements, training events were generally scheduled from 9:30 AM to 2:30 PM. This allowed the attendees to be at their workplace to start the day, and then travel up to one hour to the training site. The ending time allowed time for traveling back to the workplace in time to wrap up the day's business. Now, with the challenges of reduced budgets, reduced workforce, and increased travel restrictions, this format is not proving to be as successful. Travel and overnight expenses are sometimes barriers to attending workshops and conferences. Offering training opportunities online allows employees the opportunity to participate in training events without having to travel.

The development and implementation of the Academy has been conducted as a research project of the Iowa Highway Research Board (IHRB). The IHRB is composed of representatives that represent state, county, and local transportation issues and interests. A Technical Advisory Committee (TAC) has been assembled to oversee the development and implementation of the Academy. The TAC identified 10 core modules that would be initially developed, presented online, and incorporated into professional association meetings and conferences. The committee members are:

Bret Hodne, Chair	Iowa Chapter—American Public Works Association
Mark Bair	Iowa Secondary Roads Maintenance Supervisors Association
Bruce Braun	Iowa Chapter—American Public Works Association
Jim Christensen	Iowa County Engineers' Association
Ed Engle	Iowa Department of Transportation
Tom French	Iowa Secondary Roads Maintenance Supervisors Association
Pat Miller	Iowa Chapter—American Public Works Association
Kate Murphy	Iowa Department of Transportation
Dave Shanahan	Iowa County Engineers' Association
Duane Smith	Iowa LTAP / InTrans

The TAC's vision for the Academy includes the following principles:

- The Academy will be a leadership Academy for all employees wishing to improve themselves.
- It will be an educational/training program concentrating on leadership skills development.
- Courses will relate to increasing the participants' leadership abilities.
- The audience will normally include first-line supervisors and higher, but all those aspiring to become leaders will be most welcome.
- Presentations will be conducted largely by members of Iowa's professional community.
- Educational modules will be recorded and made available on Iowa State University's (ISU) Extension Continuing Education non-credit, outreach website.
- Modules will be available on a schedule that allows participants to complete them in a timely fashion.
- Review validation questions that will be included with each module and verify the student's understanding of the materials presented.
- Certificates of completion will be awarded.

The TAC has recommended a curriculum and initial course content for 10 core modules:

1. Supervisory Techniques and Skills
2. Basic Management Skills
3. Effective Communication Skills
4. Leadership Skills
5. Community Service/Customer Orientation Skills
6. Legal Understanding
7. Fundamentals of Government
8. Finance
9. Resource Management Skills
10. Operations and Maintenance

LEADERSHIP ACADEMY PROPERTIES

The Iowa LTAP partnered with ISU Extension, the Iowa DOT, the Iowa chapter of APWA, the Iowa County Engineer's Association (ICEA), and the Iowa Secondary Roads Maintenance Supervisor's Association (ISRMSA) to develop an online, non-credit curriculum to meet the challenges of today's public agencies' economic environment. The curriculum provides training to develop public agency workforce leadership. It is designed for employees who are interested in improving their leadership skills and developing an understanding of how to successfully conduct business or those wishing to develop these skills. This partnership has focused on developing the Academy and making the educational sessions available on the Internet at the ISU Extension Continuing Education website. The name is a bit misleading because the leadership skills and training sessions will apply to anyone wishing to improve or acquire leadership skills, whether they are in the public or private sector.

By making the Academy modules available online, the need for travel is greatly reduced, and the students can access the Academy at any time of the day or night, thereby matching their varied schedules. There are 10 modules that have been identified so far, and many others have been discussed. For each module, the content is developed and speakers who will deliver the educational message are identified. The modules are presented in front of a live audience where audience interaction is a key element of the presentations. The modules are recorded by ISU Extension technicians, edited for website applications, and posted to the ISU Extension Continuing Education website. The entry point to the Academy is via the Iowa LTAP and the ISU Extension Continuing Education websites. Other entry points will be identified and evaluated.

CURRICULUM DEVELOPMENT

Developing a curriculum that meets the public agency training requirements and lessens travel demands is a viable link in developing the leadership of tomorrow. This is the leadership that will manage the responsibilities of tomorrow and cope with continued reduced resources along with increasing demands for higher service levels.

As the TAC worked through the development of the first Academy modules, a process started to develop. That process is shown in Figure 1 below. The process includes 10 steps, starting with appointing a TAC and ending by recognizing the volunteers that contributed their time and efforts to make presentations.

TAC Appointed

The important traits that were considered when individuals were asked to serve on the TAC are listed:

- Have knowledge of leadership traits and qualities
- Understand what the job of supervisor includes
- Be connected with organizations and agencies that are a source of knowledge expert speakers
- Possess critical thinking skills

Identify the Audience

This critical step is necessary to ensure that the content is appropriate and the presentation materials are available to them. The audience for the leadership Academy is described:

- First-line supervisors who direct work crew activities
- Those desiring to move up from crew-level workers to first-line supervisors
- Anyone wishing to prepare for advancement within the leadership ranks
- Professionals wishing to sharpen their leadership skills and knowledge

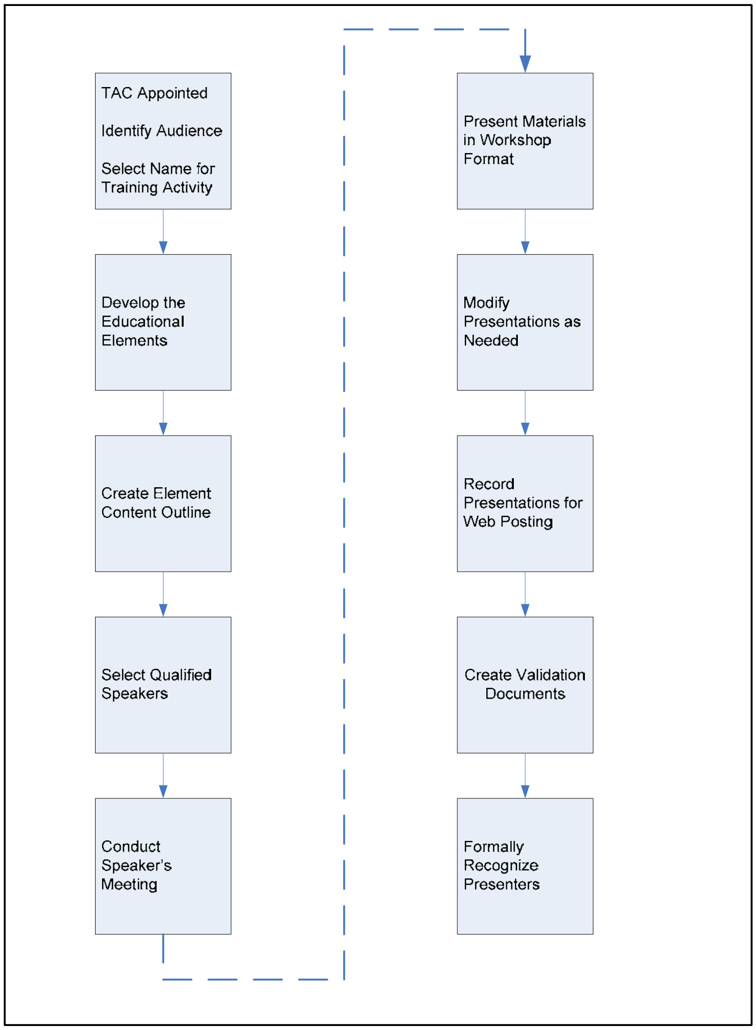
TAC Appointed
Identify Audience
Select Name for Training Activity
Develop the Educational Elements
Create Element Content Outline
Select Qualified Speakers
Conduct Speaker's Meeting
Present Materials in Workshop Format
Modify Presentations as Needed
Record Presentations for Web Posting
Create Validation Documents
Formally Recognize Presenters

Figure 1. Curriculum development

Select Name for Training Activity

The overriding thought for the name was an all-encompassing name for the audience that will participate. The decision was to name the Academy the "Iowa Public Employees Leadership Academy."

Develop the Educational Elements

A rigorous activity was conducted since such an Academy does not currently exist for Iowa public transportation employees. The following process defines the activities taken to develop the educational elements:

- Goals and objectives were established
- The Academy is to be a non-credit curriculum for non-professionals (but professionals will benefit from taking materials presented)
- Other Academy development efforts were researched
- The APWA's Leadership Academy was used as a guide during the development
- APWA's Leadership Academy guide was modified for the Iowa audience

Create Element Content Outline

This step was not conducted for the first modules developed. It didn't take long working with the speakers to realize this needed to be done prior to working with them. They need some direction and focus to keep on track and with the TAC intentions. This step provides that direction and focus.

- The end products consisted of written notes and power point presentations.
- These products were provided to the speakers once they were identified and had accepted the offer to make a presentation.
- The speakers have permission and are encouraged to make changes to the outlines and power point presentations based upon their expertise and experience.

Select Qualified Speakers

The qualities the TAC use as a guideline in identifying and selecting speakers are listed here as a reference:

- The speakers are known in Iowa and are respected for their knowledge and experience
- They are knowledgeable of the subject matter
- Someone on the TAC has seen them make a presentation(s)
- They have permission to participate and have control of their schedule

Conduct Speaker's Meeting

This is a critical step, and it should be scheduled well in advance of making the first presentations. This will develop confidence for the speakers and allow them time to make any adjustments they desire to make. The TAC selected them because of their expertise. Their expertise needs to be recognized by giving them permission to make the appropriate changes.

- Select a location that is convenient for the speakers.
- Review the goals and objectives.
- Provide an overview of the individual topics that are to be presented.
- Use the visuals that have been developed as a guide for the meeting.
- Agree on changes and adjustments to the agenda; let the speakers take the lead here.

Present Materials in Workshop Format

This step allows for a "dry run" presentation of the materials prior to taping the session for posting on the website:

- Schedule and advertise the session presentation the same as other educational workshops and conferences
- By having a workshop environment, the speakers are making their presentations to a live audience
- The speakers can monitor their presentation time and content prior to recording for the web posting
- If a registration fee is charged for the workshop, some of the development costs can be recovered
- The workshop should be considered the same as taking the class online and certificates of completion are awarded

Modify Presentations as Necessary

There are always changes that can be made to the presentation, whether for content, timing, or visuals. Any process should allow for this to occur.

- The presentations may be modified because of the length of the presentation or the content.
- This step assures that the key points are adequately covered.
- There is an opportunity to add clarifying graphics and visuals.
- The speakers gain confidence in making their presentations.
- All presentations should tie to each other and the central theme.

Record Presentations for Web Posting

It is best to do this step shortly after the workshop; 2–3 weeks seems to be comfortable for the speakers.

- Invite an audience and allow the speakers to make their presentations to a live audience rather than just to a camera.
- Encourage the audience to interact with the speakers to ensure the online version simulates a classroom type environment.
- Start the day with a light, interactive activity with the speakers that breaks the tension and relaxes them.

Create Validation Documents

These documents are the responses the students complete as they take the online class. The validation response reinforces the main points made by the speakers.

- The TAC made a decision early on in the development of the Academy to not have a graded response and a passing grade requirement. It was felt that for the intended audience, this would be a negative for some and might dilute the effectiveness of the Academy.
- These responses will be completed by the students at the end of each subject (presentation).
- A variety of formats should be utilized to avoid repetition of style.
- Responses may be in the form of answering a question, writing a response, picking the best answer from a list, or providing a true/false response.

Formally Recognize Speakers

The speakers are the key to the success of the Academy. They donate their time and expertise for the betterment of those taking the Academy classes. The speakers should feel good about their experience with the Academy and then be good supporters and encourage others to participate and perhaps even be willing to speak again.

PRODUCTS

The products that came from this research project are an identity to be used in the production of module content and promotional materials (a logo) along with a marketing plan. The Academy modules are sequenced, module content developed, and presentations scheduled. The Academy will be integrated into the conferences, workshops, and training activities of professional organizations and agencies. Measures of success will be identified and implemented, along with a process to conduct peer reviews.

BENEFITS

The Academy provides structured training for Iowa's public employees wishing to refine or develop management skills. No other program is available in Iowa for them at this time. The Academy modules are provided online and supplemented with face-to-face events held during professional organizations' conferences and workshops. The vision of the TAC is to provide computer-based training as a base and supplement it with face-to-face interaction. Given the condition of Iowa's economy and the resources available for Iowa agencies' training programs, an alternative presentation platform, such as provided by the Academy, is a solution for training. The Academy provides a format that minimizes travel and time away from the job.

LESSONS LEARNED

In conclusion, the TAC and the development teams have learned many things and have smoothed out the process. Throughout the development of the Academy, many adjustments were made to facilitate the content development, the recording activities, and the posting online. Those lessons are provided here in the hopes that they will assist others who are developing similar activities.

- Contact potential speakers early in the process.
- Have a clear idea of the subject and content they will deliver.
- Review the schedule of events as illustrated in Figure 1.
- Establish a timeline as quickly as possible and then stick to it.
- Most speakers will require the content be provided, as a minimum a draft power point presentation to use.
- The Principal Investigator (PI) should develop the validation responses to assure the content is reviewed and that there is continuity for the module.
- Iowa has many gifted and knowledgeable professionals that are willing to share their expertise and experience with others.
- Completing and posting a module should be celebrated since it is very rewarding for all who participate.
- Developing and implementing an Academy type activity becomes a team building activity with persons that may not have known all that well in the beginning.

Effectiveness of Dynamic Messaging on Driver Behavior for Late Merge Lane Road Closures

Thomas J. McDonald
Institute for Transportation
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
tmcdonal@iastate.edu

Shashi Nambisan
Institute for Transportation
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
shashi@iastate.edu

Robert Sperry
Institute for Transportation
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
rsperry@iastate.edu

ABSTRACT

Efforts to improve safety and traffic flow through merge areas on high-volume/high-speed roadways have included early merge and late merge concepts and several studies of the effectiveness of these concepts have been conducted, many using Intelligent Transportation Systems for implementation. The Iowa Department of Transportation (Iowa DOT) planned to employ a system of dynamic message signs (DMS) to enhance standard temporary traffic control for lane closures and traffic merges at two bridge construction projects in western Iowa (Adair and Cass Counties) on I-80 during the 2008 construction season. This presentation will summarize efforts to evaluate the effectiveness of the DMS system implemented on I-80 by the Iowa DOT. The primary objective of the efforts was to examine driver merging actions.

Data were collected over four weekends but only two yielded sufficient data for evaluation: one with two short periods of moderately impacted (transition) traffic flow and the other with an extended period of congestion flow. For both of these periods, a review of the data did not indicate a statistically significant impact on driver merging actions when the DMS messaging was activated as compared to free flow conditions with no messaging.

Unforeseen difficulties with data collection efforts also adversely affected the ability to draw statistically significant conclusions on the project. Examples of such difficulties include personnel safety issues associated with the placement and retrieval of counting devices on a high-speed roadway, unsatisfactory equipment performance, and insufficient congestion to activate the DMS messaging hampered efforts.

Key words: dynamic message signs—mobility—safety—work zones

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INTRODUCTION

Safety and mobility for traffic traveling through work areas on high-speed, high-volume, multi-lane roadways has been a concern for transportation agencies for many years. Over the years, many improvements in temporary traffic control have dramatically improved work zone safety for motorists and workers alike, but efforts for enhancing both safety and traffic flow have continued especially where lane closure can result in significant speed reduction, congestion, and delay.

One area where improvement efforts have focused in recent years has been in the merging practices of drivers in advance of lane closures on multi-lane roadways. Two techniques has been implemented and studied: early merge and late merge concepts. The names are quite self-descriptive. For early merge, traffic is encouraged to merge from a closed lane well in advance of the actual closure to avoid confusion and congestion at the point of closure. This concept may be most effective for relatively low traffic volume applications. The late merge concept encourages drivers to stay in their lanes until the actual merge point and then alternate in proceeding to merge into the remaining open lane(s) with the goal of maintaining maximum volume accommodation as long as possible. A late merge concept may be more effective at higher traffic volumes.

The Iowa Department of Transportation (Iowa DOT) has employed a standardized single lane closure temporary traffic control system with stationary mounted static work zone signs. Occasionally with higher traffic volumes, significant delays and queue build-up has resulted and the Iowa DOT desired to evaluate a late merge concept using dynamic message signs (DMS) to improve traffic flow through lane closures and reduce possible rear-end collisions. In addition, these signs can also be utilized to provide information to drivers about potential traffic slowdowns and delays occurring several miles in advance, thus reducing confusion and some frustration.

The purpose of this study is to evaluate any possible changes in driver behavior from the use of DMS sign messaging during periods of congestion and to make recommendations about this system’s potential value to Iowa’s current temporary traffic control (TTC) practices for long-term lane closures on high-speed/high-volume roadways.

LITERATURE REVIEW

Relying on the *Manual of Uniform Traffic Control Devices* (MUTCD), the Iowa DOT has used consistent design standards for selection, sequence, and spacing of traffic control devices for lane closures on its rural Interstate system, and the traveling public has grown accustomed to this standard.

However, early studies by Geza Pesti and Patrick McCoy in Nebraska (Pesti et al.1999) have shown that in areas of lower “commuter” traffic, these types of static signing failed to adequately handle congestion periods. Moreover, the addition of non-dynamic messages was also found to cause confusion and frustration when no congestion was present and drivers’ expectations were thus violated. This study by Pesti et al. (1999) led to the concept of variable and dynamic message signs.

The authors’ continued research (McCoy and Pesti 2001) on that topic led to the conclusion that the dynamic late merge concept can be a great safety benefit during times of heavy congestion. Having the ability to change the messages for drivers to correspond to changes in traffic flow and/or speed should promote a smoother transition as congestion develops and traffic speeds are slowed. This concept should minimize crashes as well as the frustrations of drivers during those slowdown periods. The authors also noted that selecting the most effective sign messages, types, and spacing seems to be a crucial element for each situation.

A 2004 study in Minnesota conducted by URS Corporation (URS 2004) concluded that the maximum volume throughput through single-lane construction areas on rural Interstates was approximately 1,600 vehicles/hour.

A more recent study in Virginia by Beacher et al. (2004) found a marked improvement of traffic flow when a dynamic late merge system (DLMS) was used, but only for a 3-to-1 lane reduction. No statistically significant change in the capacity was noted in a 2-to-1 lane reduction; however, little data were available for analysis. The study also noted that the percentage of heavy vehicles had a strong relationship to vehicle capacity or throughput, and a late merge concept became more efficient than the standard recommended MUTCD treatment as the percentage of heavy vehicles increased.

The Maryland State Highway Administration evaluated the effectiveness of dynamic late merge systems in highway work zone locations to measure the systems' impact on vehicle throughput, volume distribution, and queue lengths. This testing utilized portable changeable message signs (PCMS) to display messages to motorists when the dynamic late merge system is active. Remote traffic microwave sensors (RTMS) were used to detect traffic conditions. Standard TTC signs were in place to inform motorists of the work zone and merging traffic when the dynamic late merge system was inactive. The PCMS boards were activated when the RTMS detected lane occupancies of greater than 15% and were deactivated if occupancy was below 5%. The results showed that the use of a dynamic late merge system can improve traffic throughput, balance lane volume distribution, and reduce maximum queue lengths. However, placement of the PCMS and static TTC must be correct or there will be an increase in stop-and-go maneuvers by motorists confused by the messages being presented (Kang et al. 2006).

In October 2008, the FHWA's *Comparative Analysis Report: The Benefits of Using Intelligent Transportation Systems in Work Zones* (Luttrell et al. 2008) summarized the benefits of using TTS in work zones on five separate study sites in Washington, D.C.; Hillsboro, Texas; Kalamazoo, Michigan; Little Rock, Arkansas; and Winston Salem, North Carolina. Projects were accomplished between 1999 and 2006 and utilized different systems. However, with variable deployment schedules, data collection difficulties, and differing construction schedules, quantifiable benefits were difficult to assess at some sites, but a few showed some quantifiable benefits.

METHODOLOGY

With this study, the Iowa DOT desired to examine the potential benefits of using a system of speed sensors and DMS to enhance traffic flow through work zones. This system will be referred to as the DLMS. The observation sites were pre-chosen at two bridge replacement sites on Interstate 80 in western Iowa (Adair and Cass Counties). Interstate 80 in this area is a four-lane divided highway. DMS had previously been installed at these two sites for both directions of travel before this study commenced, except in the westbound lanes in Cass County. Therefore, it was determined this Cass County site would serve as a "control" location, with no DMS present. TTC for these projects consisted of complete closure of I-80 in the area of the bridge work with diversion of vehicles via median crossovers to sharing the opposing roadway in head-to-head travel with opposite direction traffic. This is a commonly used temporary traffic control scheme for this type of work in Iowa.

Operation of DMS Equipment

To properly collect data to analyze DLMS effectiveness, the process for activating these signs needed to be understood, and three possible modes of operation needed to be distinguished.

The DLMS consisted of several sign messaging units that were activated when the average of measured traffic speeds (see the sensors in Figure 1) dropped below pre-selected levels, indicating that *free* flow of traffic was hampered and congestion was beginning.

For most of the study period, the higher reduced average speed level or "trigger" was set at 50 mph for the first messaging, defined as *transition* flow for this study. Speeds below that trigger point activated the DLMS and turned on signs # 1–5 with their respective messages.

Speeds below the lower average measured speed of 30 mph were designated the *congested* flow category and activated the second messaging pattern, changing the messages on # 3–5 and turning on # 6 and 7 (shown in Figure 2), which were located 3.6 and 5.8 miles, respectively, in advance of the merge point.

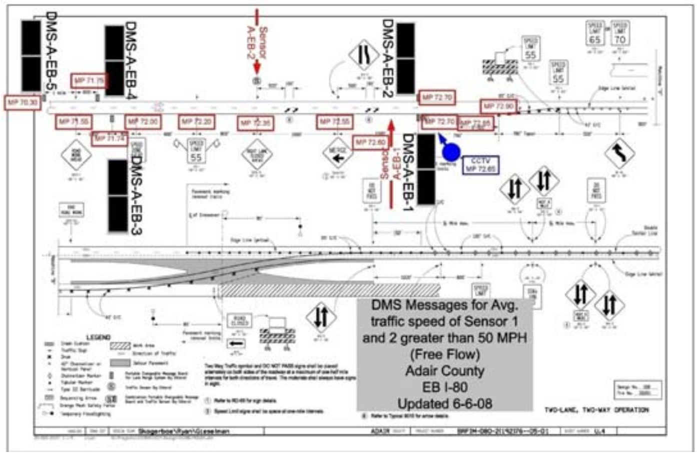
DMS-A-EB-5
DMS-A-EB-4
Sensor A-EB-2
DMS-A-EB-2
DMS-A-EB-3
Sensor A-EB-1
DMS-A-EB-1
CCTV MP 72.65
MP 70.30
MP 71.75
MP 71.55
MP 71.74
MP 72.00
MP 72.20
MP 72.35
MP 72.55
MP 72.60
MP 72.70
MP 72.90
DMS Messages for Avg. traffic speed of Sensor 1 and 2 greater than 50 MPH (Free Flow)
Adair County
EB I-80
Updated 6-6-08
TWO-LANE, TWO-WAY OPERATION
LEGEND
Skogerboe\Ryan\Gieselman
ADAIR

Figure 1. Relative positions of DMS 1–5, closest to merge point

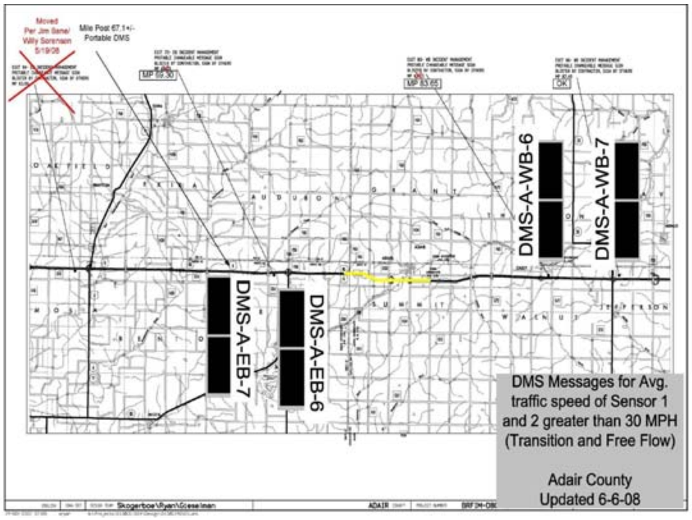
Moved Per Jim Sene/ Wally Sorenson 5/19/08
Mile Post 67.1+/- Portable DMS
MP 59.30
MP 83.65
DMS-A-WB-6
DMS-A-WB-7
DMS-A-EB-7
DMS-A-EB-6
DMS Messages for Avg. traffic speed of Sensor 1 and 2 greater than 30 MPH (Transition and Free Flow)
Adair County
Updated 6-6-08
Skogerboe\Ryan\Gieselman
ADAIR

Figure 2. Relative position of DMS 6 and 7, furthest from merge point

Refer to Table 1 below for DMS messaging at the average speed thresholds**.**

Table 1. DMS messages for these three situations

DMS Sign Identification	Traffic Flow Situations Message Presented		
	Free Flow	Transition	Congestion
EB-7	Off	Off	Slow Traffic Ahead Use Both Lanes
EB-6	Off	Off	Slow Traffic Ahead Use Both Lanes
EB-5	Off	Merge Ahead Use Both Lanes	Slow Traffic Ahead Use Both Lanes
EB-4	Off	Merge Ahead Use Both Lanes	Slow Traffic Ahead Use Both Lanes
EB-3	Off	Merge Ahead Use Both Lanes	Slow Traffic Ahead Use Both Lanes
EB-2	Off	Merge Here Take Turns	Merge Here Take Turns
EB-1	Off	Merge Here Take Turns	Merge Here Take Turns

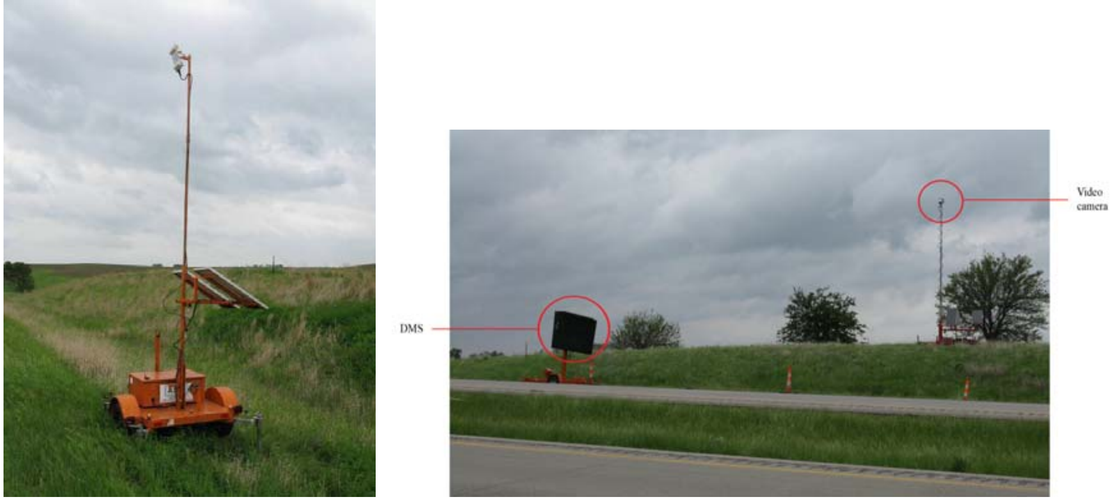

Figure 3. Typical components of the DLMS—speed sensor, left; and DMS and video camera, right

The prime contractor for the entire lane merge “package” was Quality Traffic Control (QTC) of Des Moines, IA. ASTI of New Castle, DE provided the cameras, Wavetronics speed sensors, and cellular communications. The portable DMS signs were manufactured by Precision Solar Controls (PSC) of Garland, TX.

Placement of Counters

To assess the merging actions by drivers both with and without the DLMS activated, the researchers collected traffic speeds, volumes, and classifications at three selected spot locations approaching and within the merging areas. These positions (P1, P2, and P3) are shown in Figure 4 for a typical approach installation.

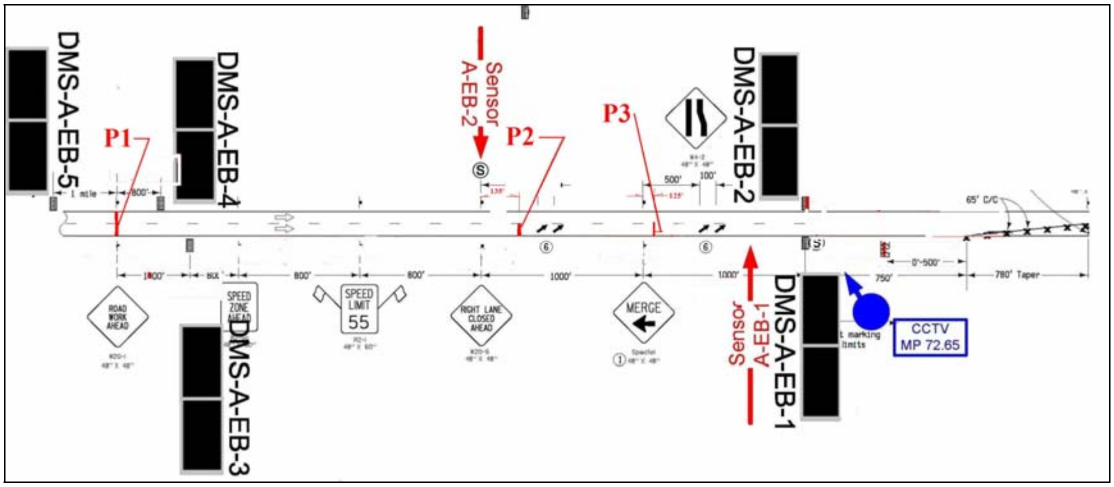
DMS-A-EB-5
P1
DMS-A-EB-4
Sensor A-EB-2
P2
P3
DMS-A-EB-2
1 mile
800'
1000'
800'
800'
1000'
1000'
500'
100'
65' C/C
0'-500'
750'
780' Taper
ROAD WORK AHEAD
SPEED ZONE AHEAD
DMS-A-EB-3
SPEED LIMIT 55
RIGHT LANE CLOSED AHEAD
MERGE
Sensor A-EB-1
DMS-A-EB-1
CCTV MP 72.65

Figure 4. Relative positions of counters with speed, static, and DMS

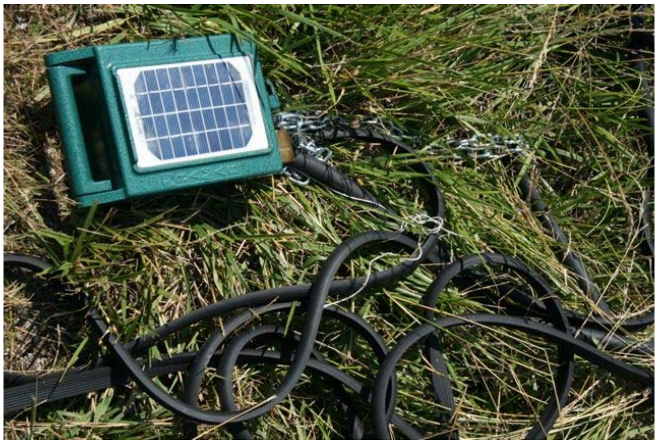

Figure 5. Typical Jamar TRAX Flex HS counter and HD half-round road tubes

By using four properly spaced data collection road tubes at the P1 counter location, traffic volume, speed, and vehicle classification were obtained in both lanes simultaneously. Two properly spaced road tubes at the positions P2 and P3 were required to record the traffic volumes, speeds, and vehicle classifications in the lane being closed. Open lane data were determined mathematically at both the P2 and P3 positions as the difference in the total P1 information (both lanes), minus the data for the respective closing lane information at P2 and P3. The typical Jamar counter used at all these positions is shown in Figure 5.

Once sufficient data were obtained, driver behavior was to be defined by the percentage of drivers (by vehicle classification) that remained in the closing lane at P2 and P3 during the three possible DMS messages. Once the parameters were selected by the DOT to determine when the DMS sign messages would activate, all traffic data for both lanes were determined at P1, and the vehicle percentages for each desired element and situation were calculated and tabulated. The process of sorting, calculating, and tabulating the data was repeated for any time periods when the measured traffic speed fell below the trigger speeds at P2 and/or P3, which activated the DLMS. Final conclusions for the DLMS effectiveness were to be based on the locations of lane merges, as defined by significant shifts in the noted percentages of vehicles, by classification remaining in the closing lanes at positions P2 and P3. The vehicle classifications for this study were 2 axle (passenger vehicles), 3–4 axle (single unit trucks), and 5+ axles (truck-trailer combinations).

DATA COLLECTION

Data collection was conducted over four separate weekends. The data collection periods were from Friday at approximately 12:00 p.m. until the following Monday at approximately 8:00 a.m. Traffic volumes during these time periods were anticipated to be highest at two separate approaches to the construction sites on I-80 in Adair and Cass Counties.

The Jamar road tubes were installed by laying the tubes in the vendor specified setup pattern and using 4 in. wide road tape to hold the tubes to the pavement. At position 1, two tubes were laid across both lanes and two tubes across one lane. At positions 2 and 3, the tubes were only installed in the lane that was to be closed. Typical road tube installations are shown in Figure 6.

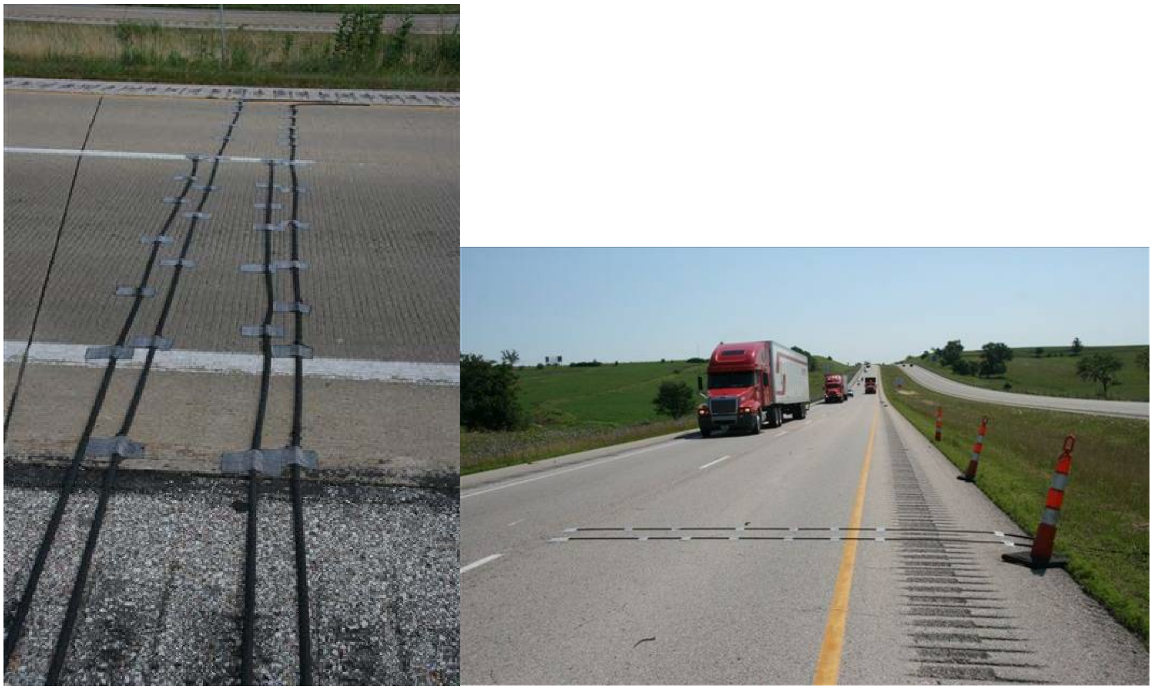

Figure 6. Typical P1 (left) and P2 or P3 (right) installation

Although Iowa DOT maintenance staff provided traffic control by temporarily closing a lane for installation and removal or the road tubes, placement of the cross road tubes at P1 locations and near the centerline at P2 and P3 locations was still potentially hazardous. To reduce the time of exposure to moving traffic, NuMetric Hi Star NC-97 plates were also installed, first as back-up devices, and finally as the only data collection units on the final weekend. These plates were centered in the middle of a lane and a cover of 12 in. wide road tape was placed over the top of the unit and the corners and edges were taped down with strips of 4 in. wide road tape (Figure 7).

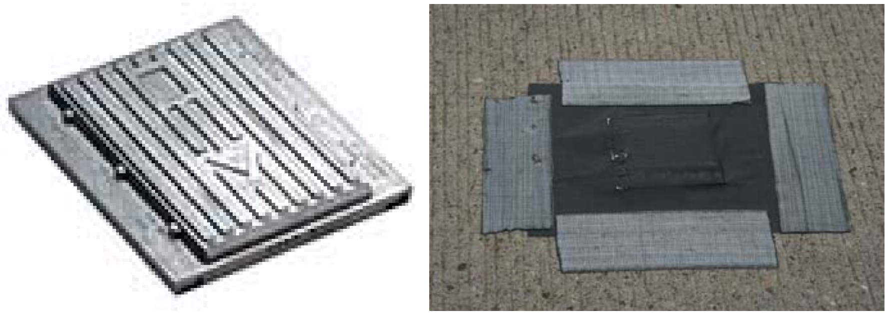

Figure 7. NuMetrics HiStar Model 97 and in position on roadway

Although cutting the tape and removing the road tubes from the pavement was a relatively quick operation, removal of residual tape from the tubes after use was a very extensive and laborious effort, although it was necessary for repeated usages. The NuMetrics plates initially used were older (1997) models and did not function well. However, new models acquired later did appear to provide usable data. Side-fire radar units, commonly known as “Wavetronics” in Iowa, were considered as an alternative for data collection, but experiences of other researchers have shown that in addition to requiring “binning” of the data into time intervals, these units cannot differentiate between lanes and may not record data accurately when two vehicles are parallel or in close proximity in adjacent lanes.

Weekend Summaries

August 1–4, 2008

Westbound Approach to Lane Closure in Cass County

No DMS signs were on site in this control location. Heavy-duty road tubes with Jamar traffic counters and NuMetrics electronic plates were both at this location for the observation period from Friday afternoon through the following Monday morning.

Westbound Approach to Lane Closure in Adair County

DMS signs were in place, with the trigger speeds preset with a 50 mph upper limit and a 30 mph lower limit for activating the sign messaging as was explained earlier. Heavy-duty road tubes were used for data collection.

August 8–11, 2008

Eastbound Approach to Lane Closure in Adair County

DMS signs were in place, with the trigger speeds preset for a 50 mph upper limit and a 30 mph lower limit for activation of messaging. In addition to the heavy-duty road tubes and Jamar counters, NuMetrics plates were again placed to gather data. The Jamar counter at location P2 did not function correctly, but the back-up NuMetrics plate did collect data for a substantial portion of this period. Therefore, the information gathered by both methods was compared and statistically tested for correlation and, when possible, the plate data was used to complete the information needed for analysis. Iowa DOT maintenance personnel reported that an extensive traffic backup occurred in the EB direction at Adair County on

Friday, August 8, from the early afternoon until approximately 9:00 p.m. and also again on Sunday, August 10, from about 5:00 p.m. to 9:00 p.m.

Eastbound Approach to Lane Closure in Cass County

DMS signs were in place with the trigger speeds preset for a 50 mph upper limit and a 30 mph lower limit for activation of messaging. Heavy-duty road tubes with Jamar traffic counters were used for this observation period from Friday afternoon through the following Monday morning. A severed road tube found at the P2 position on August 11 may have resulted when the metal end plate became embedded in a vehicle's tires.

August 14–18, 2008

Eastbound Approach to Lane Closure in Adair County

To reduce activation times for the DMS signs, the Iowa DOT reduced the trigger speeds to a 40 mph upper limit and a 20 mph lower limit for activation of the messaging. A combination of heavy-duty road tubes and NuMetrics plates was utilized for data collection, but due to malfunctioning equipment and lack of traffic congestion periods, no usable data were obtained.

Westbound Approach to Lane Closure in Adair County

Trigger speeds for DMS sign activation were also used here, and again, a combination of road tubes and NuMetric plates were used for data collection. No usable data were collected from this observation period.

October 17–20, 2008

One final weekend was selected for data collection in anticipation of high traffic volumes generated by a popular sporting event. Only one site was used for data collection.

Eastbound Approach to Lane Closure in Cass County

Trigger speeds for the DMS signs were set with a 40 mph upper limit and a 20 mph lower limit for activation of the messaging. Only NuMetrics plates were used for data collection but the anticipated traffic congestion did not occur and no usable data were collected.

DATA SUMMARY

Overview of Data

Two short periods of *transition* speed messaging, one a period of 6 min involving some 211 vehicles, and a second period of 39 min involving about 1,038 vehicles occurred during the afternoon of August 1 at the westbound Adair County site. These events were the only evidence found of the initial DMS sign activation during a transition period (average speed falling below the upper trigger).

A *congestion* speed event during the weekend of August 8–11 was of sufficient duration (about 8½ hours) that approximately 5,000 vehicles were affected. Many of those vehicles indicated speeds of less than 10 mph at the P1 position, which is the location of the initial static TTC signing (Road Work Ahead) from the point of lane closure. This speed reduction and traffic back-up period extended from 12:30 p.m. to 9:00 p.m. on August 8.

There was also a reduced speed and queue build-up period with speeds less than 10 mph on the evening of Sunday, August 10, between about 5:45 p.m. and 8:15 p.m. This period could not be analyzed because one of the Jamar counters had stopped operating earlier and a full set of data for the period was therefore unavailable. However, the active DMS signs that were most distant (9.5 miles) from the closure point provided additional warning to over 8 miles of potentially queued traffic, which included vehicles that had not yet reached the static TTC signing. Therefore, the DLMS did provide additional information and degree of safety for drivers, despite the apparent minimal impact on driver behavior regarding merging actions.

A summary of collected late merge data is illustrated below in graphical form. These graphs show the percentages of vehicles in the closing lane at locations P1, P2, and P3, by classification under all the available DMS options. The information is first shown for the control situation in the top left graph, where no DMS signs were in place. In the top right graph all usable data was combined from all weekends where the DMS signs were in place but not activated (i.e., the speeds remained above the upper trigger speed). The graph on the bottom left shows the merging data for transition flows of traffic (DMS signs 1–5 activated). And finally, the graph on the bottom right illustrates the data for the flow of traffic during congestion periods with all DMS messaging activated.

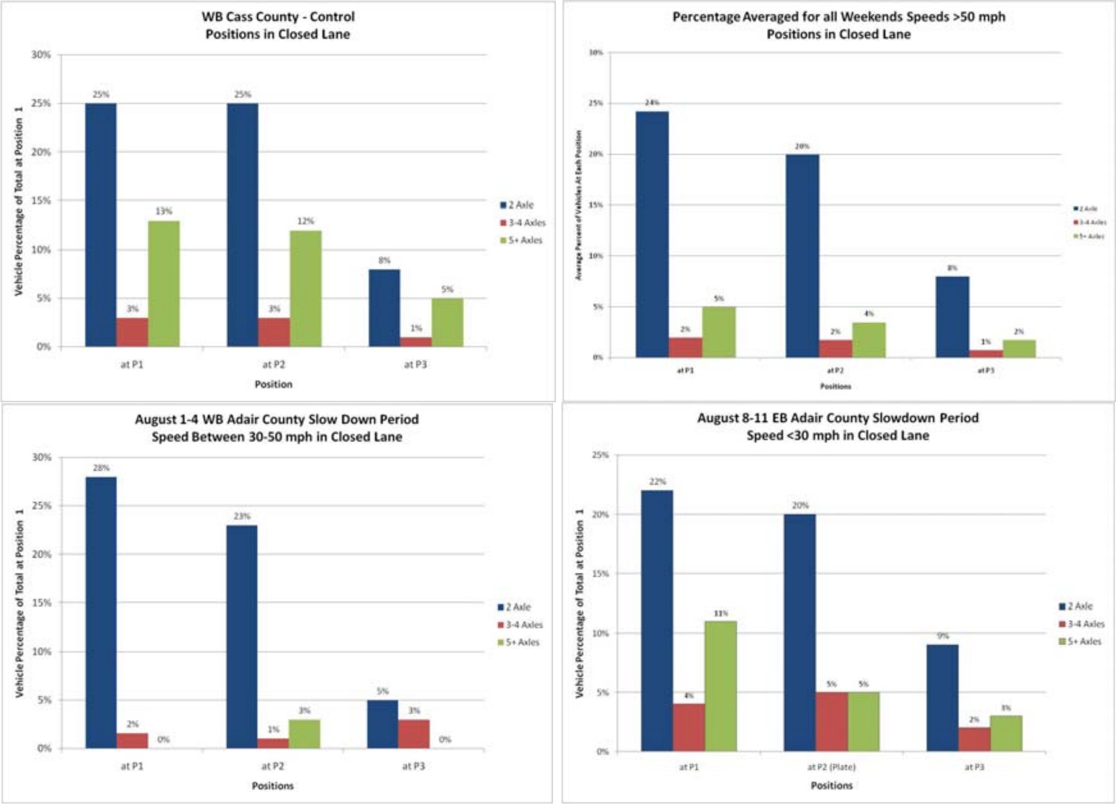

Figure 8. Percentage of total traffic, by classification, traveling in the closing (merging) lane

Note that the percentage of early merging taking place by cars and heavy trucks is less with no DMS signs present than with them in place but not activated. This might indicate that the presence of DMS signs on the site promotes early merging; however, in comparing merging practices for all three vehicle classifications in all four DMS messaging options, little discernable impact from the DMS signs can be concluded.

Comparison of Data Collection Systems

During data review and analysis, some significant variations in the data were noted between collection systems. To assess these differences, a comparison of road tube and NuMetrics plate data was made with data from the Iowa DOT permanent automatic traffic recorders (ATRs) at a Cass County (Atlantic) site. No definitive conclusions could be drawn from this comparison and more evaluation on this concern is warranted.

CONCLUSIONS AND RECOMMENDATIONS

Collection of relevant project data for analysis proved to be challenging. In addition to safety issues with the placement and retrieval of counting devices in close proximity to moving traffic on an Interstate highway, unsatisfactory equipment performance and insufficient traffic congestion to activate the DLMS hampered data collection efforts. The numerous problems with the collection equipment used as detailed in the project report compromised efforts to gather uniform, consistent, and relevant data.

Although the Jamar road tube data collection system seemed to be the most consistent and reliable devices used in this study, the severe damage to the heavy-duty, “D” road tubes from high-volume commercial traffic resulted in incomplete data collection because of tube failures early in the collection process. If this study were to be repeated, a more resilient and reliable hold-down system than tape should be considered for the tubes. Additionally, a more secure method for retaining the necessary tube end plugs in place needs to be developed. Using flat metal plate hold-downs at both ends of the tubes (off the traveled way where possible) would be a recommendation, but fastening those plates securely to the pavement structure would remain problematic both in potential damage to the pavement surface and in the time of exposure to moving traffic during the installation process.

For less exposure to moving traffic during equipment placement and pickup, the use of the newer model NuMetrics plate counters (NC-200) should be strongly considered, especially if this system can provide accurate and reliable data. The newer plates, which have the capability of identifying individual vehicles, seem to provide improved versatility and reliability than the older (Hi Star NC-97) models.

Since neither of the project data collection systems were found to be entirely reliable and comparable, further comparison testing of data collection devices, including the Iowa DOT’s ATR counters need to be undertaken to determine comparative accuracy and the circumstances when each system is most appropriate for use. It is recommended to undertake a comparative study of traffic data collected by road tubes, plate collectors, DOT ATR units, side fire radar units, and manual counts. However, it should be noted that variation in data from the Jamar road tubes and NuMetrics plates, while statistically significant, were in fact quite minor and acceptable for most studies.

Although data collection equipment did not function properly at times and traffic volumes at the collection sites were not sufficient to activate DMS messaging during much of the study period, some conclusions can be drawn and recommendations made following analysis of the usable data.

Results of statistical analyses of the data from two weekends when traffic speed reductions and congestion were observed did not indicate any significant impact on driver merging behavior, regardless of vehicle classification, from the DMS messaging deployed. Other DMS sign deployment arrays and variation in messaging may yield other results.

It should be noted that DMS messaging conflicted with existing static TTC signing during full activation with static signing indicating merging and the DMS signs showing “Use Both Lanes.” This apparently conflicting information may have resulted in confusion for drivers and less compliance with the late merge option. Future deployments of a DLMS could consider and address this possible conflict.

It was concluded that traffic volumes on I-80 in the western Iowa study sites may be insufficient to adequately test the capability and effectiveness of the DLMS. Therefore, it is not possible to predict a lane capacity (volume per hour), at which a system like this should be deployed. The traffic volumes observed with this study were, for the most part, below that level. The use of this type of DLMS to encourage late merging and potentially enhanced traffic capacity might be considered for other, more heavily traveled segments of Iowa’s Interstate system where hourly volumes are closer to a single-lane capacity of approximately 1,500 vehicles per hour (vph). In fact, current DOT practice is to consider extraordinary mitigation for potential delay and queue build-up when traffic volume is expected to surpass 1,350 vph per lane.

Although the DLMS had been in service for several weeks before this study began, it was obvious that with a complicated system such as this, numerous technical difficulties can, and do, occur. Also, the small number of activations, or trigger events experienced at these sites (as recorded by both the data collectors and the e-mail notification system that was developed later to advise interested parties of potential traffic slowdowns) would make the DLMS a very expensive tool unless deployed where frequent activation would be assured. This also points to the need to evaluate the speed thresholds to trigger the DMS (50 and 30 mph for most of this evaluation).

Activation of advance warning messaging on the DMS units located most distant from the actual merge point apparently provided sufficient information to drivers during the congestion period on the August 8–11 weekend since no crashes were recorded or complaints received from drivers during that time. Therefore, using traffic speed sensors, real-time activated changeable message signs located in advance of lane restrictions could possibly be used to advise traffic of potential congestion ahead less expensively than a complete DLMS. The type of system deployed for this study should probably be reserved for roadways with considerably higher traffic volumes than usually exist at the locations evaluated in this study. With more frequent periods of reduced speeds and congestion, the DLMS would prove to be much more effective.

ACKNOWLEDGMENTS

The authors would sincerely like to thank the Iowa Department of Transportation (Iowa DOT) for sponsoring this research, and especially Willy Sorenson, PE, Intelligent Transportation Systems Engineer, for advice and reference materials provided. Special thanks are also due to the Iowa DOT maintenance personnel in Adair County, who provided the researchers with much needed protection from traffic on the Interstate; Frank Redecker from Iowa DOT District 5 for the gathering of traffic data with NuMetrics traffic counting plates; and Dr. Shauna Hallmark of the Center for Transportation Research and Education for her assistance with the statistical review of the data.

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2008 Floods in the City of Des Moines

William G. Stowe
City of Des Moines Public Works Department
216 SE 5th St.
Des Moines, IA 50309
wgstowe@dmgov.org

ABSTRACT

This presentation reviews the City of Des Moines' experiences in the floods of 2008, with specific attention on the Des Moines River and the Raccoon River flood stages and lessons learned from 1993.

Key words: City of Des Moines—floods of 2008—flood stages

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Effect of Pavement Type on Fuel Consumption and Emissions

Palinee Sumitsawan
Department of Civil Engineering
The University of Texas at Arlington
P.O. Box 19308
Arlington, TX 76019-0308
palinee.sumitsawan@mavs.uta.edu

Stefan Romanoschi
Department of Civil Engineering
The University of Texas at Arlington
P.O. Box 19308
Arlington, TX 76019-0308
romanoschi@uta.edu

Siamak A. Ardekani
Department of Civil Engineering
The University of Texas at Arlington
P.O. Box 19308
Arlington, TX 76019-0308
ardekani@uta.edu

ABSTRACT

The effect of pavement type on fuel consumption was investigated. Significant differences in fuel consumption and emissions rates were observed on rigid versus flexible pavement surfaces. The difference in rates could result in substantial differences in the total energy consumption and carbon footprints during the design life of roadway facilities and should be considered in life cycle cost analyses of alternative designs. Fuel consumption measurements were made on multiple runs by driving an instrumented van over two new pavement sections: a rigid and a flexible section of two parallel city streets. The two sections were both tangent sections with identical gradients and similar roughness. All other factors that could influence fuel consumption were either controlled or kept the same during the test runs. Two different driving modes were also performed: constant speed of 30 mph and acceleration from zero to 30 mph at a rate of 3 mph/second. All tests were conducted under dry pavement conditions with a four factor-level experimental design—two pavement types and two driving modes. The differences in fuel consumption rates were determined to be statistically significant at 10% level of significance for the constant speed runs with the rate for the rigid section being lower. Under the acceleration mode, while the rigid pavement runs again showed lower rates, the differences were not statistically significant. The total fuel consumption amounts were then used to estimate the total annual CO_2 productions in the Dallas-Fort Worth region in Texas under each pavement surface type. Under similar urban driving conditions (30 mph), the rigid pavement results in a total fuel savings of about 177 million gallons and a reduction of approximately 0.62 million metric tons of CO_2 per annum.

Key words: carbon footprints—cleanup cost—emissions—fuel consumption—fuel measurement

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RESEARCH OBJECTIVES

The goal of this study is to investigate the effect of pavement type on fuel consumption and emissions. The study emphasis is on urban driving cycles at non-highway speeds, as more than half of the vehicular fuel consumption in the United States is due to urban driving. If significant differences in fuel consumption and emissions rates are observed across various pavement surface types, they may result in substantial differences in the total energy consumption and carbon footprints during the design life of roadway facilities. As such, those differences should be considered in life cycle cost analyses of alternative pavement designs.

The proposed study has entailed the use of an instrumented van to make fuel consumption measurements over two new pavement sections—a Portland cement concrete (PCC) and an asphalt concrete (AC) section. The two sections selected have similar geometric characteristics and differ only in the type of pavement. They are both tangent sections of two parallel collector streets in the city of Arlington, Texas, and have identical gradients and similar roughness. In the course of the fuel consumption measurements, every attempt is made to either control all other factors that could affect fuel consumption or keep the factors that cannot be controlled the same. These include vehicle weight, fuel tank level, tire pressure, ambient temperature, humidity, and wind speed and direction. Two different driving modes (constant speed and acceleration) are used in the test run measurements. For each run, in the cruise mode, the speed is kept at 30 mph, while in the acceleration mode, an acceleration of 3 mph/second is used. Tests are conducted under dry pavement condition. A four factor-level experimental design is utilized, including two pavement types (PCC vs. AC) and two driving modes (constant speed vs. acceleration). To obtain statistically meaningful results, six runs are made for each factorial combination, resulting in a total of 24 runs.

Given the millions of vehicle miles traveled (VMT) annually in major U.S. cities, even minute reductions in the fuel consumption rate per mile can be significant in the total fuel consumption and emissions savings during the design life of a project. To this end, as part of this study a procedure will be developed to quantify, given an estimate of the VMT and the vehicle mix, the total fuel consumption and carbon footprints over the design life of a pavement. Although the focus of the study is on city street urban driving where air quality concerns are paramount, the results could be easily extended to other classes of roadways.

LITERATURE REVIEW

The Transportation Research Board (TRB) Special Report 285 states that the vehicular fuel consumption accounts for half of the total energy consumption in the United States and about half of that amount is estimated to be due to the urban city driving at speeds below 40 mph. The oil crisis of the 1970s led to numerous research studies on vehicular fuel consumption. This led to advances in automotive design with lighter vehicles with more efficient engines, more energy efficient tires, smoother roadway alignments, and traffic engineering measures such as better timed traffic signals and national speed limit regulations.

The Elemental fuel consumption model developed by scientists at the General Motors (GM) research lab in Warren, Michigan, (Evans, Herman, and Lam 1976) was the widely accepted model among the fuel consumption models developed in the 1970s. This model showed that the fuel consumption in a vehicle varies greatly due to variables such as speed, acceleration-deceleration cycle, vehicle mass, and mechanical conditions of the vehicle such as tire pressure, wheel alignment, carburetion system, ambient conditions, and pavement surface conditions. The model speculated that about 75% of the variability in a vehicle’s fuel consumption is explained by speed alone. Also, an important factor influencing the fuel consumption rate is the rolling pavement resistance, which is primarily a function of the pavement surface condition and type. The fuel consumption differences due to rolling resistance were expected to be particularly significant for trucks and other heavy vehicles.

The most comprehensive study of the effect of pavements on fuel consumption was funded in the early 1980's by the World Bank as part of the Highway Design Model development effort (Archondo-Callao and Faiz 1994). The life cycle cost in the Highway Design Model included user costs in addition to conventional construction, maintenance, and rehabilitation costs. The user costs were mainly the vehicle operating costs and exogenous costs, such as the cost the society incurs as the result of road usage. The vehicle operating costs contained the vehicle characteristics, such as engine size, speed, tire conditions, etc., and the road characteristics, such as smoothness and slope of the longitudinal profile. The smoothness and slope of the longitudinal profile became the only pavement characteristics used in the model for estimating the vehicle operating costs. The other pavement characteristics, such as the pavement type, became statistically less significant since data from both paved and unpaved roads were used. To enhance the Highway Design Model work, a New Zealand study (Walls and Smith 1998) further suggested that the smoothness of the longitudinal profile has little impact on the fuel consumption for paved roads in good condition.

Studies by Papagiannakis and Delwar (1999, 2001) resulted in a software program that highlighted the importance of incorporating vehicle operating costs in the life cycle cost analysis (LCCA) of pavement projects. Their findings were later implemented in the Pavement Management System program of the Washington State Department of Transportation. They also paid special attention to the effect of roughness on the vehicle operating costs to illustrate the increase in these costs with the deterioration of the pavement. In addition, the research by Zaniewski, Butler, Cunningham, Elkins, Paggi, and Machemehl (1982) and by Zaniewski (1989) has been the only effort to date to systematically assess the effect of pavement surface material type on fuel consumption. Their study pointed out that fuel consumption of a truck when travelling on PCC pavements is lower than when travelling on AC pavements. Because their study was focused on fuel consumption of trucks on highways and also due to other limitations of the methodology employed, this study has received substantial criticism. Partly due to these issues, Zaniewski's findings have not been widely adopted by the pavement engineering community. Zaniewski's findings could also allow incorporating fuel economy improvements and emissions reductions in LCCA of design alternatives for highway pavements. However, it is not readily clear whether and to what extent they are applicable to city streets, where urban carbon footprint is an increasingly important consideration in the analysis of design alternatives.

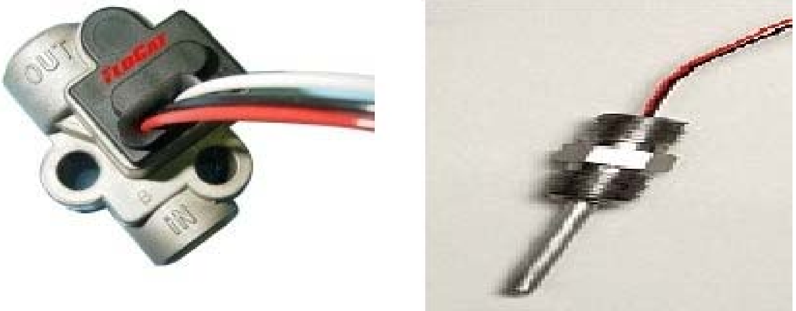
Vehicular fuel consumption and emissions are two crucial concerns in both transportation and environmental issues. Mobile sources generate VMT, and as they consume energy, they are the leading contributors to air pollution. According to the U.S. Bureau of Transportation Statistics (BTS), there were 250,851,833 registered vehicles in the United States in 2006. Approximately 93.5% of those vehicles were classified as passenger cars, SUVs, or single-unit trucks. Among three common fossil fuels—petroleum, natural gas, and coal—96% of the 2007 United States' primary transportation energy consumption relied on petroleum or crude oil (Energy Information Administration, U.S. Department of Energy). This trend continues despite the oil price increases, which peaked at over $140 a barrel in June 2008. Gasoline, which is the main product from crude oil refining, is one of the major fuels consumed in the United States with a consumption level of over 142 billion gallons in 2007. The vehicular fuel consumption accounts for about half of this amount. As such, the transportation sector is the largest emitter of CO_2 among energy-use sectors, such as industrial, residential, and commercial sectors. In motor vehicles, CO_2 is the by-product of the combustion process released to the atmosphere as a tailpipe emission. It is one of the greenhouse gases that causes global warming. Between 1990 and 2007, the energy-related CO_2 emission of the transportation sector grew 27.7% over the 17-year period and has grown by 1.4% per year since 1990 (Energy Information Administration, U.S. Department of Energy). As a result, improving energy efficiency of the transportation sector, including improving vehicle shape, engine, tire tread, and roadway design, plays a vital role in reducing fuel consumption and exhaust gas emissions.

The current study sets out to systematically measure the influences of pavement surface type on fuel consumption with an emphasis on urban streets. Once these influences are quantified, a secondary objective is to estimate the associated carbon footprint and cleanup costs in analyzing the life cycle cost of city street pavement alternatives.

EXPERIMENTAL DESIGN AND DATA COLLECTION

In achieving the research objectives, fuel consumption measurements are made using an instrumented vehicle driven over two types of pavement surfaces (PCC and AC) under two driving modes (constant speed and acceleration). In order to isolate the effect of pavement type on fuel consumption, attempts are made to either control or record all other key variables that might influence fuel consumption. These include vehicle weight, tire pressure, wind speed and direction, ambient temperature, atmospheric pressure, humidity, elevation, roadway gradient and curvature, and surface roughness.

The tests are performed using an instrumented vehicle equipped with an onboard data acquisition system. The fuel sensor, the temperature sensors, and the data acquisition system (Figure 1) are hooked up to the engine as schematically shown in Figure 2. Two fuel sensors measure instantaneously the amount of fuel entering the engine and returned to the tank. The temperatures of the fuel entering the engine and returning to the tank are also measured using two temperature gauges. In addition to the fuel amounts and fuel temperatures, the data acquisition system also records the instantaneous vehicle speed.



(a) Fuel meter (b) Temperature gauge

(c) Data acquisition system

Figure 1. On-board instruments

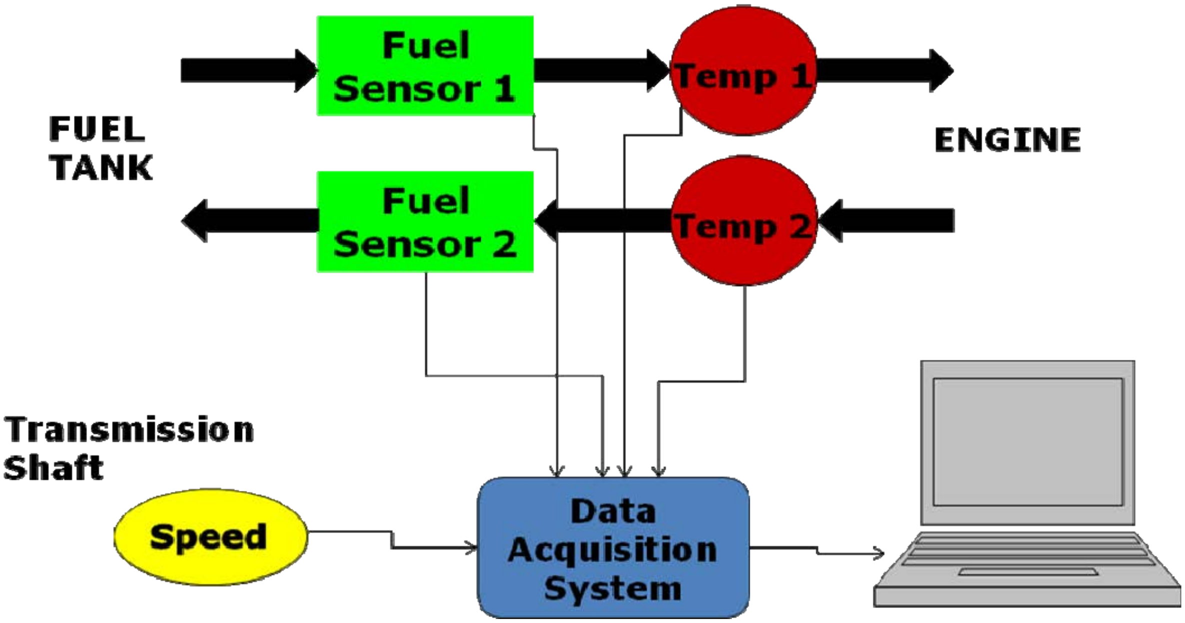

Fuel Sensor 1
Temp 1
FUEL TANK
ENGINE
Fuel Sensor 2
Temp 2
Transmission Shaft
Speed
Data Acquisition System

Figure 2. Schematic diagram of the sensor and the data acquisition system

Two types of city street sections, a PCC versus an AC, have been selected. Except for the surface material, the two sections have similar gradients, curvature, age, and roughness rating. The PCC section selected for this research is the Abram Street located in Arlington, Texas. The AC test section is the Pecandale Drive, also located in Arlington, Texas. The two sections run parallel and are two city blocks apart. The Texas Department of Transportation made roughness measurements for these sections resulting in International Roughness Index (IRI) measurements of 174.6 in/mile for the PCC and 180.6 in/mile for the AC section. These IRI values are only 3% different and are both in the IRI range for new pavements. Regarding the profile of the two road sections, a survey was performed. It was determined that the average longitudinal gradient for the two sections was identical at +1.2% in the eastbound direction (direction of observations).

The variables recorded for each measurement run included the following:

- Date
- Time
- Ambient air temperature
- Atmospheric pressure
- Humidity
- Wind speed/direction
- Temperature of fuel flowing into/out of the tank
- Vehicle weight
- Tire pressure
- Status of auxiliary devices (A/C, Radio, Headlights, Windows)

The experimental design has two factors (pavement type and driving mode) and two levels for each factor (PCC versus AC and constant speed of 30 mph versus a 3 mph/sec acceleration mode). The experimental factors and levels are also shown in Table 1. The two levels and two factors are varied together, yielding four treatment combinations or responses as shown in Table 2. Table 3 shows the two-level, two-factor experimental design.

Table 1. The experimental factors and levels

Factor	Description	Level A	Level B
I	Pavement Type	PCC	AC
II	Driving Mode	Constant Speed*	Acceleration**

*Speed is kept constant at 30 mph during the data collection run.

**Data are collected during accelerating from zero to 30 mph at a rate of 3 mph/sec.

Table 2. The four factor-level combinations

Factor-Level Combination	Pavement Type	Driving Mode
1	PCC	Constant Speed
2	PCC	Acceleration
3	AC	Constant Speed
4	AC	Acceleration

Table 3. The two-level, two-factor design for the experiments

Run	Description
1	PCC, Constant Speed
2	PCC, Acceleration
3	AC, Constant Speed
4	AC, Acceleration

A number of observational repeat tests are necessary for each run of the above table in order to be able to draw statistically significant conclusions. It is expected that six observational repeat tests in each run will be sufficient to yield a 90% level of confidence with a ±10% error, although this also depends on data variance and tolerable level of error. If larger than expected variability is observed from one run to the next as experimental runs are made, the number of repeat tests may have to be increased.

RESULTS

The fuel consumption measurements for dry pavement condition were performed in November 2008. Four sets of factors measured were as follows:

- PCC, Constant Speed
- PCC, Acceleration
- AC, Constant Speed
- AC, Acceleration

The fuel flow rate in gallons per minute and the cumulative fuel consumed in each scenario were retrieved from the onboard data acquisition system. Two examples of the raw data plot are shown in Figure 3 for PCC at constant speed and in Figure 4 for PCC under the acceleration driving mode.

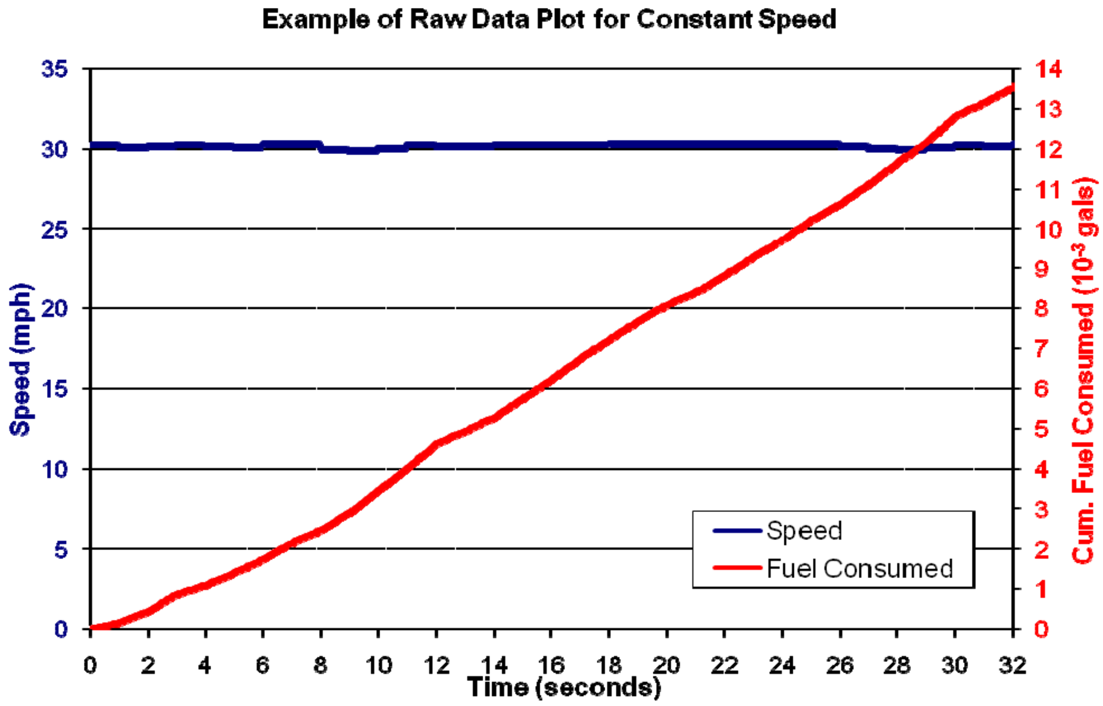


Figure 3. Example of raw data plot for PCC pavement under constant speed mode

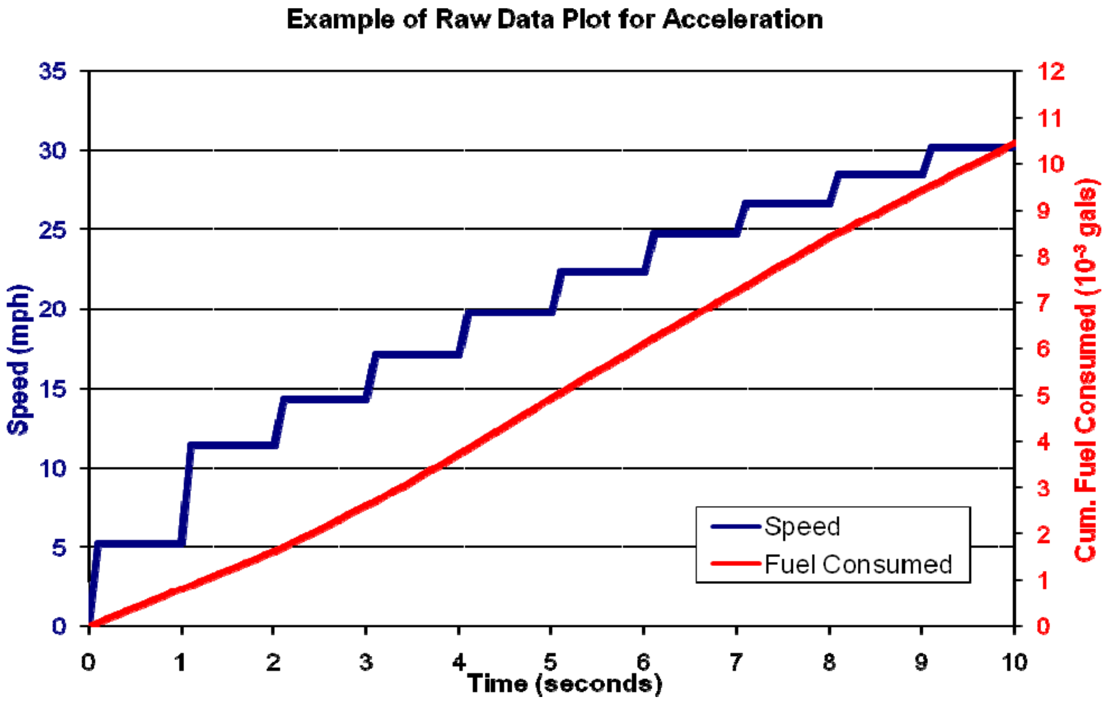


Figure 4. Example of raw data plot for PCC pavement under acceleration mode

Table 4 summarizes the average fuel consumption rate for each of the four dry pavement and driving mode runs. In the constant speed mode (Figure 3), a cruise speed of 30 mph was maintained throughout the test section. Under the acceleration driving mode (Figure 4), the fuel data were collected while accelerating from zero to 30 mph in 10 seconds, yielding an average acceleration rate of 3 mph/second. For each driving mode, the total fuel consumed was recorded, and the corresponding consumption rate in gallons per mile was computed.

Table 4. Average fuel consumption rates for PCC versus AC sections

	Average Fuel Consumption (10^{-3} gals/mile)	Test Conditions
PCC, Constant Speed	40.7	**Date:** November 7, 2008 **Temperature:** 69 °F **Pressure:** 30.08 in. Hg **Wind:** 7 mph W (tailwind) **Engine:** Warm **Tire Pressure:** 50 psi **Tank Level:** Full **IRI (in./mi):** 174.6 (PCC), 180.6 (AC) **Longitudinal Slope:** +1.2% (PCC), +1.2% (AC)
AC, Constant Speed	42.7	
PCC, Acceleration	236.4	
AC, Acceleration	236.9	

In both driving modes, the fuel consumption rate for the PCC pavement was observed to be lower than the rate for the AC pavement. These observed differences in fuel consumption rates were, however, tested for statistical significance at 10% level of significance. One-sided t-tests were conducted to probe whether the fuel rates on the PCC section were statistically lower than the rates on the AC section. It was determined that in the constant speed mode, the lower fuel consumption rate on the PCC section was in fact statistically significant at 10% level of significance. However, this was not the case under the acceleration mode. Therefore, it can be concluded that under constant travel speed of 30 mph, the PCC section results in statistically lower fuel consumption rate than the AC section. Under an acceleration of 3 mph/sec, while the PCC still resulted in a slightly lower consumption rate, the difference in consumption rate was found not to be statistically significant.

Under the constant speed scenario, the fuel rates were applied to the annual VMT in the Dallas-Fort Worth (DFW) region of Texas. The total annual VMT in the nine-county DFW region is estimated to be 62,697 million miles per year. The fuel consumption rates were applied to this VMT to obtain total annual fuel consumption estimates for a hypothetical mix of vehicles as shown in Table 5 (for PCC) and Table 6 (for AC). The CO_2 emissions in the PCC case were estimated using the following empirically derived regression model (Afotey 2008):

$$CO_2 \text{ amount in grams/sec} = 0.867 + 0.011\ V + 1.172\ a + 0.208\ aV,$$

where V is the velocity in mph and a is the acceleration rate in mph/second. The CO_2 emissions for all other cases were estimated as a ratio of the fuel consumption rate for each respective case relative to the field-measured rate for the PCC section.

Table 5. Calculations of annual fuel consumption and CO_2 emissions for the DFW region of Texas under dry PCC pavement and constant speed mode

Average Vehicle Mass (lbs)	% in the mix	VMT (million miles/yr)	Fuel Rate (gals/mi)	Fuel Consumed (million gals/yr)	CO_2 Rate (grams/mi)	Total CO_2 (million metric tons/yr)
3,000	35	21,944	0.0407*	893.1	143.64	3.15
4,000	33	20,690	0.0543	1,122.8	191.52	3.96
5,000	14	8,778	0.0678	595.4	239.40	2.10
6,000	10	6,270	0.0814	510.4	287.28	1.80
7,000	8	5,016	0.0950	476.3	335.16	1.68
∑	100	62,697		3,598.0		12.70

*Measured in the field

Table 6. Calculations of annual fuel consumption and CO_2 emissions for the DFW region of Texas under dry AC pavement and constant speed mode

Average Vehicle Mass (lbs)	% in the mix	VMT (million miles/yr)	Fuel Rate (gals/mi)	Fuel Consumed (million gals/yr)	CO_2 Rate (grams/mi)	Total CO_2 (million metric tons/yr)
3,000	35	21,944	0.0427*	937.0	143.64	3.31
4,000	33	20,690	0.0569	1,178.0	191.52	4.16
5,000	14	8,778	0.0712	624.7	239.40	2.20
6,000	10	6,270	0.0854	535.4	287.28	1.89
7,000	8	5,016	0.0996	499.7	335.16	1.76
∑	100	62,697		3,774.8		13.32

*Measured in the field

The field-measured fuel rates under the constant speed mode in Tables 5 and 6 represented a 3,000 lb vehicle (the instrumented test vehicle). For the purpose of calculations summarized in these tables, fuel consumption rates for all other vehicle classes were estimated from the field-measured rate based on the mass ratio of the two respective classes. For example, a 6,000 lb vehicle was estimated to have twice as large a fuel consumption rate than the 3,000 lb test vehicle. The total fuel consumption amounts per annum then were estimated using those rates and the total vehicle miles of travel for each vehicle class.

The overall results for the constant speed mode are summarized in Table 7. In Table 7, if the annual vehicle miles of travel in the DFW region took place at a constant speed of 30 mph on PCC pavements, the statistically lower fuel rate could result in an annual fuel savings of 177 million gallons and an annual CO_2 reduction of about 0.62 million metric tons. Assuming an average fuel cost of about $2/gallon and an average CO_2 cleanup cost of about $18/metric (Eco Business Links 2009), these differences would amount to a fuel and cleanup cost savings in the DFW region of approximately $354 million and $11 million per year, respectively. These cost savings should possibly be considered in the LCCA of alternative city street pavement projects.

Table 7. Total annual fuel consumption and CO_2 emissions for the DFW region of Texas under each pavement type

	Fuel Consumed (million gals/yr)	Total CO_2 (million metric tons/yr)
PCC, Constant Speed (30 mph)	3,598	12.70
AC, Constant Speed (30 mph)	3,775	13.32

CONCLUSIONS

Under dry surface conditions at urban driving speed of 30 mph, fuel consumption per unit distance is lower on concrete pavement than on asphalt pavement. Moreover, the observed difference in fuel consumption rate is statistically significant at 10% level of significance with the rate for the concrete pavement being lower. The potential savings in fuel consumed and CO_2 emissions over the life of the project could be substantial and should be considered in the life cycle cost analysis of alternative projects and in sustainable development considerations such as the carbon footprint of a roadway project.

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Spatial Analysis of Crash Location Relative to Population Change

Jielin Sun
Department of Urban and Regional Planning and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
jielin-sun@uiowa.edu

Lei Sun
Department of Urban and Regional Planning and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
lei-sun@uiowa.edu

Adwetta Joshi
Department of Urban and Regional Planning and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
jielin-sun@uiowa.edu

Paul Hanley
Department of Civil and Environmental Engineering and the Public Policy Center
University of Iowa
209 South Quadrangle
Iowa City, IA 52242
paul-hanley@uiowa.edu

ABSTRACT

In this paper, we evaluate the extent of the spatial evolving relationship between vehicle crash locations and population characteristics. Using Iowa crash records from 1990 through 2008 and the U.S. Decennial Census of Population and Housing and American Community Survey for the same period, we test a hypothesis that the location and change in crash severity are responsive to population changes.

We will examine the hypothesis with three spatial analysis approaches. The first approach fits spatial regression models using the spatial taxonomy defined by urban, urban fringe, and rural. The analysis corrects for autocorrelation of the crashes to major roads by employing Moran’s Index and controls for population and crash characteristics. The second approach defines and calibrates two kernel density models for population distribution and severe traffic crash distribution over the study period. The third method extends the concept of kernel density to include constraints created by the underlying network. This method identifies linear clusters along roadways and correlations between location and population. Crash characteristics are also obtained.

Our analysis shows that the location and changes in crash severity are responsive to population changes. Additionally, the spatial distribution of the effects of road safety is correlated with sub-population characteristics. Our analysis shows that (1) growing areas have more locations with decreasing crash rates, (2) younger areas experience more benefits in the form of reduced crashes, and (3) the magnitude of crash rate reductions is significantly lower in areas that have a population composed of a majority of elderly residents.

Key words: population characteristics—spatial analysis—vehicle crash locations

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Antioxidant Effect of Bio-Oil Additive ESP on Asphalt Binder

Sheng Tang
Department of Civil, Construction, and Environmental Engineering
Iowa State University
174 Town Engineering
Ames, IA 50011
tsheng@iastate.edu

R. Christopher Williams
Department of Civil, Construction, and Environmental Engineering
Iowa State University
482 Town Engineering
Ames, IA 50011
rwilliam@iastate.edu

ABSTRACT

Bio-oil is a dark brown, mobile liquid derived from the thermo-chemical processing of bio-mass. Bio-oils generally contain water and lignin. Lignins are highly-available, well-studied carbohydrate derivative known for their antioxidant properties. For asphalt pavements, oxidation can cause deterioration, a long-term aging process, and eventually result in cracking. Therefore, bio-oil could potentially serve as an antioxidant additive in asphalt mixtures. The main objective of this study is to evaluate the effects of lignin-containing bio-oil for utilization in asphalt binder. Using bio-oil as an antioxidant in asphalt production could prove to be an economical alternative to conventional methods while being conscious of the environment and increasing the longevity and performance of asphalt pavements.

Three bio-oils derived from corn stover, oak wood, and switch grass are tested and evaluated by blending with three conventional asphalt binders. The binders in order of their susceptibility to oxidative aging, include two binders from the Federal Highway Administration's (FHWA's) Materials Reference Library (MRL), AAM-1 and AAD-1, as well as a locally produced polymer modified asphalt binder (LPMB). Bio-oil was added to the asphalt binders in three different percentages by weight, 3%, 6%, and 9%. The Superpave testing and performance grading procedure from AASHTO M 320 was used to examine the antioxidant effects and determine the optimum fraction of bio-oil added to the binders. In addition, performance tests for an asphalt mixture containing the bio-oil modified asphalts were conducted. The experimental asphalt samples for dynamic modulus testing were mixed by adding optimum percentages of bio-oil modified asphalt in the aggregate with a common gradation. Statistical methods are applied and used to determine the statistically significant bio-oil treatment effects.

Generally, the corn stover, oak wood, and switch grass derived bio-oil indicate that there is potential to increase the high temperature performance of asphalt binders. However, the increase in high temperature performance adversely affects the low temperature binder properties. The overall performance grade ranges vary depending on the combinations of three different binders and bio-oils. According to the data, some binders show antioxidant effects. Interestingly, the dynamic modulus test results do not necessarily coincide with the asphalt binder test results and suggest greater mix performance improvement than identified by the binder test results.

Key words: antioxidant additive—asphalt—bio-oil

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INTRODUCTION

Bio-oil is a dark brown, mobile liquid derived from the thermo-chemical processing of bio-mass. This liquid can be directly utilized as renewable fuel or taken as a source of valuable chemicals (A.V. Bridgewater 1999). The thermo-chemical process is known as pyrolysis, which can be divided into traditional and fast pyrolysis. Bio-oil mainly contains carbohydrate derivatives, water, and lignin, which is a highly available, well-studied antioxidant.

The purpose of the work was to evaluate the outcome of adding a small amount of electrostatic precipitant (ESP) bio-oil derived from corn stover, oak wood, and switch grass on the rheological properties of three conventional asphalt binders, as well as to detect the effect of applying the bio-oil modified binder to hot mix asphalt (HMA) mixtures.

MATERIALS AND EXPERIMENTAL

Materials

Three asphalt binders were chosen for this study: two binders from the Federal Highway Administration's (FHWA's) Materials Reference Library (MRL), AAM-1 and AAD-1, as well as a locally produced polymer modified asphalt binder (LPMB). AAM-1 is a PG 64-16 West Texas Asphalt, which is less susceptible to oxidative aging. AAD-1 is a PG 58-22 California Coastal asphalt, which is more susceptible to oxidative aging comparing with AAM-1 (Mortazavi and Moulthrop 1993). LPMB is a styrene-butadiene-styrene (SBS) polymer-modified PG 58-22 binder. The chemical compositions of the two MRL binders are shown in Table 1.

Table 1. Chemical contrast of AAD-1 and AAM-1 (Mortazavi and Moulthrop 1993)

Component Composition	AAD-1	AAM-1	Elemental Composition	AAD-1	AAM-1
Asphaltenes	23.9	9.4	Carbon	81.6	86.8
Polar aromatics	41.3	50.3	Hydrogen	10.8	11.2
Napthene aromatics	25.1	41.9	Oxygen	0.9	0.5
Saturates	8.6	1.9	Sulfur	6.9	1.2

Three experimental bio-oil fractions used for this study are derived from corn stover, oak wood, and switch biomasses. All of the bio-oils were obtained from the same source, Iowa State University Center for Sustainable Environmental Technologies (CSET). CSET operates a pilot-scale biomass conversion system in the Biomass Energy Conversion Facility (BECON), located in Nevada, Iowa. Bio-oil samples were obtained from different condensers in the bio-oil pilot plant, but only the ESP bio-oils were tested in this study. This is due to the ESP bio-oil having higher lignin concentration and low water content as compared to other bio-oil derived fractions. Table 2 illustrates the characteristics of bio-oil from the different fractions.

Table 2. Characteristics of bio-oil from different condensers

Property	Cond. 1	Cond. 2	Cond. 3	Cond. 4	ESP
Fraction of total oil (wt%)	6	22	37	15	20
pH	-	3.5	2.7	2.5	3.3
Viscosity @40oC (cSt)	Solid	149	2.2	2.6	543
Lignin Content (wt%)	High	32	5.0	2.6	50
Water Content (wt%)	Low	9.3	46	46	3.3
C/H/O Molar Ratio	1/1.2/ 0.5	1/ 1.6/ 0.6	1/ 2.5 / 2	1/ 2.5 /1.5	1/1.5/ 0.5

Among the three ESP bio-oil fractions, the lignin content ranges varied too. Oak wood has the highest lignin fraction by weight. The second highest lignin containing sample is the corn stover bio-oil. The fractionated characteristics of the three ESP bio-oils are shown in Table 3.

Table 3. Characteristics of bio-oil

Property	Corn Stover	Oak Wood	Switch Grass
Lignin (wt%)	82.3	83.9	81.0
Moisture (wt%)	16.8	15.4	17.8
Solid (wt%)	0.6	0.6	1.1
Ash (wt%)	0.3	0.1	0.2

Preparation of ESP Bio-oil Modified Asphalt

A high speed shear mixer was used to prepare the ESP bio-oil modified asphalt. The bio-oil and asphalt binder mixing was conducted at 155°C at 5000 rotations per minute shearing speed for one hour. Table 4 illustrates the mixing treatment group combinations with the 0% bio-oil binders (the initial binders) being the experimental control groups.

Table 4. Treatment group combinations

Bio-oil Types	AAD-1	AAM-1	LPMB
Corn Stover	0,3,6,9 (wt%)	0,3,6,9 (wt%)	0,3,6,9 (wt%)
Oak Wood	3,6,9 (wt%)	3,6,9 (wt%)	3,6,9 (wt%)
Switch Grass	3,6,9 (wt%)	3,6,9 (wt%)	3,6,9 (wt%)

Standard Aging Procedure

The rolling thin film oven (RTFO) was used to stimulate the asphalt short-term aging as described in ASTM D2872. The long-term binder aging was addressed by the pressurized aging vessel test (PAV, ASTM D 6521) by using the residue from the RTFO test. The short-term aging is described as the aging during the hot-mix asphalt (HMA) production and construction. The long-term aging represents approximately 12 years of aging in the field.

Test Methods

A dynamic shear rheometer (DSR) was used to test three replicate samples for each binder/bio-oil combination according to ASTM D 7175 (2005), which was used to characterize rheological properties of the binders at high and intermediate temperatures. The complex modulus (G*) and phase angle (δ) were determined with a DSR for the initial binder and residual binder after every asphalt aging treatment (RTFO, PAV). The complex modulus (G*) and phase angle (δ) later were used to find the high and intermediate critical temperatures and the performance grades (PG) ranges.

A bending beam rheometer (BBR) was applied to evaluate the treatment group's susceptibility to thermal cracking at low service temperatures (Roberts et al. 1996, The Asphalt Institute 2003). Two key properties, stiffness (S) and change in stiffness (m-value) were recorded according to ASTM 6648 (2001). The BBR test was used to determine the low critical temperatures.

All of the 30 bio-oils and binder combinations underwent the asphalt performance test according to AASHTO M320 for grading an asphalt binder. The grading procedures are shown in Figure 1.

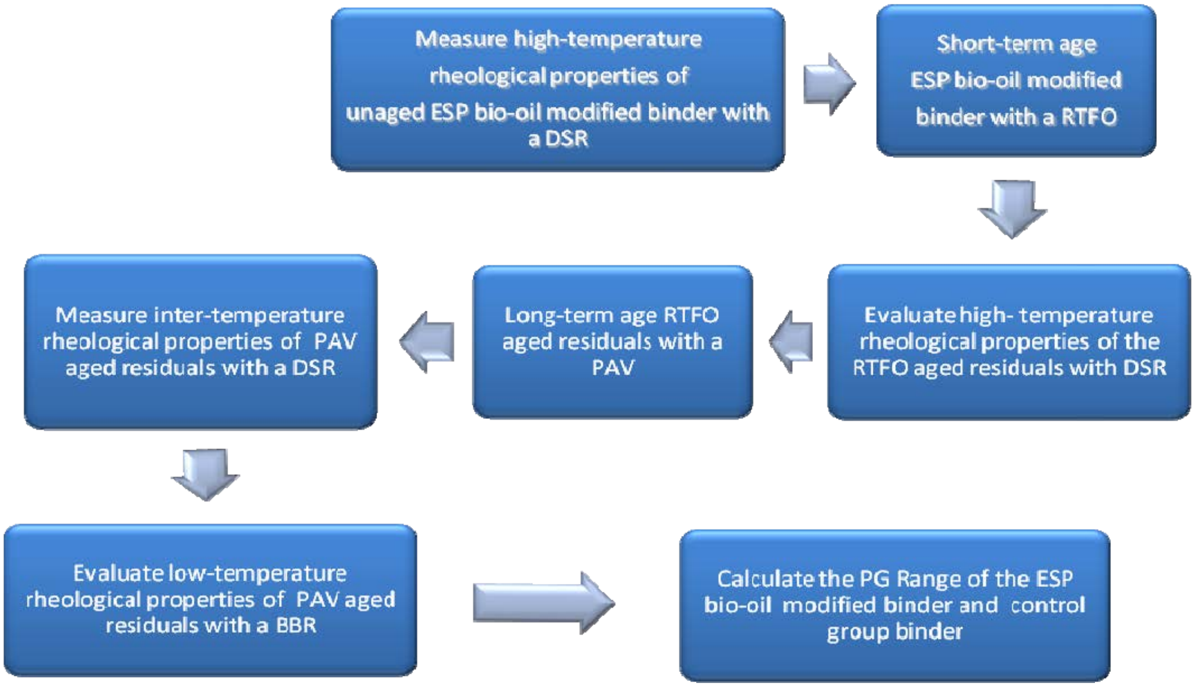
Measure high-temperature rheological properties of unaged ESP bio-oil modified binder with a DSR
Short-term age ESP bio-oil modified binder with a RTFO
Evaluate high- temperature rheological properties of the RTFO aged residuals with DSR
Long-term age RTFO aged residuals with a PAV
Measure inter-temperature rheological properties of PAV aged residuals with a DSR
Evaluate low-temperature rheological properties of PAV aged residuals with a BBR
Calculate the PG Range of the ESP bio-oil modified binder and control group binder

Figure 1. Experiment grading procedures

Dynamic modulus testing of the asphalt mixtures (ASTM D3497-79) were used to detect the effect of applying the bio-oil modified binders to the HMA mixtures. The test was conducted at three temperatures (4°C, 21°C, and 37°C) and at nine frequencies ranging from 0.01 Hz to 25 Hz. The 21°C dynamic modulus-master curve was created from the five replicate test samples. All test specimens were compacted to 7% ± 1% air voids at the same 5.6% optimum binder content.

RESULTS AND DISCUSSION

Performance Grade Test

The average critical temperatures (high, intermediate, and low), as well as the performance grade range (binder service range) for the treatment combinations are listed in Table 5. The combinations that resulted in an increase in the PG range (RTFO Aged Tc–Low Tc) are shown in bold italics.

Table 5. Critical temperature grade

			Average Critical Temperatures (^{o}C)				Grade Range (^{o}C)	
Asphalt ID	**Co-product (%)**	**Co-product (type)**	**AverageUnaged High T_c**	**AverageRTFO Aged High T_c**	**AveragePAV aged Int T_c**	**AveragePAV Aged Low T_c**	**Unaged High Tc - Low Tc**	**RTFO Aged Tc - Low Tc**
AAD-1	9	Corn Stover ESP	58.91	66.04	19.04	-21.77	80.68	87.81
	9	Oakwood ESP	60.23	72.49	22.63	-17.00	77.23	89.49
	9	Switchgrass ESP	55.78	64.12	18.44	-21.52	77.30	85.65
	6	Corn Stover ESP	60.26	65.80	18.90	-20.06	80.32	85.85
	6	Oakwood ESP	59.80	68.12	19.76	-18.60	78.40	86.72
	6	Switchgrass ESP	57.77	64.06	17.56	-21.84	79.60	85.90
	3	Corn Stover ESP	62.16	66.23	18.45	-21.53	83.69	87.76
	3	Oakwood ESP	61.15	67.01	20.48	-19.39	80.54	86.40
	3	Switchgrass ESP	59.36	64.27	17.14	-24.54	83.90	88.81
	0	Initial binder	62.27	65.38	17.33	-25.18	87.46	90.56
AAM-1	9	Corn Stover ESP	64.96	68.16	19.86	-11.97	76.94	80.13
	9	**Oakwood ESP**	**65.37**	**69.83**	**20.56**	**-13.04**	**78.40**	**82.87**
	9	**Switchgrass ESP**	**62.24**	**67.54**	**19.47**	**-13.98**	**76.22**	**81.52**
	6	Corn Stover ESP	65.26	67.69	19.69	-12.08	77.34	79.77
	6	**Oakwood ESP**	**65.92**	**69.20**	**20.46**	**-12.76**	**78.69**	**81.96**
	6	**Switchgrass ESP**	**63.15**	**66.91**	**19.61**	**-15.39**	**78.53**	**82.30**
	3	Corn Stover ESP	65.56	67.22	19.15	-12.60	78.16	79.82
	3	Oakwood ESP	66.96	67.44	20.25	-12.66	79.62	80.10
	3	Switchgrass ESP	64.27	66.57	19.32	-15.05	79.32	81.61
	0	Initial binder	67.77	66.68	20.26	-15.06	82.83	81.75
LPMB	9	Corn Stover ESP	62.73	68.91	20.31	-17.66	80.39	86.58
	9	Oakwood ESP	63.59	71.07	21.26	-16.66	80.25	87.73
	9	**Switchgrass ESP**	**61.87**	**68.19**	**19.70**	**-22.28**	**84.14**	**90.47**
	6	Corn Stover ESP	64.25	68.29	19.48	-18.04	82.28	86.33
	6	**Oakwood ESP**	**65.36**	**69.74**	**20.44**	**-19.24**	**84.60**	**88.98**
	6	**Switchgrass ESP**	**63.12**	**69.68**	**20.16**	**-20.71**	**83.83**	**90.39**
	3	Corn Stover ESP	65.58	68.97	19.19	-19.47	85.05	88.44
	3	**Oakwood ESP**	**65.46**	**69.52**	**19.26**	**-20.18**	**85.64**	**89.70**
	3	**Switchgrass ESP**	**65.33**	**68.70**	**18.34**	**-21.91**	**87.24**	**90.61**
	0	Initial binder	65.97	66.74	20.09	-22.01	87.98	88.75

A distinct increase of the high critical temperature grade was observed for every bio-oil modified binder after the RTFO aging application except switch grass ESP bio-oil modified AAD-1 binder as shown in Figures 2, 3, and 4. The larger high critical temperature grade for bio-oil modified binder than the initial ones indicates the extra-hardening of the asphalt during the construction aging processes. This could imply the HMA mixtures are less susceptible to rutting after field construction, due to the stiffer asphalt binder and the higher dynamic modulus of the HMA mixtures (Xiang Shu, Baoshan Huang 2007).

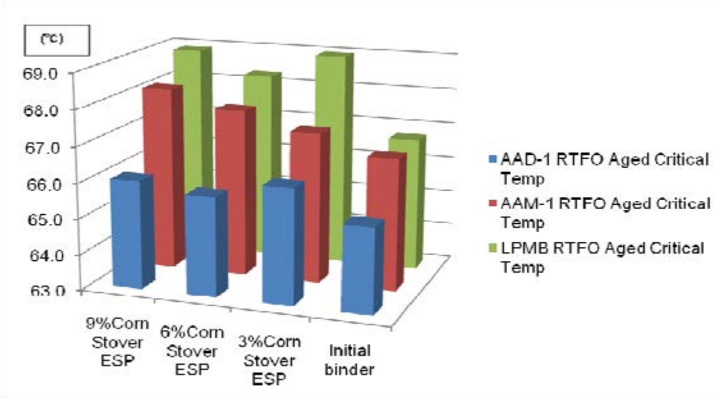

Figure 2. Corn stover ESP bio-oil modified binder's RTFO aged high critical temperatures

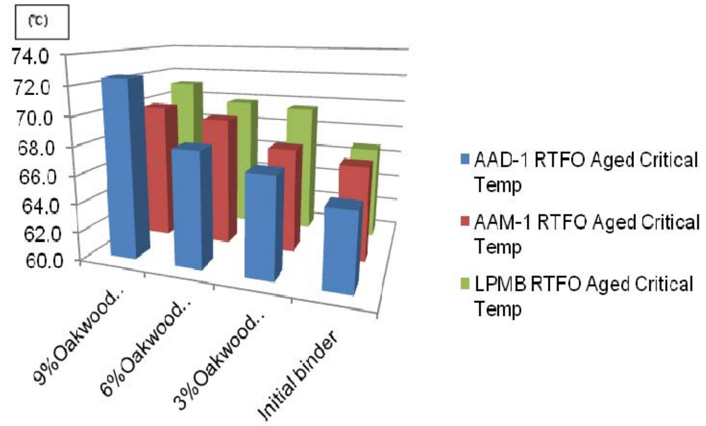

Figure 3. Oak wood ESP bio-oil modified binder's RTFO aged high critical temperatures

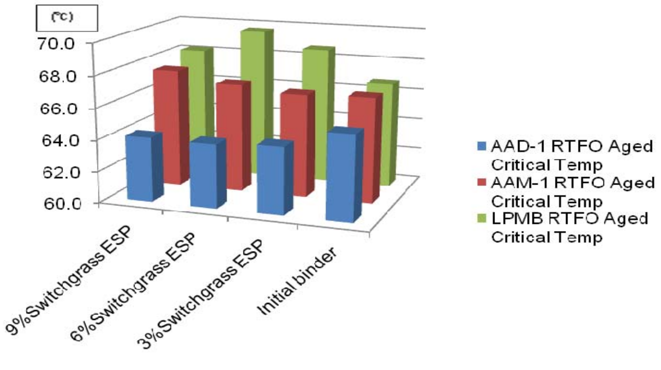

Figure 4. Switch grass ESP bio-oil modified binder's RTFO aged high critical temperatures

The trend of the intermediate critical temperature grade was not prominent for PAV aged residuals. Long-term aging effects were more complex than the short-term aging, and the results varied with the binder and bio-oil types. As shown in Figures 5, 6, and 7, an increment of intermediate critical temperature grade was detected for every bio-oil modified AAD-1 binder, and results in fatigue resistant benefit. But the intermediate critical temperature for bio-oil modified binders AAM-1 slightly decreased, since AAM-1 was less susceptible to oxidative aging (Mortazavi and Moulthrop 1993). The situation for bio-oil modified LPMB binder was complex, likely due to the LPMB's SBS modification. The rheological properties for polymer-modified asphalt during the aging processes are dependent upon the structural characteristics of the incorporated polymer (M.S Cortizo et al. 2004). However, the size exclusion chromatography technique can be applied to analyze the molecular size of the polymer modified asphalt to explain the rheological phenomena for a future study.

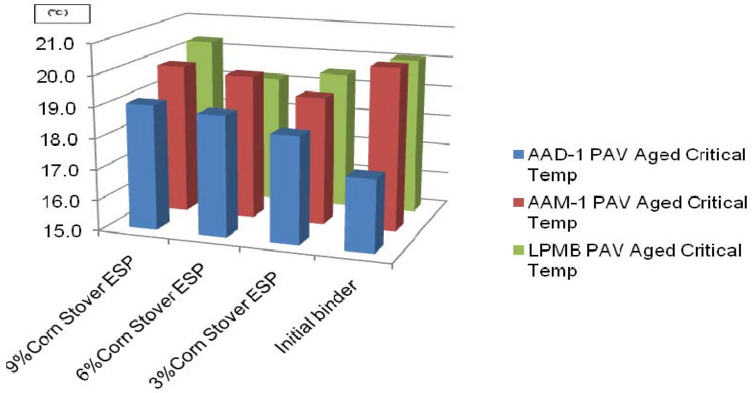

Figure 5. Corn stover ESP bio-oil modified binder's PAV aged intermediate critical temperatures

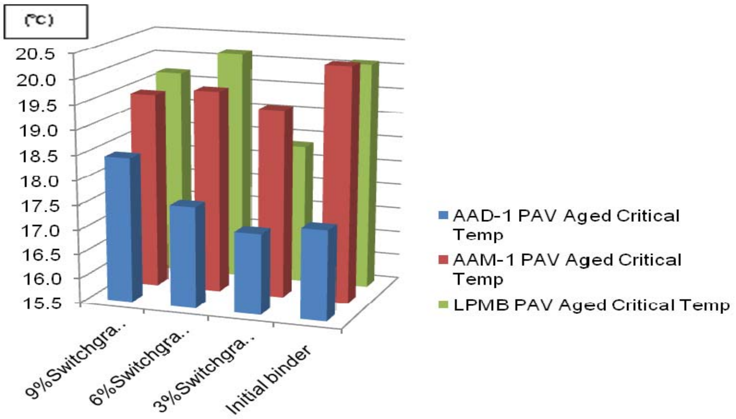

Figure 6. Switch grass ESP bio-oil modified binder's PAV aged intermediate critical temperatures

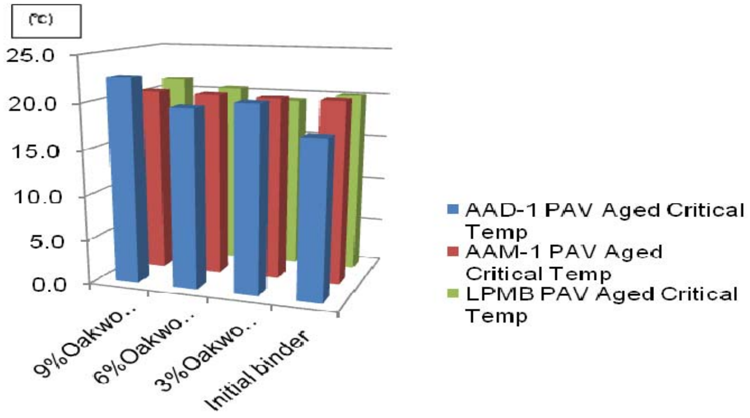

Figure 7. Oak wood ESP bio-oil modified binder's PAV aged intermediate critical temperatures

For most of the oak wood and corn stover bio-oil treatment combinations, the low critical temperature grades were decreased in different amounts by the variation of the binders and the fraction of the added bio-oil. The order in range decrease scales was AAM-1 < LPMB < AAD-1. The stiffening effect benefits the high temperature properties but could introduce some adverse effect to the low temperature performance. Mainly, the more bio-oil added, the greater the increase in the critical low temperature. However, the switch grass bio-oil added in all three binders did not adversely affect the rheological properties at low temperature as much as the other two bio-oils, especially for the LPMB binder. The 3%, 6%, and 9% switch grass modified LPMB; 3% and 6% switch grass modified AAM-1; and 3% switch grass modified AAD-1 were detected to maintain similar critical low temperatures as the initial binder without a statistically significant difference. The statistical analysis consisted of student t-test at an alpha level of 0.05.

The overall effects of the bio-oil additive were estimated by the performance grade (PG) range (RTFO critical high temp – BBR critical low temp). Table 6 presents a summary of bio-oil treatment combinations demonstrating positive or neutral effect on the PG grade. A student-t least significant difference (LSD) with a type 1 error of 0.05 was used to determine the statistically significant difference between average mean values of performance grades.

Table 6. Summary of bio-oil treatment combination effect

Binder	Positive Effect	ΔT°C	Neutral Effect
AAD-1	None		Possible 9% Oak wood
AAM-1	9% Oak wood	+1.13	3% Switch grass 6% Switch grass 9% Switch grass 6% Oak wood
LPMB	3% Switch grass 6% Switch grass 9% Switch grass 3% Oak wood	+1.86 +1.63 +1.71 +0.95	6% Oak wood 9% Oak wood 3% Corn stover

For the AAD-1 binder, no combination of bio-oil showed an extension of the performance grade. Even adding oak wood bio-oil yielded a relevant increase in the critical high temperature, the greater drop in critical low temperature mitigated the benefit of the asphalt hardening by aging. The high temperature improvement is not sufficient to redeem the low temperature deterioration.

For AAM-1 binder, there are still no bio-oil combinations that can increase the performance range. The switch grass bio-oil slightly increases the critical high temperature, and nearly maintains the critical low temperature range, which is detected as a statistically insignificant increase in the performance grade.

The bio-oil can widely be applied to the SBS modified binder, LPMB. Switch grass and oak wood are suitable for the bio-oil modification.

Dynamic Modulus Test

To evaluate the HMA mixture using bio-oil modified binder, the 9% oak wood bio-oil modified binder, which is commonly beneficial or neutral for every binder, was selected to mix with aggregate for dynamic modulus testing. Figures 8, 9, and 10 present the 21ºC dynamic modulus master curve for applying the 9% oak wood bio-oil modified binders (AAD-1, AAM-1, and LPMB).

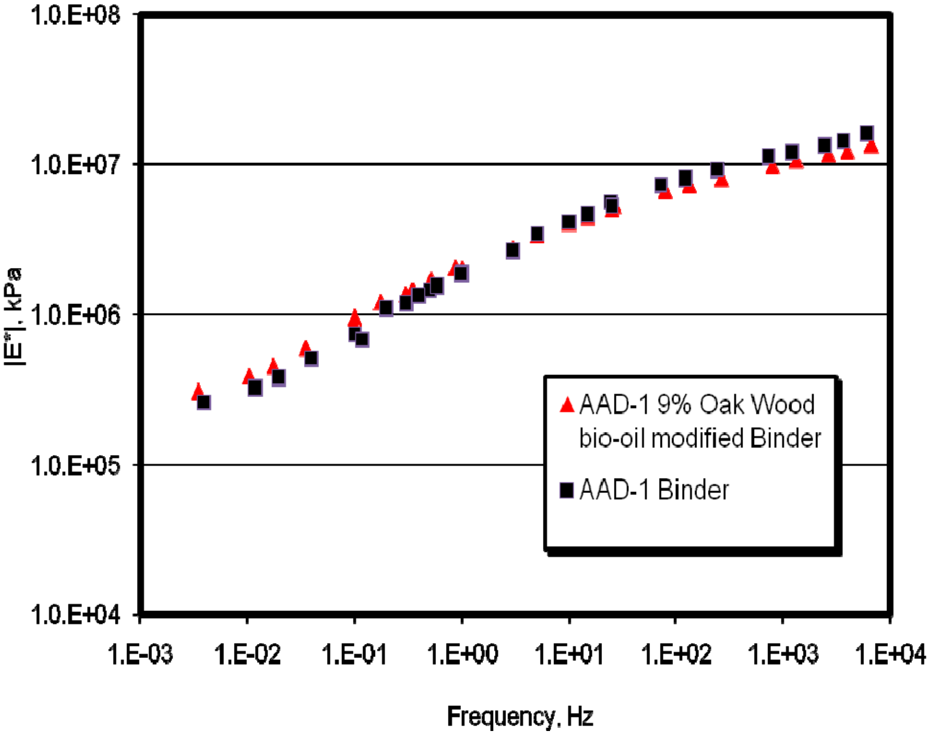

Figure 8. AAD-1 master curve

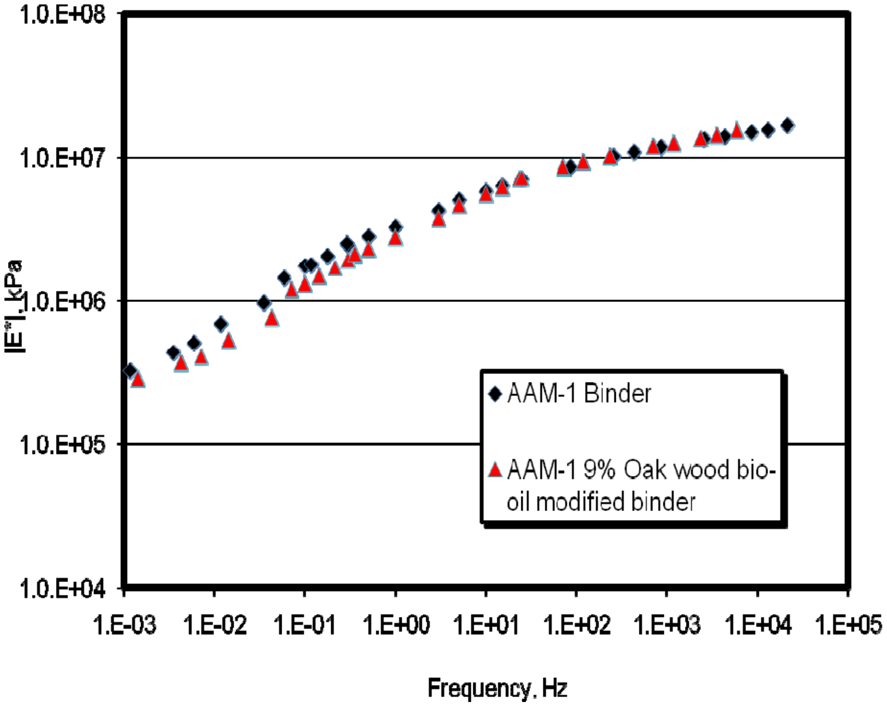

Figure 9. AAM-1 master curve

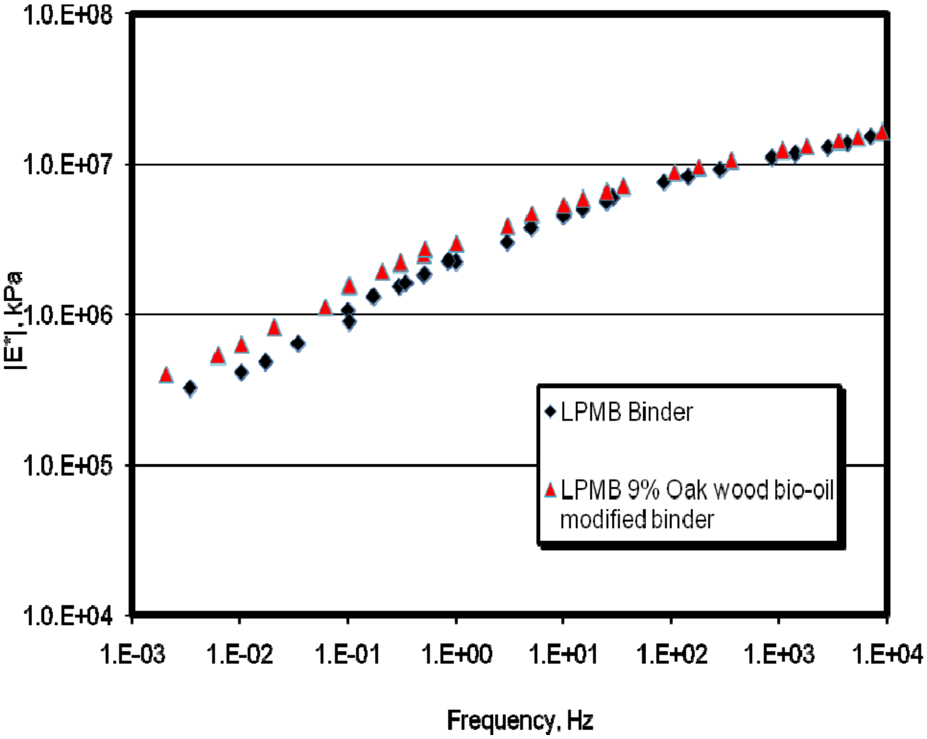

Figure 10. LPMB master curve

The differences for the E* values between initial binder and the bio-oil modified binder can be seen from the master curves. In Table 5, 9% oak wood treatment improved the RTFO critical high temperatures by 7.1°C for AAD-1, by 3.0°C for AAM-1, and by 4.9°C for LPMB. Based on the sensitivity analysis, using stiff asphalt binders is an effective way to increase the dynamic modulus values (Xiang Shu, Baoshan Huang 2007). This statement proved to be true for the AAD-1 and LPMB, but the opposite observation was generated for the AAM-1 binder mixture. The stiffer binder AAM-1 modified by 9% oak wood has a relatively smaller modulus than the initial AAM-1 binder. One possible explanation is that the 3oC increment in critical temperature is insufficient. It is clear that bio-oil modified LPMB mixtures have advantages in low frequency or high temperature environments. Additionally, the bio-oil modified ADD-1 mixture seems to have potential benefits for resisting fatigue at intermediate temperatures, and the thermal cracking at low temperatures or high frequency ambience, since the E* for bio-oil modified AAD-1 is smaller than that of the initial one (Qunshan Ye et al. 2009).

CONCLUSIONS AND RECOMMENDATIONS

Research Findings

Mainly, findings include that the bio-oil modified binders are stiffer than the initial binders when subjected to the aging process, and this shows benefits for rutting resistance. On the other hand, the stiffer bio-oil modified asphalt usually sacrifices some ability to prevent thermal cracking. The overall performance grade ranges vary depending on the combinations of three different binders and bio-oils. Mix results, however, do not agree with the binder test results.

Future research will examine other bio-oils (switch grass and corn stover) in the same asphalt binders. Also, the use of a tall oil to enhance the low temperature properties of the binders and examine their effect on mix testing will be done. Additional low temperature mix fracture testing such as the semi-circular bend test will be conducted.

ACKNOWLEDGMENTS

The authors would like to thank the Iowa Energy Center for their generous support and Bill Haman for his technical support.

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Common Sense Sustainability for Concrete Pavements

Peter Taylor
National Concrete Pavement Technology Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
ptaylor@iastate.edu

ABSTRACT

This is a presentation on the progress being made on the CP Roadmap (the nation's strategic plan for concrete pavement research) with an emphasis on the goals, products, and status of activity on the Sustainability Track.

This presentation is part of the Sustainable Concrete Pavement Technologies session.

Key words: concrete—CP Roadmap—economic—sustainability

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Remembering Tom Maze

In 1995, to mark the 75th anniversary of the Highway Research Board (today's Transportation Research Board), Dr. Maze and the Iowa DOT's Ian MacGillivray planned the first biennial conference that we now call the Mid-Continent Transportation Research Symposium.

Tom Maze, professor of civil engineering at Iowa State University, died of heart failure on June 8, 2009. His presence, and of course his contributions to the transportation and education communities, are greatly missed in Iowa and around the nation.

You can read reminiscences from many of Tom's friends and colleagues online: www.intrans.iastate.edu/news/2009/mazeremembered.htm. The simple stories, ranging from playful to profound, capture the unique personality behind Tom's professional biography. We hope you'll take a minute to share your own memories via the online form.

His mark at Iowa State University

Dr. Maze began his engineering career at Iowa State University, earning a B.S. in civil engineering from ISU in 1975. He went on to receive a master's degree from the University of California, Berkeley, in 1977 and a PhD from Michigan State University in 1982. He joined the faculty in the School of Civil Engineering and Environmental Science at the University of Oklahoma in Norman, where he was director of the Oklahoma Highway and Transportation Engineering Center.

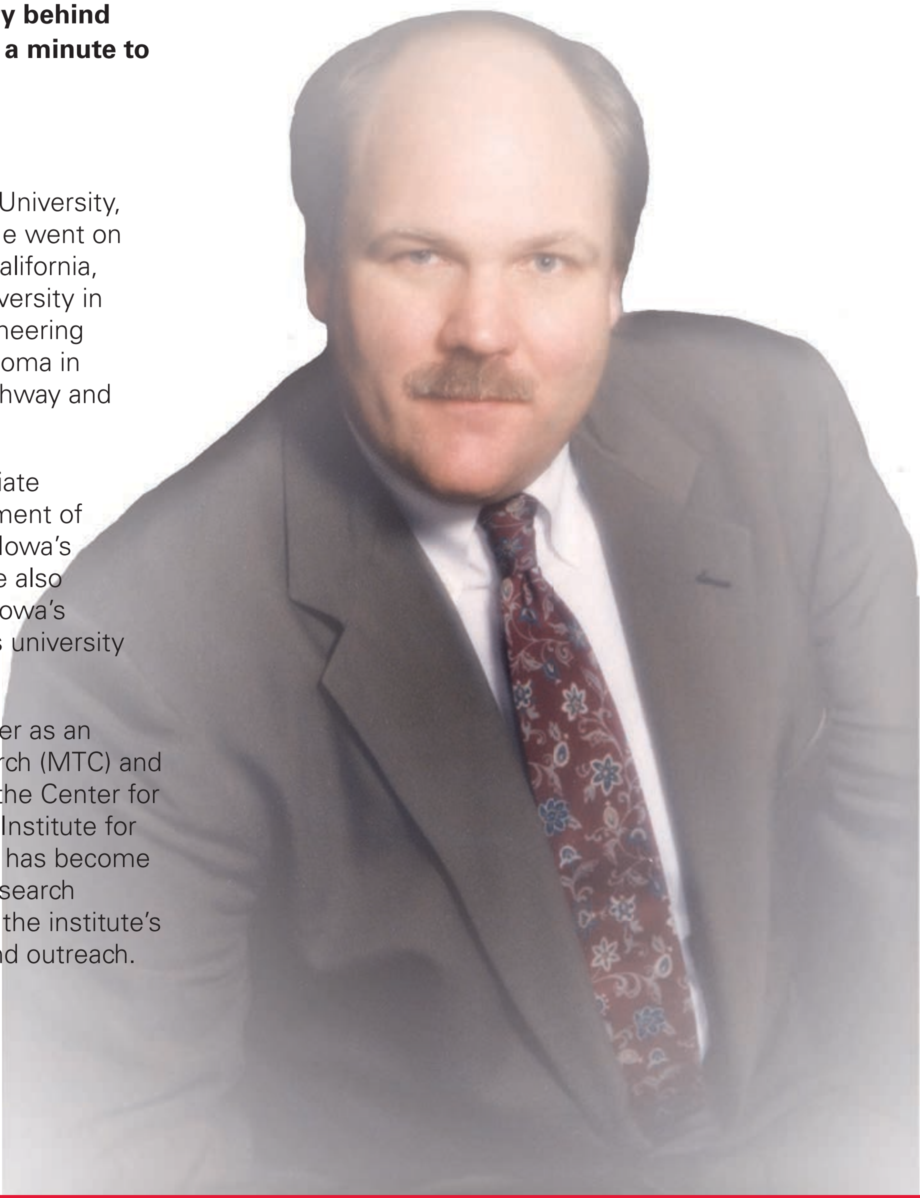
In 1988, Dr. Maze returned to Iowa State as an associate professor in the transportation division of the Department of Civil and Construction Engineering and as director of Iowa's Local Technical Assistance Program (LTAP). In 1990 he also assumed co-directorship of Iowa State/University of Iowa's Midwest Transportation Center (MTC) (the U.S. DOT's university transportation research program for region 7).

At this point he initiated the Iowa Transportation Center as an umbrella organization for transportation-related research (MTC) and outreach (LTAP) at Iowa State. Through the ITC (later the Center for Transportation Research and Education, and now the Institute for Transportation), Dr. Maze grew a robust program that has become one of the leading university transportation-related research programs in the United States. One of his legacies is the institute's solid reputation for research, academic excellence, and outreach.

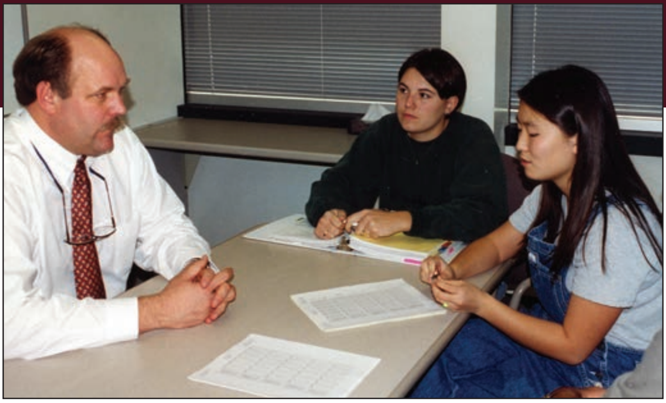
Memorial scholarship

An academic scholarship is being established in Dr. Maze's name. You can contribute online (www.foundation.iastate.edu/maze) or write a check payable to Iowa State University Foundation (with the memo "Maze Scholarship Fund") and send it to the following address:

Chris Knight
Civil, Construction, and
Environmental Engineering
394 Town Engineering
Iowa State University
Ames, IA 50011



IOWA STATE UNIVERSITY
Institute for Transportation



A visionary

Dr. Maze's vision regarding advanced transportation technologies led to demonstrations and innovations in intelligent vehicle systems for commercial vehicles, transportation-related applications of geographic information systems, and traffic and safety engineering innovations. He also led significant efforts in transportation planning and in statewide management systems for transportation infrastructure that have been models for other states. In recent years, he has focused on weather-related issues through a final program he initiated in 2003, the Center for Weather Impacts on Mobility and Safety (CWIMS).

In 1996, on behalf of Iowa State University, Dr. Maze helped forge a progressive transportation research management agreement between the university and the Iowa Department of Transportation. This efficient administrative tool continues to facilitate the center's quick response to identified transportation-related research and technology needs in Iowa and reflects a level of university-agency partnering that is the envy of many states.

Teacher and mentor

Dr. Maze taught more than 70 courses covering some 30 topics. He was a demanding but generous academic mentor to graduate students at Iowa State, nine of whom were doctoral candidates in civil engineering. He also took a keen interest in nurturing new faculty members and guided them in developing successful careers. He developed a unique, graduate-level academic enrichment and learning community program at Iowa State called Transportation Scholars. The heart of the Scholars program is a semester-long series of multidisciplinary seminars that Iowa State shares with other universities through distance technologies. To date, more than 200 Scholars have received stipends to work on research problems at Iowa State, met national experts in the transportation community, presented papers for student peers and local professionals, and participated in special events like the annual meeting of the Transportation Research Board.

Mid-Continent Transportation Research Symposium

Dr. Maze was intensely involved in enhancing transportation technology transfer through professional training and outreach activities. In partnership with the Iowa DOT, he initiated the biennial Mid-Continent Transportation Research Symposium in 1996. This popular event provides a professional, Midwest venue for disseminating national research in a format similar to the Transportation Research Board's annual meeting.

Through ongoing activities like the symposium, as well as the extensive body of work he leaves behind, Dr. Maze's accomplishments will continue to have an impact across Iowa, the country, and beyond for a very long time.

Quantifying Environmental Impact: What Should We Measure and How Is It Done?

Thomas Van Dam
Applied Pavement Technology, Inc.
Jutila Center for Global Design and Business, Suite 321
200 Michigan Street
Hancock, MI 49930
TVandam@appliedpavement.com

ABSTRACT

Understanding the environmental footprint of different construction systems is critical for making choices that truly result in more sustainable pavement systems. This session will explain what is important to measure and how to measure it and current approaches for analyzing the environments benefits of different construction systems.

This presentation is part of the Sustainable Concrete Pavement Technologies session.

Key words: environmental footprint—measure—sustainable

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Lessons Learned from 100% Airport Network Condition Survey

Jose A. Villarreal
Department of Civil Engineering
Kansas State University
2151 Fiedler Hall
Manhattan, KS 66506
josev@ksu.edu

George Laliberte
Kansas Department of Transportation
Dwight D. Eisenhower State Office Building
700 SW Harrison
Topeka, KS 66603
georgel@ksdot.org

Mustaque Hossain
Department of Civil Engineering
Kansas State University
2124 Fiedler Hall
Manhattan, KS 66506
mustak@ksu.edu

ABSTRACT

Network-level pavement condition surveys were performed to assess the condition of all paved general aviation airports runways in Kansas in 2008. The pavement condition, in terms of PCI, for each of the 137 paved runways surveyed was calculated following the methodology outlined by ASTM D 5340-04 and adopted by the Federal Aviation Administration. Approximately 68% of the runway pavement sections surveyed were in “good” to “satisfactory” condition. Almost one-third of the network can be rated as “good.” About 21% of the sections surveyed were in “fair” condition. Overall, the condition of the entire network can be rated as “satisfactory.” This 100% network survey study, the first of its kind in Kansas, demonstrates that a MicroPAVER-based pavement management system (PMS) can be developed for general aviation airport pavements in Kansas. The approach and system used to conduct the surveys are applicable to any future network-level condition survey. Through colorful anecdotes, the authors share lessons learned from their experience and give insight about crew responsibilities, training, scheduling, logistics, selection of sample units, handling of data, safety, and supplies needed on the road. Also, advice on unconventional situations like handling encounters with wildlife, dealing with broken equipment, surviving the unpredictable Midwest weather, and other adversities encountered on the road is presented.

Key words: airport pavement—condition survey—lessons—network PMS—PCI

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Facing the Future of Transportation Finance: Challenges and Opportunities in a New Century

Martin Wachs
Transportation, Space, and Technology Program
The RAND Corporation
1776 Main Street
P. O. Box 2138
Santa Monica, CA 90407-2138

ABSTRACT

The nation and states are facing dramatic shortfalls in transportation revenues, the depth of which mark a turning point in the 80-year history of transportation finance. Fundamental reform is needed because the transportation system cannot be maintained, much less expanded, under current financing arrangements. The history of transportation finance illustrates that user financing is essential to assure both efficiency in system performance and equity in terms of sharing the burdens of payment. User fees also provide a path toward environmental sustainability. The history of user financing, going back to the beginning of the motor fuel tax before 1920, holds the key to solutions that are relevant 100 years later. While motor fuel taxes have served well, they are indirect user fees, and new technology enables us to gradually institute far more direct systems of user financing over the coming 20 years . Reliance on general taxes for transportation, such as local option sales taxes, are currently popular, but they are only useful in the short term as a transition to a system more fully based on direct user financing.

The paper that is included in these proceedings was previously published in the summer 2009 edition of *Issues in Science and Technology*.

Key words: finance—motor fuel tax—transportation revenue

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Wachs, "After the Motor Fuel Tax: Reshaping Transportation Financing,"
Summer 2009, p. 85-88, Summer 2009 by the University of Texas at Dallas, Richardson, TX.

MARTIN WACHS

After the Motor Fuel Tax

Reshaping Transportation Financing

Congress should seize the opportunity to shift to a system of direct user fees to support transportation activities.

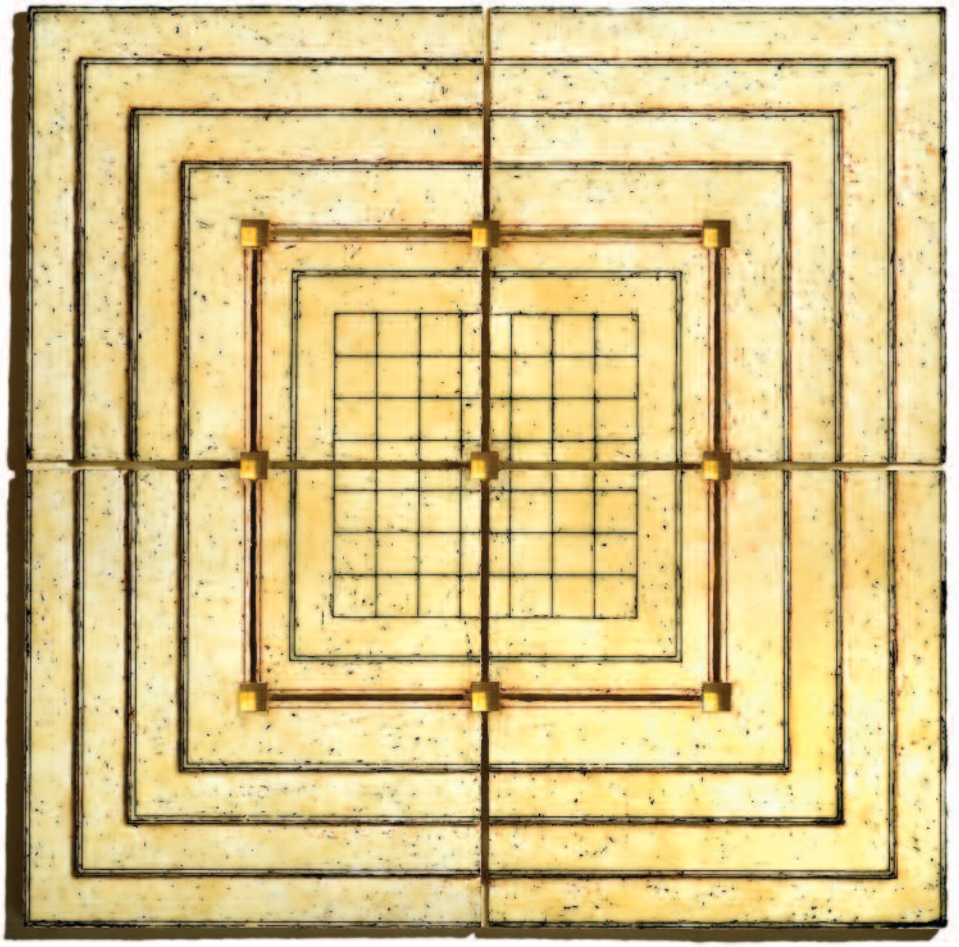
Congress will soon begin considering a new transportation bill that is expected to carry a price tag of $500 billion to $600 billion to support a huge number of projects nationwide. Public debate over the bill is certain to be intense, with earmarks and "bridges to nowhere" being prominently mentioned. But what could become lost in the din is that Congress may well take an important first step in changing the very nature of how the nation raises funds to support its roads and other components of the transit system. Or Congress may lose its courage. If so, the nation will miss a critical opportunity to gain hundreds of billions of dollars in needed revenue for transportation, to reduce traffic congestion, and to price travel more fairly than has been the case for a century.

At issue is whether Congress will continue to rely on the federal motor fuel tax and other indirect user fees as the primary source of revenue for transportation projects, or whether it will begin a shift to more direct user fees. Many observers expect that Congress will step up to the job, but it is far from a done deal. If Congress does act, it will begin what is likely to be a decades-long transition to some form of direct charging on the basis of miles driven.

In its reliance on user fees to support transportation projects, the United States operates differently from most other nations. Most countries tax fuels and vehicles, but they put the proceeds into their general funds and pay for roads and transit systems from the same accounts they use for schools, health care, and other government programs. The United States has preferred to link charges and payments for the transportation system more directly, through a separate system of user-based financing. User fees include gasoline taxes, tolls, vehicle registration fees, and truck weight fees. User fees, imposed by all 50 states as well as the federal government, are intended to charge more to those who benefit from the transportation system and who also impose costs on the system by using it. At the federal level, the largest source of revenue from users for half a century has been the federal excise tax on gasoline and diesel fuel. Proceeds are kept separate from the general budget at the federal level and in most states. Revenues are deposited into separate trust funds, with this money reserved for building, operating, and maintaining transportation systems to directly benefit those who paid the fees. User fees at the federal level, for example, paid more than 90% of the cost of building the national interstate highway system.

One problem, however, is that the federal motor fuel tax, which is a major source of transportation system support, has not been raised for many years; it has been set at 18.4 cents per gallon since Ronald Reagan was president. As the price of gasoline rose during this period, Congress proved reluctant to charge drivers more for road improvements. In fact, when the price of gasoline spiked recently, Congress briefly considered lowering the federal motor fuel tax but backed away after considering the enormous backlog of infrastructure needs and the deteriorating condition of the nation's transportation system. In addition to losing value because of inflation with the passage of time, motor fuel tax revenue is falling in relation to road use because of improved vehicle fuel economy. Higher miles-per-gallon ratings are good for the economy, energy independence, and reduced air pollution. But better fuel economy also means that motorists drive more miles with each fill up at the pump and actually pay substantially less through fuel taxes per mile of driving than they did in past years.

Many supporters of transportation investments continue to believe that the best way to raise desperately needed money to maintain and expand highways and mass transit would be to raise those user fees rather than to turn to general taxes, which are also under stress and are used to fund many other critical programs. But the trend is in the opposite direction. Gradually, faced with a genuine national shortage of funds for transportation infrastructure because fuel taxes have not kept pace with costs, voters in several states have been asked to approve increases in sales taxes to fill the growing gap



CHERYL GOLDSLEGER, *Nine Points,* Wax, oil, pigment, resin on wood, 52 x 52 inches, 2005.

between transportation needs and the revenues available from user fees. Also, as the balance in the federal highway trust fund dipped below zero in September 2008, Congress approved a "one-time" transfer of $8 billion from the nation's general fund into the trust fund to avoid the complete shutdown of federal highway programs. Another such transfer may soon be needed because the transit account within the trust fund is now approaching a zero balance as well.

A century of taxes

In their common form, motor fuel taxes were invented before 1920. With intercity auto and truck traffic growing dramatically, states were strapped in their efforts to pay from general funds for desperately needed highways. Because the need for and costs of state roads varied roughly in proportion to traffic levels, it made sense to cover the costs of those roads by charging the users. Tolls were considered at the time the fairest way to charge users, but they had a major drawback. The cost of collecting tolls—constructing toll booths, paying toll collectors, revenue losses from graft and pilfering, and delays imposed on travelers—absorbed such a large proportion of toll revenue that in some instances they exceeded the revenue generated. Further, developing interconnected road networks required the construction and maintenance of expensive-to-build links (over waterways or through mountain passes) and some lightly used links that could not be financed entirely by locally generated toll revenues.

The solution to this dilemma came when states, starting with Oregon in 1918, adopted an alternative form of user fee: motor fuel taxes. The state charged for road use in rough proportion to motorists' travel, and charged heavier vehicles more than lighter vehicles because they used more fuel per mile of travel. Still, fuel taxes did not quite match tolls in terms of fairness, because they did not levy charges at precisely the time and place of road use. However, fuel taxes cost much less to collect and administer than tolls, and they soon became the nation's principal means of financing its main roads. When the federal government decided in 1956 to implement intercity highways on a national scale, it increased federal fuel taxes and created the Federal Highway Trust Fund, emulating the user-pays principle that had been successful in the states.

Recently, however, two major changes suggest that even if the people and government of the United States prefer to continue to rely on user-based financing, the time may have come to end reliance on motor fuel taxes and to introduce a new approach. The first change is the result of recent improvements in technology. There no longer is a need to rely on toll booths and the manual collection of coins and bills to implement a more direct system of user fees. By charging road users more precisely for particular trips at particular times on specific roads, electronic toll collection—known in some regions as EZPass and FASTRAK—is efficient and widely accepted by motorists.

The second change is more subtle but probably more important. Reliance on the taxation of motor fuels as a source of program revenue in an era of growing concern about fuel efficiency and greenhouse gas emissions creates an unacceptable conflict among otherwise desirable public policy goals. Although higher taxes on fuels might in the near term generate more revenue and encourage the production of more fuel-efficient vehicles that emit less carbon dioxide, the seemingly beneficial relationship between taxation and the achievement of environmental goals breaks down in the longer term. If the nation succeeds in encouraging the vast majority of truckers and motorists to rely on plug-in hybrids and, later, on electric vehicles, vehicles powered by fuel cells, or even vehicles powered by solar energy, it will still be necessary to pay for road construction, maintenance, and transit systems. It can be argued that users should still logically be responsible for bearing their costs, even if they drive nonpolluting vehicles. The nation should not continue programs that discourage government pursuit of dramatic gains in energy efficiency over the longer term for fear that it will lose the revenue needed to build and operate highways and mass transit systems. And quite simply, the nation cannot rely on the gas tax as a road user fee when cars are no longer powered by gasoline.

The road to direct user charges

Motor fuel taxes can continue to provide a great deal of needed revenue for a decade or two. But several types of more efficient, and more equitable, user charges are ready to be phased in. For example, current technology will enable government agencies to institute vehicle miles traveled (VMT) charges as flat per-mile fees. Gradually, agencies could charge higher rates on some roads and lower rates on others to reflect more accurately than do fuel taxes the costs of providing facilities over different terrain or of different quality. This would end cross subsidies of some travelers by others and make travel more efficient by encouraging the use of less congested roads. Unlike gasoline taxes, more direct road user charges also could vary with time of day, encouraging some travelers to make a larger proportion of their trips outside of peak periods, easing rush hour traffic.

In the short term, direct user fees could simply replace fuel taxes in a revenue-neutral switch, but they are attractive, in part, because they can become more lucrative as travel increases, while allowing charges to be distributed more fairly among road users. Initially, some vehicle operators might be allowed to continuing paying motor fuel taxes rather than the newer direct charges, but eventually gas and diesel taxes would be phased out.

Several countries in Europe already are electronically charging trucks directly for miles they drive on major highways, and the Netherlands intends to expand its program to passenger cars. In the United States, Oregon and the Puget Sound Regional Council in the Seattle area have conducted operational trials demonstrating the feasibility of VMT fees, and the University of Iowa is carrying out six additional trials in other parts of the country. The results of these trials are quite encouraging, but questions remain, including questions about optimal technologies.

One thing is clear: Innovation is afoot. In the Oregon trial, for example, a clever innovation allowed drivers of vehicles equipped for the trial program to "cancel" their ordinary fuel taxes when filling up their tanks at service stations and to instead charge VMT fees as part of the bill. This enabled participating and nonparticipating vehicles to function in similar ways.

The most sophisticated trial systems make use of vehicles that are equipped with global positioning system (GPS) satellite receivers and digital maps that enable charges to be varied across political boundaries, by route, and by time of day. But GPS signals are not always available, and these systems also incorporate redundant means for metering mileage. For example, they may have a connection to the vehicle odometer or a link to an onboard diagnostic port that has been included in cars manufactured since 1996 to comply with environmental regulations. None of these systems is perfect, all have implementation costs, and not every vehicle is yet equipped to accommodate each device.

It also is clear that any technological innovation affecting hundreds of millions of vehicles is bound to be complicated by many social and political concerns. Indeed, one of the greatest barriers to the implementation of VMT fees may well be the widespread perception that this approach constitutes an invasion of privacy. It is not yet apparent that metering road use is any more threatening to privacy than using cell phones to communicate, but there is genuine concern that somehow the government will be able to track the travel of each citizen without his or her knowledge. Most technology and policy experts agree, however, that these systems can be structured so that privacy is maintained—for example, by maintaining records in individual vehicles rather than in a central repository and by erasing them after payments are made. It also is possible that many motorists would prefer to forgo privacy protection in order to have access to detailed bills showing each and every trip so that they can audit their charges to be sure they are paying for trips they actually made.

Such issues will need to be addressed sooner rather than later in a reasoned public discussion. For its part, Congress, as it debates the new transportation bill, should consider alternative paths that can be followed in order to ease the adoption of direct user fees. Of course, Congress could still reject such a transition and instead simply raise motor fuel taxes to provide needed revenue. Or in a less likely move, it could commit the nation to funding an increasing portion of its road and transit bills from general revenues.

But the hope in many quarters is that Congress will accept the opportunity and begin specifying the architecture of a national system of direct user charges. This early effort could address a number of questions, such as whether there should be a central billing authority, whether travelers should be able to charge their road use fees to their credit cards, and whether drivers should pay VMT fees each time they fill up the tank or pay them periodically, as with vehicle registration fees. Congress also should consider expanding the current trials in various locations to demonstrate some technology options on a much larger scale. Even better, it should complement such efforts by putting an early system into actual application on a voluntary or limited basis.

For numerous reasons, then, the time is near for Congress to act, and for citizens to ensure that it does. The debate that is about to begin will indicate whether the nation's system of governance has the ability to make complex technological choices that are both cost-effective and just.

Martin Wachs (wachs@rand.org) is director of the Transportation, Space, and Technology Program at the RAND Corporation and professor emeritus of civil and environmental engineering and of city and regional planning at the University of California, Berkeley.

Feasibility of Using Cellular Telephone Data to Determine the Truckshed of Intermodal Facilities

Ming-Heng Wang
Transportation Research Institute
University of Kansas
1530 W. 15th Street, 2160 Learned Hall
Lawrence, KS 66045
mwang4@ku.edu

Steven D. Schrock
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
1530 W. 15th Street, 2150 Learned Hall
Lawrence, KS 66045
schrock@ku.edu

Thomas Mulinazzi
Department of Civil, Environmental, and
Architectural Engineering
University of Kansas
1530 W. 15th Street, 2150 Learned Hall
Lawrence, KS 66045
tomm@ku.edu

ABSTRACT

This paper analyzes the feasibility of using cellular telephone location data to determine the geographic extent of trucks from intermodal facilities. The feasibility analysis includes three aspects: location technology, cell phone penetration, and truck tracking methodology. As the results shows, the cell phone location technology is able to provide accurate location data. It can provide updated location information with no time limitation. Cell phones could be located within an average of 100 meters or less of their actual position. Although only partial cell phone data are available for the truck tracking system, a larger sample size is expected during a longer time period observation when more cell phone data are integrated. The principle and process of truck filtering and tracking from the cell phone database was proposed in this study. To extract trucks from the cell phone data, the analysis network should be categorized into several types of regions based on the land use and truck movement characters. In principle, those tracked cell phones frequently located around truck stops and traveling along the Intestate routes are considered as trucks. Further field study is needed to verify the accuracy of the truck identification and derive the calibration factor for generating truck trips using cell phone location data.

Key words: cellular telephone location—geographic extent—truckshed—intermodal facility

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INTRODUCTION

In today's world of global supply chains, the manufacturing of goods is increasingly spread out throughout the world. The widening supply chain has increased the demand for freight movement across the country. It has also created new opportunities for the railroad industry to compete with the trucking industry for long-haul operations through the development of rail-truck intermodal facilities. As intermodal facilities spread through the region and the nation, one of the major questions is the geographic extent (truckshed) that will impact the local and regional transportation network. There is a need to better understand the impacts of this truckshed on the transportation network. Thus, understanding the extent of the reach of the facility is an important first step to mitigate any negative consequences.

There are many methodologies that can be used to catch the geographic extent of trucks from intermodal facilities. Traditional origin-destination (O-D) methods, such as roadside and truck driver surveys, may be performed but are time consuming. These methods are increasingly unpopular due to the disruption they cause to traffic. Innovative technologies have become available that can reflect the traffic condition and also catch O-D data by tracking vehicle trajectories and destinations. Among these new technologies, the cellular phone tracking technology is one of the potential methods in tracking vehicle location and movement. It has received strong interest from the transportation community. A number of researchers (Ygnace et al. 2000; Lovell 2001; Smith 2006; Cayford 2006; Liu 2008) have evaluated the application of cell phones in travel speed and travel time estimation as well as identifying congestion sections. Some researchers also used cell phone data to derive the O-D matrix (Caceres et al. 2007; Sohn and Kim 2008). However, the application of a cell phone tracking system for determining the truckshed from an intermodal facility has not been addressed.

The objective of this paper is to analyze the feasibility of using cellular telephone location data to determine the geographic extent of trucks related to intermodal facilities. The feasibility analysis includes three aspects: technology, penetration analysis, and a subject (truck) filtering and tracking methodology. First, the cell phone location tracking technologies and related research into its reliability were reviewed and analyzed. Cell phone penetrations, such as market penetration and cell phone signal coverage from cell phone carriers, were also investigated. Then, truck tracking, filtering rules, and procedures were developed to identify the trucks and track their movement from intermodal facilities. The remaining challenge and perspective of using cell phone data in determining the truckshed are discussed. Finally, conclusions and recommendations are listed based on the feasibility analysis in the end.

RESEARCH SCOPE

The purpose of this study is to determine the feasibility of using cellular phone tracking technology in determining the geographic extent of truckshed data from intermodal facilities and understanding the impact of intermodal facilities on the traffic network. The feasibility analysis is going to pursue the following questions:

- Can cell phone tracking technology enable tracking accurate locations to realize a long-haul truckshed?
- Does the sample size of cell phone signals provide enough data to reflect the truck volume and impact on the network?
- How is it possible to determine if the cell phone is a truck or not?
- How is it possible to to track the truck movement based on cell phone location data?

The feasibility study answered those questions and is presented and discussed in three aspects:

1. Technique analysis: Available technologies, which were used to obtain the location information using the cell phone signal, were briefly introduced. Literature on the accuracy of cell phone location tracking system tests and studies were also reviewed.
2. Data coverage and penetration analysis: The number of cell phone subscriptions and signal coverage were investigated. The cluster of possible types of cell phone data and portion of cell phones in use were also analyzed.
3. Truck identification and tracking process: The process, which can extract data from thousands of anonymous cell phone calls to determine if the cell phones are trucks or not, was proposed. Cell phone locations can be tracked by time and space. Criteria used to filter the data were based on the land use and truck movement and stop characteristics.

The structure and operation process of cell phone location data provided by Airsage, Inc. are briefly introduced. The challenge and perspective of using cell phone location data are also discussed.

TECHNOLOGIES ANALYSIS

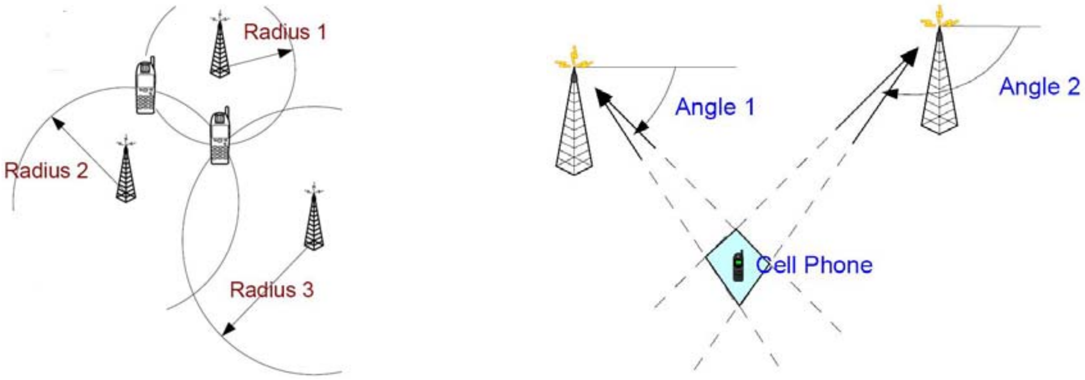
Cell phones are now used as the basic and regular communication tool around the world. Cellular phone location technology began to be one of the new technologies for achieving traffic information after the E911 technology was implemented. In 1996, the Federal Communications Commission (FCC) mandated E911 requirements that cellular location should be provided when 911 emergency calls come to emergency management authorities (FCC 2001).

Cell phone location tracking technologies can generally be divided into two categories: network-based and handset-based. A network-based system utilizes signal information from cell phones to derive their location. In network-based systems, one or several base stations (signal towers) are involved in locating a cell phone. All required measurements are conducted at the base stations, and the measurement results are sent to a location center where the position is calculated. There is no requirement to make any changes to the current handsets. However, the cell phone must be in active mode (i.e., in “talk” mode or sending a signal through the control channel) to enable location measurement. Handset-based systems rely on global positioning system (GPS) enabled wireless phones. The GPS unit in the handset determines the location of a phone, and this information is relayed from the cell phone to a central processing system maintained by the wireless carrier.

Cell Phone Location Calculation and Limitation

According to the requirement by the FCC (2001), location accuracy and reliability should be 100 meters for 67% of calls and 300 meters for 95% of calls for network-based solutions; 50 meters for 67% of calls and 150 meters for 95% of calls for handset-based (GPS-enabled) solutions. Depending on the technology, calculation methodology, and signal path, the accuracy of cell phone location estimation are varied.

For a network-based system, two major methods (as shown in Figure 1) are used to calculate the location of the cell phone. The first one is a triangulation method. In ideal conditions, the cell phone location can be calculated exactly using the triangulation method with computed distances from three nearby stations. However, in reality, the computed distances are dependent on the reflections, diffraction, and multipath occurrences of the phone signal. The triangulation method result is an area instead of a point. The accuracy of triangulation method is about 50–200 meters (Openwave, Inc. 2002)



Triangulation method **Angle of arrival method**

Figure 1. Cell phone location calculation methods (Openwave, Inc. 2002)

The other method is an angle of arrival method. In this method, special antenna arrays are installed at the base stations to calculate the direction the signal. Thus, two stations are enough to calculate in what direction the cell phone signal is coming from. Considering the effects of multi-propagation, this method has an accuracy of about 50–300 meters (Openwave, Inc. 2002).

For handset-based systems using the GPS satellite system to calculate the position of the cell phone, the accuracy is between 5 and 30 meters (Openwave, Inc. 2002). The accuracy is affected by factors such as the ionosphere, troposphere, noise, clock drift, ephemeris data, multipath, etc. The use of GPS in cell phones as a location device also suffers from three main disadvantages (Zhao 2000):

1. GPS signals are too weak to detect indoors and in urban canyons, especially with small cellular-sized antennas.
2. The time required to obtain a GPS position is relatively long, ranging from 60 seconds to a few minutes due to the long acquisition of the satellite navigation message.
3. Due to long signal acquisition time, GPS power dissipation is very high. The computations drain the battery of the phone.

The assisted GPS method, where the wireless network uses a server to perform the calculations and to transmit to the phones, can solve the delay and power consumption issues. In addition, the wireless network can use the differential GPS method to reduce the errors. In this method, a tower with a known position is equipped with a GPS receiver to estimate the total error. The errors are roughly the same in nearby areas; the estimated error can be transmitted to the phone for compensation. Thus, the corrected location in a nearby area is the non-corrected GPS location minus the estimated error. The accuracy is improved to 15 m or less (Wunnava et al. 2007).

Cell Phone Location Technology Application and Test

Several researchers have tested the accuracy of location tracking and the estimation of travel speed using cell phone tracking technology. The results showed that most of cell phone location systems can provide reasonably accurate position data. They were unsuccessful in producing traffic flow information.

CAPITAL, deployed in the Washington, D.C. area in the mid-1990s, was the first major deployment of wireless location technology (WLT) in the United States. The system was able to locate cellular phones within 100 meters of their actual position. The accuracy of the position estimates improved considerably as the number of cellular towers providing directional information increased. Speed information could not be calculated because at least four positions are needed to calculate speed. Less than four position estimates were collected 80% of the time (UMD 1997).

The other deployment using U.S. Wireless was conducted in the San Francisco Bay region in California (Yim and Cayford 2001). Researchers at the University of California, Berkeley obtained 44 hours of wireless location data. They were generally able to determine the location of the cellular phone on the roadway network. They found that the location estimates of cellular phones were regularly accurate within 60 meters, although 66% of cellular devices tracked had at least one outlier with an error of more than 200 meters (Smith et al. 2003).

A research team of the Virginia Transportation Research Council at the University of Virginia (2005) investigated and reviewed over 16 deployments of WLT-based monitoring systems both in the United States and abroad. They concluded that most systems did not produce data of sufficient quality or quantity to provide reliable traffic condition estimates. A similar task was done by a research team at Florida International University (Wunnava et al. 2007). They investigated the maturity of cell phone technologies for application as real-time traffic probes for travel time estimations along the highways and roadways. They found that the cell phone technology is feasible to determine travel time estimations under the normal conditions of free traffic flow, but it is not accurate in congested traffic conditions. The accuracy decreases rapidly as congestion increases.

Feasibility of Cell Phone Location Technology in Truckshed

For a truckshed tracking system, the location data needed are trucks traveling from an intermodal facility to an area in the network or long-haul destination. During the truckshed tracking process, the network will be divided into several areas depending on the land use, network distribution, and state boundaries. The cell phone location data are tracked from area to area. A small range of error in location estimation is accepted for the truckshed tracking system.

According to previous research and tests, location estimation using cell phones was able to provide reasonably accurate location data. Cell phones could be located within an average of 100 meters or less of their actual position. Therefore, existing cell phone location technology is feasible to be used in long-haul truckshed tracking from an intermodal facility.

DATA COVERAGE AND PENETRATION ANALYSIS

Cell phone data coverage and penetration analysis includes the number of cell phone subscribers, the cell phone signal service area, and the portion of cell phones in use that can be tracked.

Cell Phone Penetration

By the end of 2008, there were more than 270 million cell phone subscriptions in the United States, which is about 87% of the total U.S. population (CTIA 2009). About 17.5% of U.S. households are wireless-only. The cell phone market penetration covers nearly all ranges of vehicle users, especially for truck drivers. Thus, using cell phones to develop a truckshed would cover all truck locations, if all the cell phone signals are available.

However, there is no integrated system that could provide all cell phone signals available up to now. The United States is the most competitive in the cell phone market in the world. The top four carries represent only 86% of the market (CTIA 2009). The cell phone signal data belong to different cell phone carriers.

The top four cell phone carriers covered 28.5%, 26.7%, 18.2%, and 12.1%, respectively, of the cell phone market at the end of 2008 (CTIA 2009). Atlanta-based Airsage, Inc. is one of the providers of location, movement, and real-time traffic information based on cellular signaling data. (Airsage 2009)

Cluster of Cell Phone Data

Since the location data are available only when the cell phone is in use, only a partial number of drivers using the intermodal facility can be detected using cell phone location data. Considering the truckshed tracking process using cell phone location data, the cluster and relationship of the number of truck drivers and available cell phone location data in the facility area are illustrated in Figure 2.

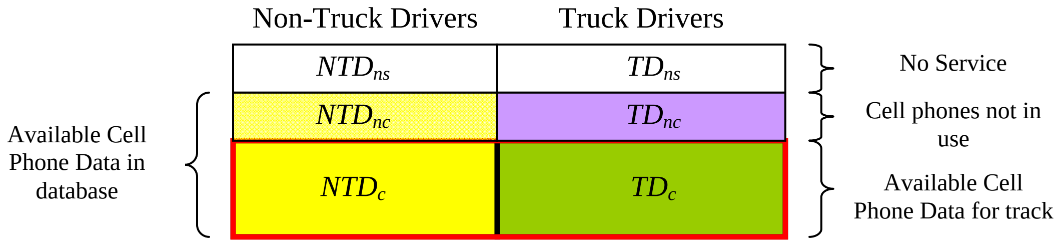


Figure 2. The cluster and relationship of truck drivers and available cell phone data

Total people within the intermodal facility = $\{NT_{ns}, NT_{nc}, NT_c, T_{ns}, T_{nc}, T_c\}$, where
NTD_{ns} = non-truck drivers who do not have a cell phone in provider database,
NTD_{nc} = non-truck drivers who did not use cell phone during data collection period,
NTD_c = non-truck drivers who used cell phone during data collection period,
TD_{ns} = truck drivers who do not have a cell phone in provider database,
TD_{nc} = truck drivers who did not use cell phone during data collection period, and
TD_c = truck drivers who used cell phone during data collection period.

“Non-truck drivers” include employees, workers, customers, train drivers, and possible residential passing by drivers. “No Service” data consist of the drivers who do not have a cell phone or the cell phone data are not available in the provider’s database. “Cell phones not in use” represent those drivers who do not use their cell phone when they are in the facility during the data collection period. The portion of this cluster would decrease over a long time period of observation. “Available cell phone data for track” are the location data when the cell phones carriers make a call or send a message. These data include truck and non-truck drivers’ cell phone data and are the only known data for the cell phone tracking system during the data collection period.

Based on the cluster distribution above, only partial cell phone data are available for the truckshed tracking system. One of difficulties of using cell data to track a truckshed is determining the relationship between the characteristics of sample size and the entire volume of trucks from the intermodal facility. Fortunately, as the cell phone becomes the basic communication tool and more cell phone signal data will be integrated, more cell phone data will be available in the provider database. For a long time period of observation, the available cell phone data for tracking will increase since the probability of cell phone carriers making cell phones will increase. Therefore, the sample size of cell phone data would be enough for long-haul truck tracking from the intermodal facility.

TRUCK IDENTIFICATION AND TRACK PROCESS

One of the major challenges of tracking a truckshed using cell phone data is how to subtract the truck volume from other available cell phone data. All available cell phone data include truck and non-truck carriers' location data in the intermodal facility. The first step of tracking trucks is to catch all available cell phone data within and from the targeted intermodal facility. This step can be done directly by matching the cell phone location data to the facility. Every cell phone signal located within the facility area during the data collection period is an initial data point and used for tracking.

The second step is to identify which cell phone signal point is a truck. Since the cell phone data are anonymous, the vehicle type can only be identified by tracking its moving path and destination. Thus, a database with possible truck stops, such as gas stations, rest areas, and logistic warehouses in the analysis network, is needed. A set of filtering truck rules also needs to be established according to the truck movement and truck stop database.

Truck Stop Database

The characteristics of truck movement and traveling pattern and destination are different from passenger cars. The big difference is that most trucks travel for long distances and frequently stop at popular truck stops, such as gas stations, rest areas, and logistic warehouses. Thus, those popular and possible truck stop locations should be established during the truckshed process. The Interstate highway network and city land use information could be referenced to create the database. Truck stop information available on the Internet also can be embedded into the database. During the tracking process, the observed network is divided into several regions based on the truck stops and land use information. In principle, the analysis network can be categorized into several types of regions based on the land use and truck movement characteristics. Possible types of regions include intermodal facilities, truck stops, rest areas, supermarket and logistic warehouses, business regions, and residential regions.

Truck Filter Rules and Tracking Process

Conversely, passenger car trips from the intermodal facility most likely are local and short distance. Those passenger cars can be filtered based on their destination and filter rule. For example, if the cell phone data from the intermodal is found in the residential or business area most of time, this point will be considered as a non-truck driver.

Depending on the size and the divided regions of the network, the truck filtering rules are varied. However, the principle of the filtering rules should be based on the results of a long tracking period. Figure 3 illustrates the procedure of filtering truck and tracking a truckshed. During the analysis process, only and all those cell phone signals that have ever appeared within the facility area are filtered and tracked along the entire network data. Those tracked cell phones frequently located around truck stops with a certain traveling pattern and following the filtering rules could be considered as trucks. All the filtered truck cell phones are embedded into the network map to derive the truckshed and present the impact of intermodal facility on the entire network.

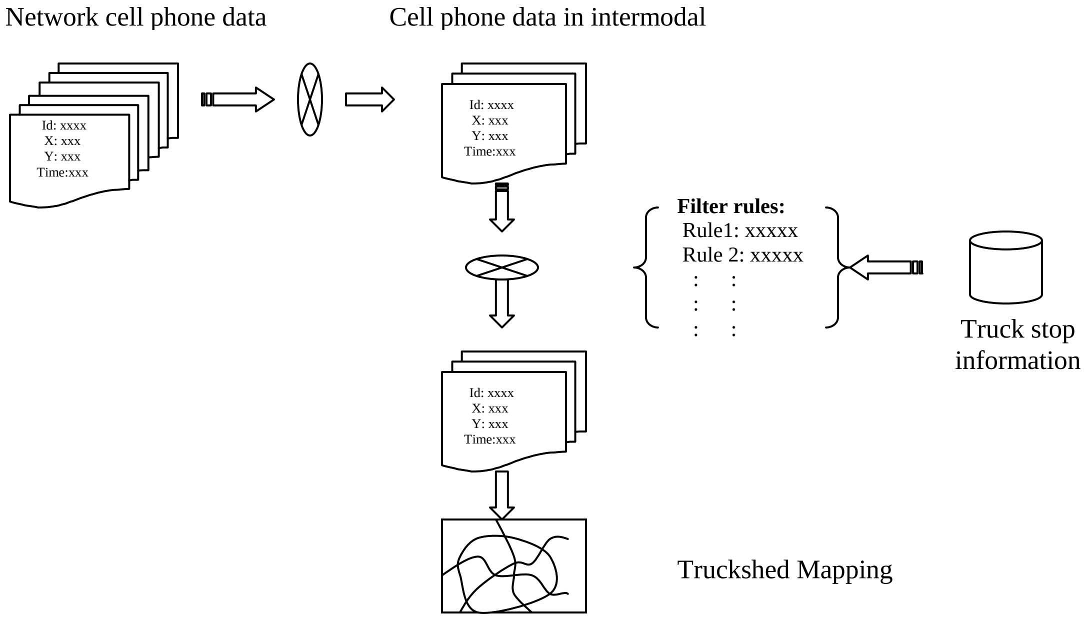

Network cell phone data
Cell phone data in intermodal
Id: xxxx
X: xxx
Y: xxx
Time:xxx
Id: xxxx
X: xxx
Y: xxx
Time:xxx
Filter rules:
Rule1: xxxxx
Rule 2: xxxxx
Truck stop information
Id: xxxx
X: xxx
Y: xxx
Time:xxx
Truckshed Mapping

Figure 3. Procedure of filtering truck and tracking the truckshed

Every cell phone signal point includes a unique identification number, location data in the X and Y axes, and operation time. An electronic map is also needed to embed all the cell phone locations into the designed region of the network. Data are sorted and tracked in the format shown in Table 1.

Table 1. Sorted cell phone signal data

Point ID	**t1**	**t2**	**T2**	**t2**	**…**	Filter result
1	**n1(0,0)**	**N1(x,y)**	**n1(x,y)**	**n1(x,y)**	**…**	**T or NT**
2	**n2(0,0)**	**N2(x,y)**	**n2(x,y)**	**n2(x,y)**	**…(Location)**	**T or NT**
3	*******	**N3(0,0)**	**n3(x,y)**	**n3(x,y)**	**…**	**T or NT**
⋮	⋮	⋮	⋮	⋮	**…**	⋮

where
ID = cell phone initial ID as the new point appears in the intermodal facility,
T = time interval; system is scanned based on a set time interval (30 sec, 5 min, etc.), and
N(x,y) = cell phone location; the region where the cell phone locates is determined.

Vehicle types are determined based on the cell phone point tracking data and proposed truck filter rules in Table 2 for the network. Note the filtering rules are supposed to be changed based on the analysis system.

Table 2. Truck filter rule for the sample network

Filtering rule	IF the cell phone point	THEN the point is
1	Stays in the same region more than 4 hrs	Non-truck driver
2	Heads to a business or residential area	Non-truck driver
3	Moves forward and backward between intermodal and logistic warehouse and supermarket	Truck driver
5	Is long-haul traveling along the Intestate	Truck driver

If the cell phone signal doesn't appear after leaving the intemodal facility during the data analysis time period, the data point will be considered as invalid data. However, for long-haul tracking, this kind of data would be limited.

LOCATION PROVIDER AND CHALLENGE

Cell Phone Location Provider

Cell phone location data will be provided by Airsage, Inc., a major cell phone location provider in the United States. AirSage has long-term cooperation with Sprint wireless company to provide traffic condition and traffic speed. Airsage system has been tested in several metropolises (Smith 2006; Liu 2008). Recently, Airsage has contracted with Verizon Wireless to generate more cell phone data, which will cover nearly 50% of cell phone data in the United States.

Figure 4 shows the coverage of cell phone location and traffic condition data provided by the Airsage system. The system provides real-time, historical and predictive traffic information for 127 U.S. cities. Most of the data concentrate on the major metropolis areas. The area is expected to expand as more cell phone data are included.

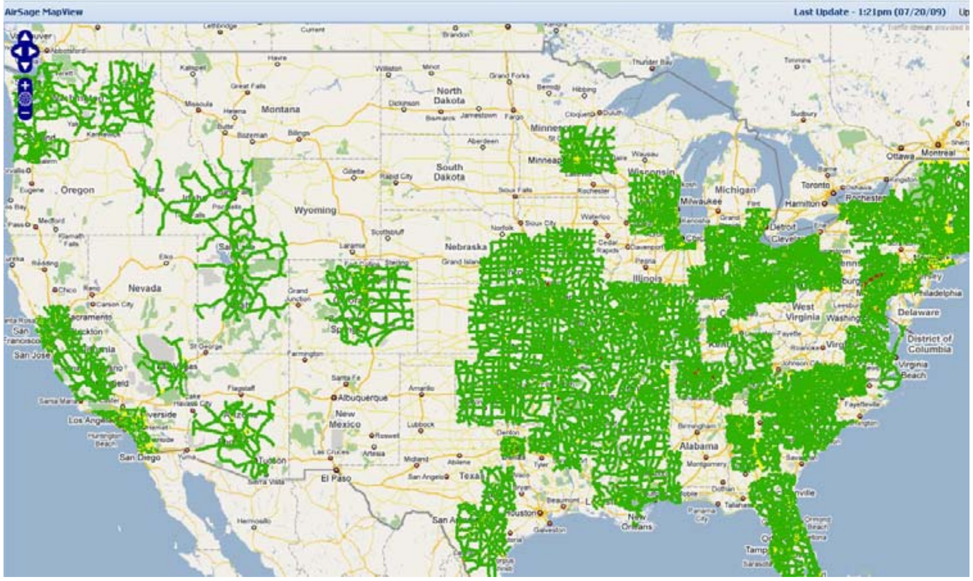

AirSage MapView
Last Update - 1:21pm (07/20/09)
Montana
North Dakota
South Dakota
Oregon
Wyoming
Nevada
Nebraska
Michigan
Wisconsin
New Mexico
Texas
Alabama
Delaware
Illinois
West Virginia

Figure 4. Airsage live traffic service area (Airsage, Inc.)

Location data provided by Airsage system are quite simply a coordinate along an X and a Y axis. In addition, every cell phone has a unique identity number (not cell phone) and the time when the cell phone is in use condition. The location data are provided in a 30 second basis. Every 30 seconds, all the locations of all cell phones in use in the network is recorded and presented. Relative to movement data, AirSage claimed on their website that the Airsage system has the ability to anonymously track the approximate origin and destination of nearly every cell phone signal down to a few hundred feet.

Challenge and Perspective

Although the cell phone provider can provide a wide range of cell phone location, several challenges still remain for using cell phone data to generate the truckshed from an intermodal facility.

1. Simple size issues: The cell phone location can only be detected when the cell phone is in use. If the truck drivers did not used a cell phone as the facility, the data would not be tracked. Fortunately, long-term observations may conquer this issue if the trucks routinely return to the facility.
2. Database storage: Since the cell phone location data for the entire network are recorded every 30 seconds, especially for long-term and wide-range network tracking, the storage needed to record these data is expected to be huge.
3. Truck stops and land use categorization: Although truck stop information is available, to embed this information into a truck tracking system is still a big challenge. Since the region designed for filtering trucks will affect the accuracy of truck identification, dividing the network into appropriate regions would be a difficult task.
4. Truck data calibration and verification: Since only partial truck location can be tracked, field trip data collection is needed to check the portion of trucks that have been tracked by using cell phones. The calibration factor is also needed for generating the truck trips from the intermodal facility.

Cell phone location technology is a convenient and effective methodology to track the geographic extent (truckshed) from an intermodal facility. It can provide an updated truckshed with no time limited. As more cell phone data are integrated in the provider’s database, more precise location data will be available to use in tracking the truckshed from an intermodal facilities.

CONCLUSION AND RECOMMENDATION

Understanding the extent of the reach of the intermodal facility is an important first step to mitigate any negative consequences. In order to determine the feasibility of using cellular telephone location data in deriving the geographic extent (truckshed) from intermodal facilities, this paper conducts the feasibility analysis in three aspects: technology, penetration analysis, and truck tracking methodology.

According to previous researches and tests, location estimates using cell phones were able to provide reasonably accurate location data. Cell phones could be located within an average of 100 meters or less of their actual position. The cell phone location technology is feasible to use in long-haul truckshed tracking from the intermodal facilities.

The cell phone data coverage and penetration analysis showed that only partial cell phone data are available for the truckshed tracking system. Prospectively, more cell phone signal data will be integrated in the provider database, which will increase the sample size and coverage of cell phone location data. For a long time period of observations, the available cell phone data for tracking will also increase since the probability of cell phone carriers making cell will increase.

Since the cell phone data are anonymous, the vehicle type can only be identified by tracking its moving path and destination. Thus, the truck identification and tracking process should including a database covering truck stops and land use information as well a set of truck filtering rules. The analysis network can be generally categorized into several types of regions based on the land use and truck movement characteristics. Depending on the size and the divided regions of the network, the truck filtering rules are varied. In principle, those tracked cell phones frequently located around truck stops and traveling along the intestates for long trip are considered as trucks.

Some challenges for using cell phone data in tracking trucks still remain, such as sample size, data storage, and land use categorization for truck filtering. Long-term observations are needed to increase the sample size obtained in the intermodal facility. Field data collection for verifying the accuracy of the truck identification and deriving the calibration factor of generating truck trips are areas for further study.

ACKNOWLEDGEMENTS

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Performance Properties of Ternary Mixes

Paul Tikalsky
Department of Civil and Environmental Engineering
University of Utah
104 CME, 122 Central Campus Dr.
Salt Lake City, UT 84112
tikalsky@civil.utah.edu

Kejin Wang
Department of Civil, Construction, and Environmental Engineering
Iowa State University
492 Town Engineering
Ames, IA 50011
kejinw@iastate.edu

Peter Taylor
National Concrete Pavement Technology Center
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
ptaylor@iastate.edu

ABSTRACT

The work to date and the initial findings from the TPF-5(117) project on ternary concrete mix designs will be presented. The objective of this project is to improve the performance of concrete pavements while beneficially incorporating supplementary cementitious materials (SCMs).

This presentation is part of the Sustainable Concrete Pavement Technologies session.

Key words: concrete—SCM—ternary mix

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Concrete Pavement Surface Characteristics Program: An Update

Paul Wiegand
National Concrete Pavement Technology Center
Iowa State University
2711 South Loop Drive Suite 4700
Ames, IA 50011
pwiegand@iastate.edu

Dale Harrington
Snyder and Associates
2727 SW Snyder Blvd.
P.O. Box 1159
Ankeny, IA 50023
dharrington@snyder-associates.com

Robert Otto Rasmussen
Transtec Group, Inc.
6111 Balcones Drive
Austin, TX 78731
robotto@thetranstecgroup.com

Ted Ferragut
TDC Partners, Ltd.
417 South Saint Asaph Street
Alexandria, VA 22314-3747
tferragut@tdcpartners.com

ABSTRACT

The acoustical characteristic of pavement surfaces continues to be an important factor for pavement owners. The National Concrete Pavement Technology Center has been leading a major study to understand how to optimize surface characteristics of concrete pavements. The findings from this research will be reviewed as well as the interim guidance being given to the participants in the study.

This presentation is part of the Sustainable Concrete Pavement Technologies session.

Key words: acoustical characteristic—better practices—concrete pavements

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Alternative Test Methods for Measuring Permeability of Asphalt Mixes

R. Christopher Williams
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
428A Town Engineering
Ames, IA 50011
rwilliam@iastate.edu

Joe Schroer
Missouri Department of Transportation
1617 Missouri Blvd.
P.O. Box 270
Jefferson City, MO 65102
joe.schroer@modot.mo.gov

Mohamed Abdel Raouf
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
136 Town Engineering
Ames, IA 50011
maraouf@iastate.edu

ABSTRACT

One of the primary assumptions in structural pavement design for conventional pavements is that a flexible (hot mix asphalt) pavement be impermeable. The basis for this design approach is to minimize moisture infiltration and thus maintain adequate support from the underlying unbound materials. In recent years, with the implementation of the Superpave mix design system, hot mix asphalt (HMA) pavements have been produced with coarser gradations than previously with the Marshall mix design method. A non-destructive method, such as permeability testing, also has the potential to partially characterize the HMA quality as more timely than destructive methods and not leave imperfections in a newly constructed pavement.

This presentation identifies the nominal maximum aggregate size (NMAS), the theoretical maximum specific gravity of the mixture (Gmm), and thickness of the pavement or core as statistically important factors influencing permeability and air voids. Generally, larger NMAS mixtures have an influence of lower permeability and lower air voids than smaller NMAS mixtures. Higher Gmm mixtures generally produced mixtures with higher permeability and higher air-void values. Although statistically significant, the influence of thickness varied from one method/technology to another.

Three methods of permeability testing were identified as viable: the Kentucky Air Permeameter, the Karol-Warner Permeameter, and the National Center for Asphalt Technology (NCAT) Permeameter. This paper recommends utilizing an NCAT Permeameter for field testing as part of the quality assurance/quality control process. The specific criteria for using an NCAT Permeameter as part of a percent within limit specification is 1560 x 10^{-5}cm/sec for the upper specification limit and 0 cm/sec for the lower specification limit. Although the literature did not identify criteria for the NCAT Permeameter, 125 x 10^{-5} cm/sec average permeability criteria for the Karol-Warner device has been identified by Maupin at the Virginia Transportation Research Council as a criteria. The paper identified the viability of using a Karol-Warner Permeameter as part of the mix design as it has a strong relationship to the NCAT Permeameter, which is not capable of testing gyratory-compacted samples. A corresponding Karol-Warner Permeability criteria identified in this study is an upper specification limit of 530 x 10^{-5} cm/sec and 0 cm/sec for the lower specification criteria and results in an average permeability value of 265 x 10^{-5} cm/sec.

Key words: air voids—density—HMA—permeability—SMA

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Laboratory and Field Evaluation of the 24th Street Bridge

Terry J. Wipf
Bridge Engineering Center
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
tjwipf@iastate.edu

Travis Hosteng
Bridge Engineering Center
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
kickhos@iastate.edu

Brent M. Phares
Bridge Engineering Center
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
bphares@iastate.edu

Jake Bigelow
Bridge Engineering Center
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
jbigelow@iastate.edu

ABSTRACT

The Iowa Department of Transportation (Iowa DOT) replaced the 24th Street Bridge in Council Bluffs, IA, which crosses over Interstate 80/29. The design includes prestressed, precast deck panels that are longitudinally post-tensioned on composite steel girders. The 24th Street precast deck panels represent a step forward from previous applications of similar systems in Iowa and strive to further improve the accelerated bridge construction design concept. The testing and evaluation for this structure consists of two primary components: a laboratory component and a field component. The laboratory testing program was intended to help answer specific design and construction questions of the Iowa DOT for the final design. The main focus of the field-testing program was to evaluate the structure both during and after construction. Results from laboratory testing and field testing are presented herein.

Key words: accelerated bridge construction—bridge—precast concrete

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1. INTRODUCTION

The Iowa Department of Transportation (Iowa DOT) replaced the 24th Street Bridge in Council Bluffs, IA, which crosses over Interstate 80/29. The design includes prestressed, precast deck panels that are longitudinally post-tensioned on composite steel girders. The 24th Street precast deck panels represent a step forward from previous applications of similar systems in Iowa and strive to further improve the accelerated bridge construction design concept. The successful implementation of the project has far reaching implications for the State of Iowa, as it will allow for continuation and development of work initiated through previous IBRC projects. The project directly addresses the IBRC goal of demonstrating (and documenting) the effectiveness of innovative construction techniques for the construction of new bridge structures.

The testing and evaluation for this structure consists of two primary components: a laboratory component and a field component. The laboratory testing program was intended to help answer specific design and construction questions of the Iowa DOT for the final design. The main focus of the field testing program was to evaluate the structure both during and after construction. Results from laboratory testing and field testing are presented herein.

1.1 Bridge Description

The superstructure of the 24th Street Bridge is comprised of prestressed, precast deck panels supported on steel girders. There are 12 lines of girders spaced at 9 ft 0 in. on center, with a maximum girder length between field splices of 121.75 ft. Refer to Figure 1 for a structural steel layout of the superstructure and a girder elevation view (the label system of the girder layout is used throughout the document).

The steel girders were designed to act compositely with the deck, requiring the deck be connected to the girders through the use of shear connectors. The composite action was obtained with the use of shear stud pockets in the precast panels. Shear studs were welded on the girder within the “pockets” of the panels; the “pockets” were then filled with grout. The shear stud pockets are spaced at 2 ft along the length of the panels. The deck panels are 10 ft long x 52 ft 4 in. wide x 8 in. thick. Each panel has 28 1 in. by 3 in. embedded ducts to house the longitudinal post-tensioning strands.

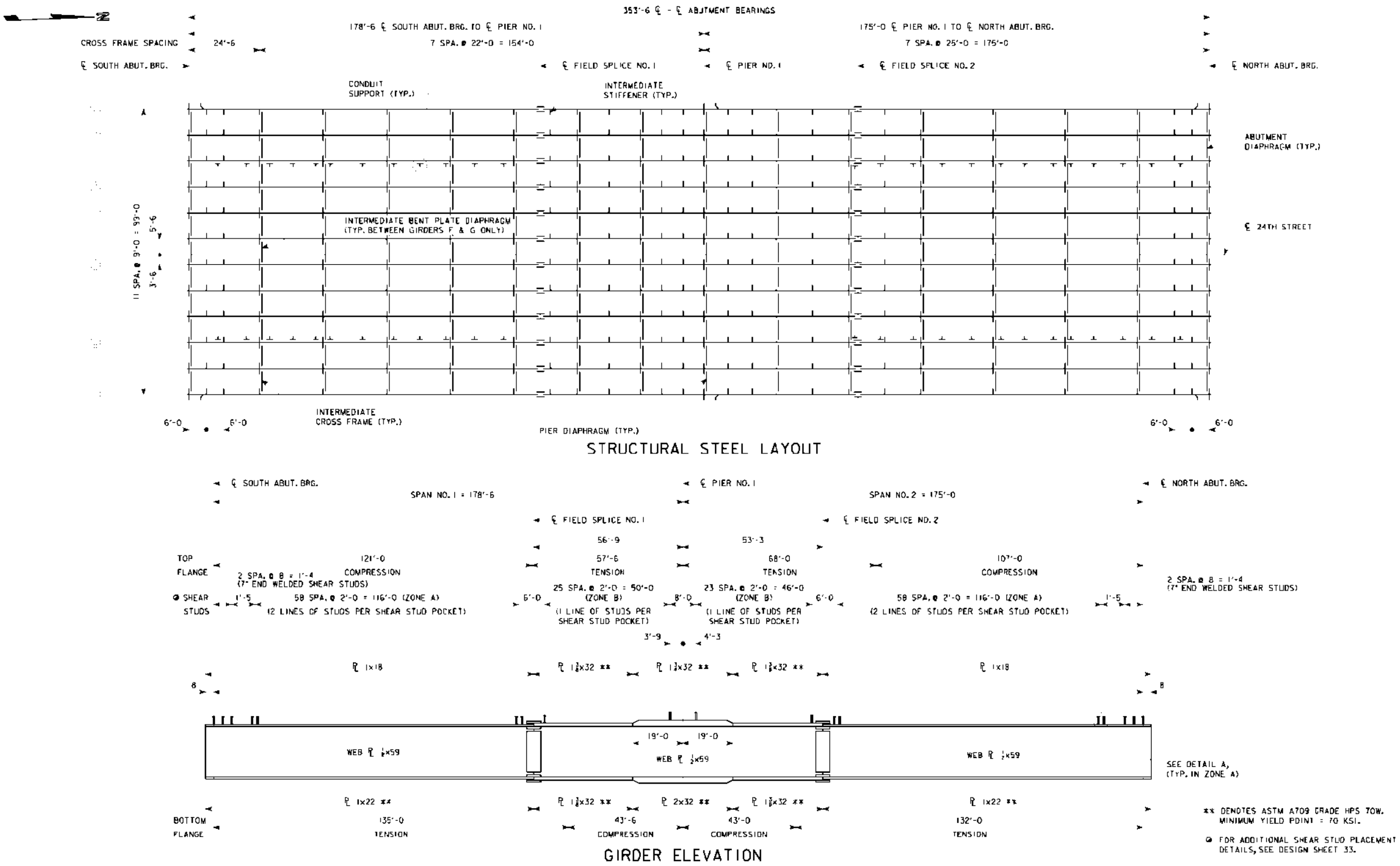

353'-6 ℄ - ℄ ABUTMENT BEARINGS
178'-6 ℄ SOUTH ABUT. BRG. TO ℄ PIER NO. 1
175'-0 ℄ PIER NO. 1 TO ℄ NORTH ABUT. BRG.
CROSS FRAME SPACING
24'-6
7 SPA. @ 22'-0 = 154'-0
7 SPA. @ 25'-0 = 175'-0
℄ SOUTH ABUT. BRG.
℄ FIELD SPLICE NO. 1
℄ PIER NO. 1
℄ FIELD SPLICE NO. 2
℄ NORTH ABUT. BRG.
CONDUIT SUPPORT (TYP.)
INTERMEDIATE STIFFENER (TYP.)
ABUTMENT DIAPHRAGM (TYP.)
INTERMEDIATE BENT PLATE DIAPHRAGM (TYP. BETWEEN GIRDERS F & G ONLY)
℄ 24TH STREET
11 SPA. @ 9'-0 = 99'-0
5'-6
3'-6
6'-0
6'-0
INTERMEDIATE CROSS FRAME (TYP.)
PIER DIAPHRAGM (TYP.)
6'-0
6'-0
STRUCTURAL STEEL LAYOUT
℄ SOUTH ABUT. BRG.
SPAN NO. 1 = 178'-6
℄ PIER NO. 1
SPAN NO. 2 = 175'-0
℄ NORTH ABUT. BRG.
℄ FIELD SPLICE NO. 1
℄ FIELD SPLICE NO. 2
56'-9
53'-3
TOP FLANGE
121'-0 COMPRESSION
57'-6 TENSION
68'-0 TENSION
107'-0 COMPRESSION
2 SPA. @ 8 = 1'-4 (7" END WELDED SHEAR STUDS)
● SHEAR STUDS
1'-5
58 SPA. @ 2'-0 = 116'-0 (ZONE A) (2 LINES OF STUDS PER SHEAR STUD POCKET)
6'-0
25 SPA. @ 2'-0 = 50'-0 (ZONE B) (1 LINE OF STUDS PER SHEAR STUD POCKET)
8'-0
23 SPA. @ 2'-0 = 46'-0 (ZONE B) (1 LINE OF STUDS PER SHEAR STUD POCKET)
6'-0
58 SPA. @ 2'-0 = 116'-0 (ZONE A) (2 LINES OF STUDS PER SHEAR STUD POCKET)
1'-5
2 SPA. @ 8 = 1'-4 (7" END WELDED SHEAR STUDS)
3'-9
4'-3
PL 1x18
PL 1⅞x32 **
PL 1¾x32 **
PL 1⅞x32 **
PL 1x18
8
8
19'-0
19'-0
WEB PL ½x59
WEB PL ½x59
WEB PL ½x59
SEE DETAIL A, (TYP. IN ZONE A)
PL 1x22 **
PL 1⅞x32 **
PL 2x32 **
PL 1⅞x32 **
PL 1x22 **
BOTTOM FLANGE
135'-0 TENSION
43'-6 COMPRESSION
43'-0 COMPRESSION
132'-0 TENSION
** DENOTES ASTM A709 GRADE HPS 70W. MINIMUM YIELD POINT = 70 KSI.
● FOR ADDITIONAL SHEAR STUD PLACEMENT DETAILS, SEE DESIGN SHEET 33.
GIRDER ELEVATION

Figure 1.1. 24th Street Bridge layout and elevation view

2. LABORATORY TESTING

2.1 Background

The bridge design calls for shear studs to be welded to the top flange of the superstructure girders within a preformed deck panel “pocket” to provide composite action between the precast panels and the girders. A constructability concern of the pocket configuration needed to be addressed; thus, a mock-up was created to test the constructability of the pocket size and grout flowability.

In order to longitudinally post-tension the deck panels, ducts are installed in the deck panels, which must then be connected in the field prior to stringing the post-tensioning strands. The design requires the ducts in adjacent precast panels be joined by a duct coupler and be sealed with a waterproof covering in order to protect the ducts from infiltration of grout when the transverse joints are cast. To test the integrity of this coupler and waterproof covering, a mock-up of the system was made and the performance verified.

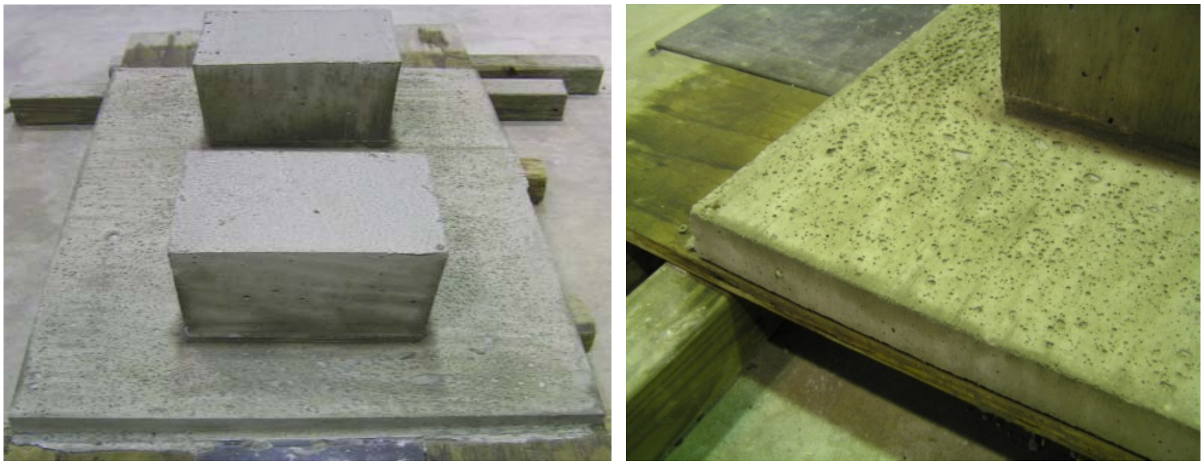
The precast panels require the transverse edges of the panels be “roughened” for shear resistance prior to grouting the transverse joints. Four different alternatives were tested to determine the most effective surface treatment for shear resistance: Control, Diamond Plate Form, Chemical Etching, and Sandblasting. Recommendations for surface treatment are made based on the test results.

2.2. Laboratory Test Results

2.2.1 Shear Stud Pocket Investigation—Bend Test and Grout Flowability

To investigate the ability to perform the necessary bend test in the specified stud pocket, a mock-up of two successive stud pockets was created out of plywood with a piece of plate steel simulating the beam top flange. The mock-up was also used to investigate the ability of grout to flow through the stud pockets into the haunch between the precast panels and the steel girder top flanges. The haunch was taken as the minimum allowable haunch. Six studs were welded to the plate steel as specified in the plans.

No difficulty was found in bending the four studs in the corners of the stud pocket. Bending of the center two studs was more arduous but still possible. The grout was then placed into the two stud pockets in the mock-up and agitated with an electric vibrator. Figure 2.1 shows the mock-up after removal of the forms, clearly indicating that grout can sufficiently flow through the stud pockets and into the haunch area in the specified dimensions.

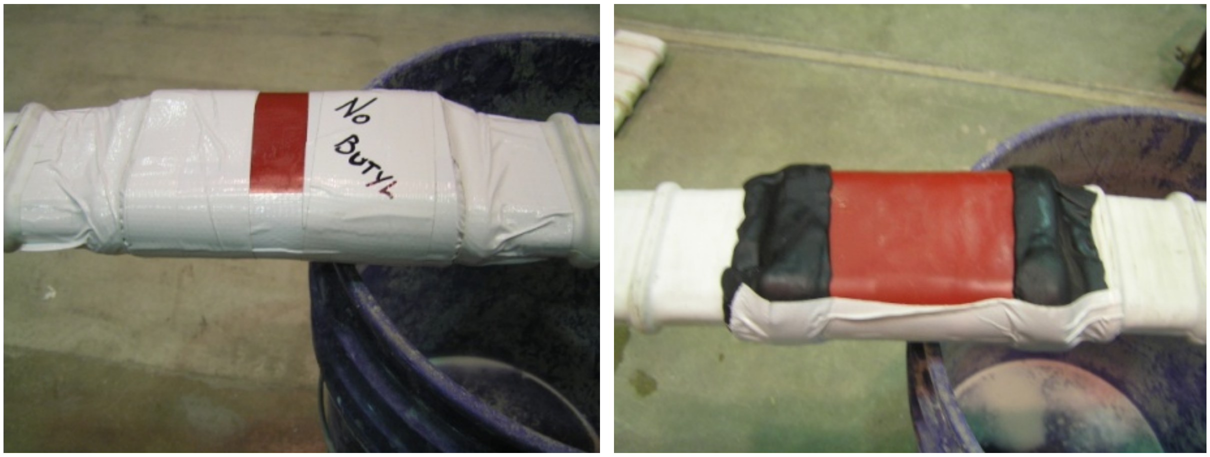


a. Grout filled pockets and haunch **b. Close up of grout at haunch**

Figure 2.1. Stud pocket and haunch mock-up after form removal

2.2.2 Evaluation of Duct Splicing Performance

The 1 in. x 3 in. duct splice connection detail was evaluated to determine if grout or moisture would seep into the duct at the connection. Two mock-up duct splices were constructed and placed in grout. One duct splice was constructed of Polyken waterproof duct tape, as shown in Fig. 3.2a. The second splice mock-up was constructed with butyl rubber strips wrapped around the joint interfaces of the duct and coupler; the longitudinal joint of the coupler was sealed with a strip of Polyken duct tape, as shown in Fig. 3.2b. Both methods of grout proofing the splices were found acceptable.



a. Waterproof duct tape **b. Butyl rubber and duct tape**

Figure 2.2. Mock-up splice connections for 1 in. x 3 in. P-T duct

2.2.3 Evaluation of the Influence of Surface Treatment on Transverse Joint Shear Transfer

The following four surface “roughening” alternatives were tested and evaluated: Control (i.e., no roughening), Diamond Plate forms, Chemical Etching, and Sandblasting. For each alternative, three specimens were tested, each specimen consisting of a 6 in. x 6 in. x 6 in. grout cube sandwiched between two concrete cubes of similar dimensions. Push-out tests were performed on each of the specimen. Figure 2.3 illustrates a typical deflection vs. load curve for the sandblasted specimen.

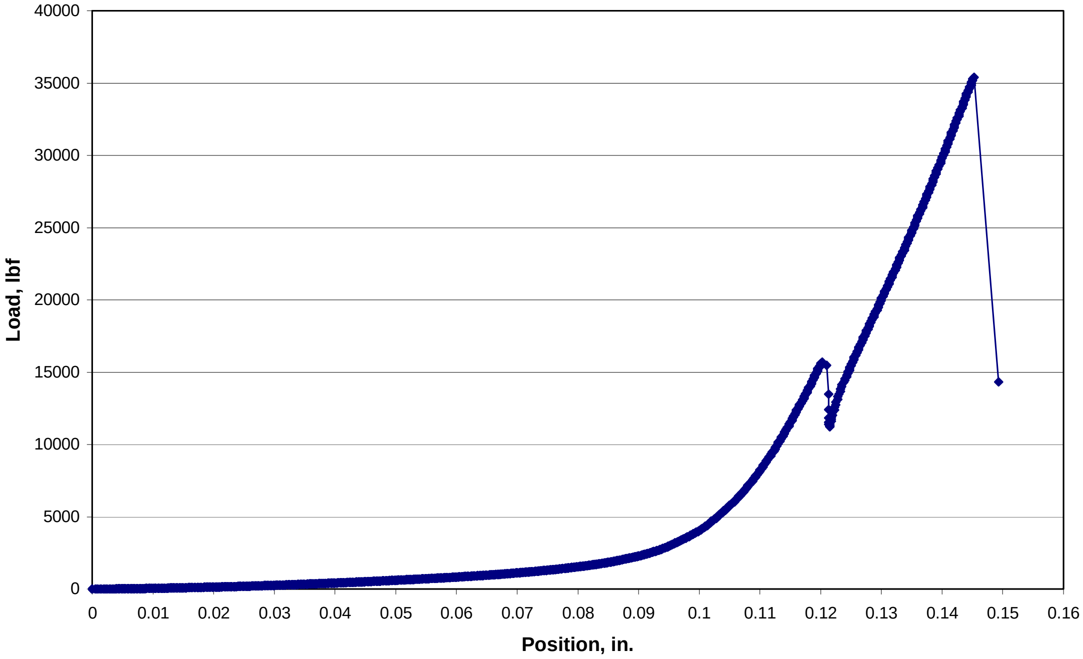


Figure 2.3. Deflection vs. load plot for Sandblasted specimen #3

A progression in higher shear bond strength is evident as one moves from the Control specimens to Diamond Plate specimens to Chemically Etched specimens to Sandblasted specimens. Sandblasting appears to be the most effective surface treatment for resistance to pure shear.

2.3 Laboratory Testing Conclusion

Three laboratory tests were conducted to evaluate design and construction issues for the 24th Street Bridge in Council Bluffs, IA. The tests consisted of evaluating the shear stud pockets,(including the stud bend test and grout flowability), duct splicing performance, and the influence of surface treatment on transverse joint shear transfer. The following conclusions were made from the laboratory testing:

1. No difficulty in installing the shear studs in the precast panel pockets was foreseen by the contractor or encountered by the research team.
2. Conducting the bend test on the studs in the precast panel pockets was feasible for all six studs.
3. Grout with the proposed slump can sufficiently flow through the stud pockets into the haunch areas. It is anticipated that air will remain trapped in these areas. The impact of the voids is not known.
4. The waterproof duct tape and butyl rubber methods of grout proofing the duct splices were both acceptable.
5. Sandblasting the surface of the concrete/grout joint was the most effective surface treatment for resistance to shear.

3. FIELD TESTING

3.1 Background

The Iowa State University Bridge Engineering Center in conjunction with the Iowa DOT developed the monitoring and evaluation plan for the bridge. The plan entailed investigating prestressed and post-tensioning strand corrosion, panel joint pressure during post-tensioning, strains during deck panel handling, and static live-load performance of the completed bridge.

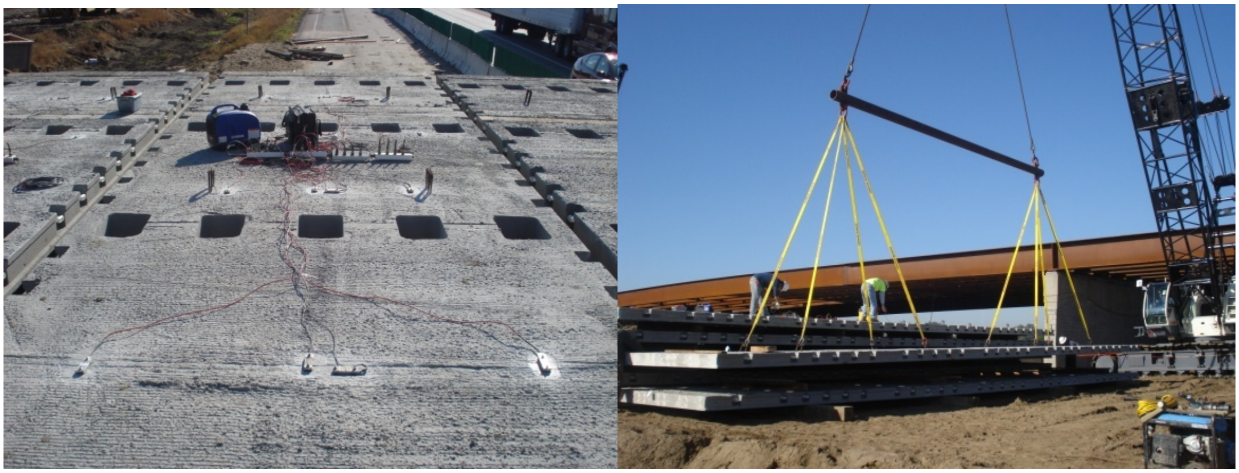
3.2. Field Test Results

3.2.1 Corrosion Monitoring

The six prestressed strands were instrumented with Vetek V2000 corrosion monitoring systems during panel fabrication, and six sacrificial post-tensioning strands were instrumented during bridge construction. The corrosion was checked after the panels were placed on the girders and approximately six months later. No corrosion is evident on any of the strands at this time.

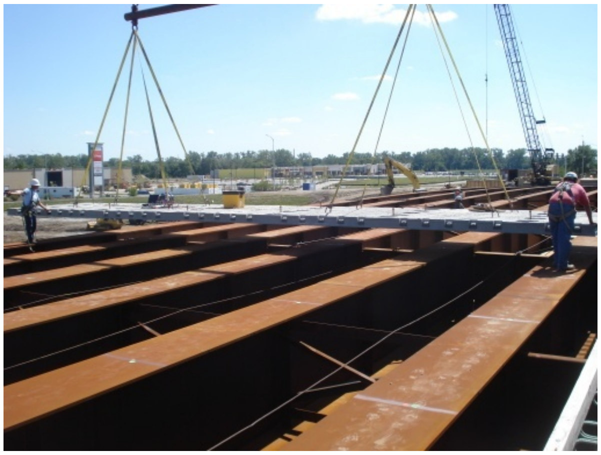
3.2.2 Handling Performance of Precast Deck Panels

To better understand how the precast panels are impacted during handling and placement, two panels were instrumented with strain gauges. These panels were monitored from the time the panels were picked up in the staging yard until placement on the bridge. Figure 3.1a shows the field layout of the strain gauges on the panel. Figures 3.1b and 3.1c show the panels being picked up from the staging yard and moved onto the bridge girders. The strain data obtained from the handling of the panels has not been evaluated for performance.



a. Strain gauge placement **b. Lifting configuration of panel**

Figure 3.1. Handling of precast deck panels



c. Placement on girders

Figure 3.1. Handling of precast deck panels

3.2.4 Diagnostic Live-Load Test

A diagnostic live-load test was conducted on the completed bridge to compare structural performance with expected design performance. Strain gauges and deflection transducers were installed on critical superstructure members, and semi-controlled vehicle loads crossed the bridge. Strain gauges were placed at the pier, mid-span, and abutment of the north span. Deflection transducers were placed at the mid-span of the north span only. Accelerometers were also placed at mid-span and quarter-span of the bridge north span. A fully loaded three axle dump truck was driven over the bridge. The transverse position of the truck was varied with six different load cases.

3.2.4.1 Deflection Transducers

Deflection transducers were installed on the bottom flange of the seven east girders. Representative time-history deflections for Load Case 2 at the north span mid-span with respect to front axle position are shown in Figure 3.2. The load truck was located in the center of the east driving lane for Load Case 2. In general all load cases produced the same deflection shape for the bridge with positive deflection when the truck was on the south span and negative deflection when the truck was on the north span. For all the load cases a deflection range of -0.32 to +0.21 in. was seen when the truck was on the south span and a range of -0.4 to +0.23 in. when the truck was on the north span for the seven girders. The maximum north span deflection of -0.41 in. corresponds to a span to deflection ratio of L/5120.

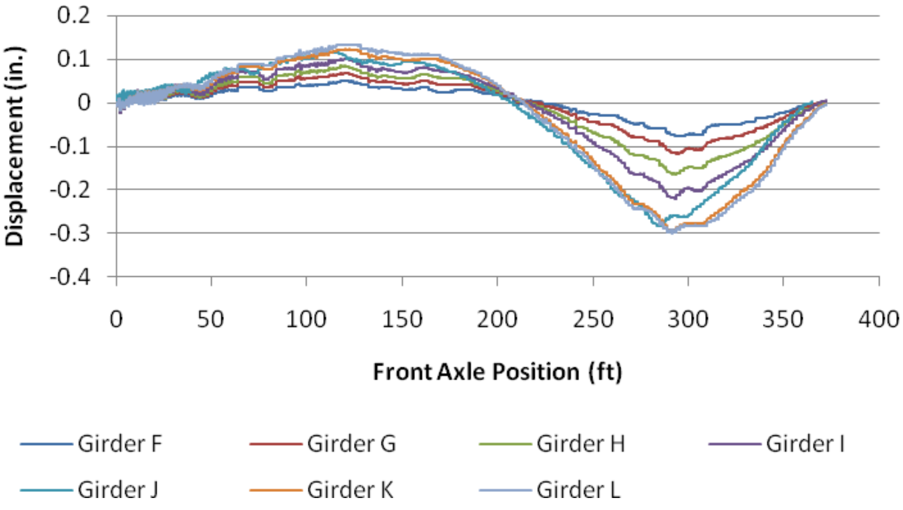


Figure 3.2. Representative time history deflections for Load Case 2

3.4.2.2 Strain Gauges

A total of 36 strain gauges were placed on the bridge, with 20 located at mid-span and 8 located at both the pier and north abutment. The cross-sectional strain gauge configurations are shown in Figure 3.3. Figure 3.4 represents the time-history strain for all the girders at the mid-span bottom flange location. Figure 3.5 shows the strain at girder J having a strain gauge configuration of #3. The top flange strain and the strain on the bottom of the slab are very similar, indicating composite action between the girder and slab is taking place. The strains also indicate the natural axis is near the interface of the slab and girder. The full range of strain measured for all gauges can be seen in Table 3.1. The largest strain range was at the mid-span and bottom flange location with a range of 104 με.

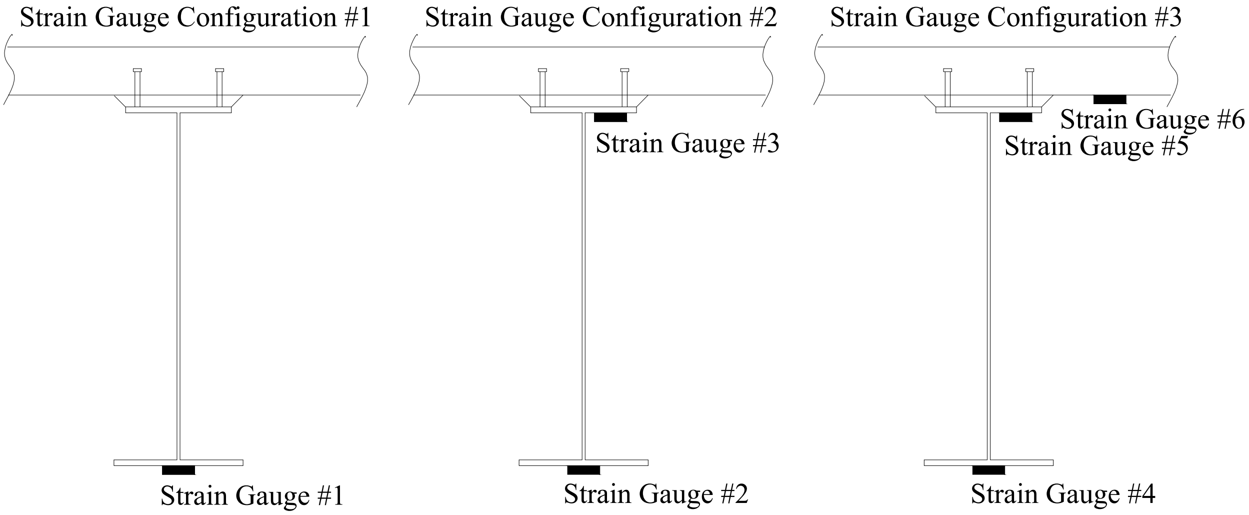

Strain Gauge Configuration #1
Strain Gauge Configuration #2
Strain Gauge Configuration #3
Strain Gauge #3
Strain Gauge #6
Strain Gauge #5
Strain Gauge #1
Strain Gauge #2
Strain Gauge #4

Figure 3.3. Cross-sectional strain gauge configuration

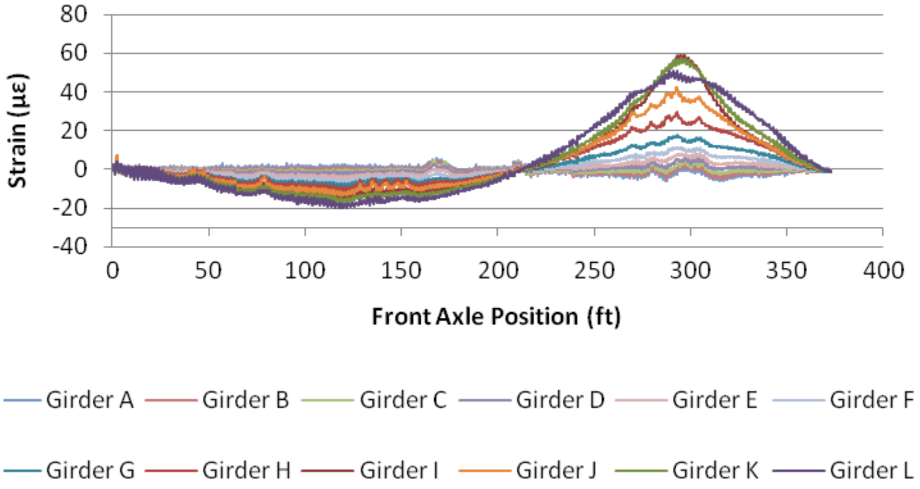


Figure 3.4. Representative time-history strain for mid-span bottom flange load location, Load Case 2

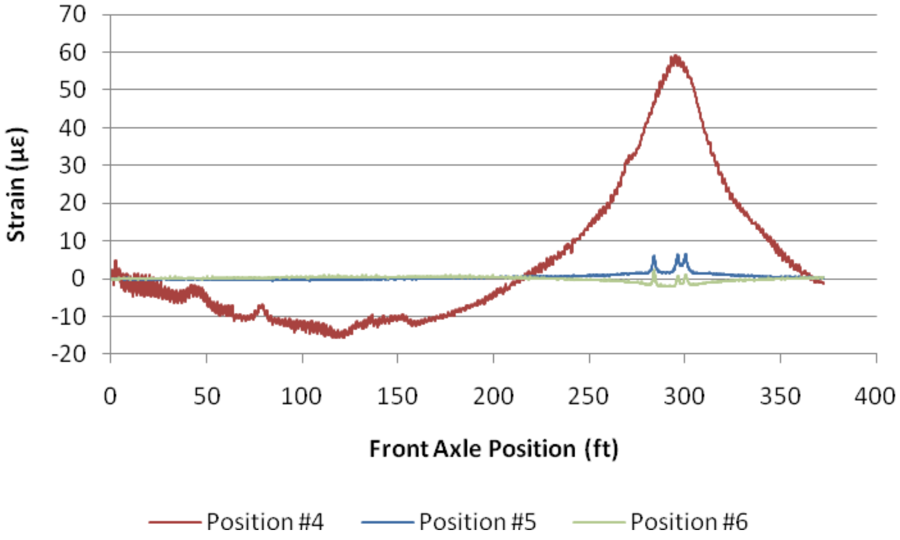


Figure 3.5. Girder J cross-sectional strain, Load Case 2

Table 3.1. Range of strain at various locations

	Strain (με)		
	Bottom Flange	**Top Flange**	**Bottom of Slab**
Abutment	-4 to +14	-5 to +6	NS
Pier	-16 to +3	-1 to + 5	NS
Mid-span	-22 to +66	-5 to + 5	-2 to + 6

NS—No strain gauge at location

3.3 Field Test Conclusion

The field testing of the 24th Street Bridge in Council Bluffs, IA, consisted of two components: evaluation of the handling performance of the precast deck panels and evaluation of the performance of the deck under semi-controlled loading conditions (including deflection and strain measurements). Based on the information obtained thus far from the field testing, the following conclusions were determined:

1. The prestressing and post-tensioning strands monitored for corrosion have indicated no active corrosion taking place.
2. The north span deflection of the bridge ranged from -0.41 in. to +0.21 in., which corresponds to a span to deflection ratio of L/5120
3. The strain over various location on the girders and slab ranged from -22 to +66 $\mu\varepsilon$, with the largest strain being located at the on the bottom flange of girder K.

Laboratory and Field Testing of Hycrete Corrosion-Inhibiting Admixture for Concrete

John Wojakowski
Hycrete, Inc.
6043 SW Urish Rd
Topeka, KS 66610
John@hycrete.com

Jennifer Distlehorst
Kansas Department of Transportation
2300 SW Van Buren St.
Topeka, KS 66611
JenniferD@ksdot.org

ABSTRACT

Hycrete anti-corrosion admixture was first tested by the University of Connecticut and later by the University of Massachusetts for the New England Transportation Consortium, a group of New England state departments of transportation. A variety of pozzolans and anti-corrosion admixtures were evaluated in these laboratory studies. Hycrete was found to have unique properties that greatly reduced both the penetration of chlorides into concrete and the corrosion of black steel reinforcing bars in cracked concrete specimens. It has been selected for field trials that are currently underway in the northeastern United States.

Based on this research, the Kansas Department of Transportation elected to evaluate Hycrete in the laboratory and in a structure.

The laboratory tests indicated that concrete with Hycrete admixture had properties similar to the control concrete, with the exception of permeability and strength. Water permeability was reduced significantly, and strength gain was slower and about 20% less than the control mixture at 28 days.

For the field evaluation, extra black reinforcing steel was placed in the shoulder concrete of two bridge decks that have been monitored for nearly five years. One bridge deck was constructed with Hycrete and the other with a standard mixture to serve as a control. Results from the fresh and hardening concrete were similar to those seen in the laboratory except for even lower early strength test results seen in the Hycrete concrete. An early crack survey showed total cracking to be comparable between the control bridge and the Hycrete bridge. Later surveys found more cracking in the Hycrete bridge in negative moment areas. Corrosion-potential readings and corrosion-rate measurements to date are both in the low range for both bridges.

Key words: admixture — chlorides—concrete — corrosion—water

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PROBLEM STATEMENT

The cost for annually repairing highway bridge corrosion damage in the United States is projected to be $8.3 billion (Koch et al. 2002). A significant portion is spent to repair the effects of corrosion of the reinforcing steel in concrete. New products and improved methods have been introduced to try to find a better, more economical solution to the problem of corrosion of the reinforcing steel in structural concrete. One new product is Hycrete DSS, an anti-corrosion admixture adapted from the automotive industry, where it was originally used in motor oil to prevent corrosion in engine cores.

In concrete, Hycrete reacts with metal ions to form an insoluble organic metallic salt, which makes the concrete highly resistant to the penetration of water and chlorides. Molecules of this salt have a long-chain hydrocarbon on one end that repels water and a polar end that also attaches to reinforcing steel and chain together, creating a layer that forms a protective film against chlorides, oxygen, and moisture. At the surface of the reinforcing steel, Hycrete reacts with ferrous ions and forms a stable layer. Ferrous ions would normally react with oxygen and form the rust that is destructive to the structure because of the loss of some of the cross section of the reinforcement and because of the spalling and cracking of the concrete that can occur from the pressure of the larger volume of the rust.

RESEARCH OBJECTIVES

Current technologies employed for corrosion protection of the reinforcing steel have been somewhat successful, but there is the ongoing quest for products that are better, faster, and more economical. As with any new product, a rather thorough evaluation is needed before it is put into widespread use to balance innovation with having a good degree of certainty in and experience with a new product. The investigations reported here were designed to evaluate the Hycrete anti-corrosion admixture in concrete using laboratory tests and a full-scale field project. These can serve as a basis for making the critical decision for wise use of the public’s investment in transportation structures.

PREVIOUS RESEARCH

University of Connecticut

Under a contract with the Connecticut Department of Transportation, the University of Connecticut conducted research on three different types of corrosion inhibitors in concrete mixtures (Goodwin, Frantz, and Stephens 2000). These were a calcium nitrate-based chemical (A), an organic-based chemical consisting of esters and amines (B), and two related salts of an organic alkenyl dicarboxylic acid. These two salts are called Hycrete DSS and Hycrete DAS in the report and had similar performance. The control concrete mixture (C) was based on Connecticut Department of Transportation specifications with a maximum water-cement ratio of 0.44 and a minimum cement content of 658 pounds per cubic yard. The calcium nitrite was dosed at 4 gallons per cubic yard, the organic amine at 1 gallon per cubic yard, and the organic salt from 1/8% to 2% by weight of the cement (about 0.5 gallons to 13 gallons per cubic yard).

Specimens with and without cracks were 3-inch diameter cylinders with a centered No. 4 reinforcing bar. Cracks were formed with a 1/8-inch wedge. The 3 x 6 in. concrete cylinders were submersed five inches into a 15% salt solution for four days, followed by a drying period of three days. Some specimens were also air dried. Linear polarization was the method used for monitoring the corrosion of the samples.

At the end of 100 weeks, chloride contents were measured at the surface of the concrete and the level of the steel for all mixtures. The Hycrete concretes had dosages of 1/2%, 1%, and 2%. Results are shown below in figure 1.

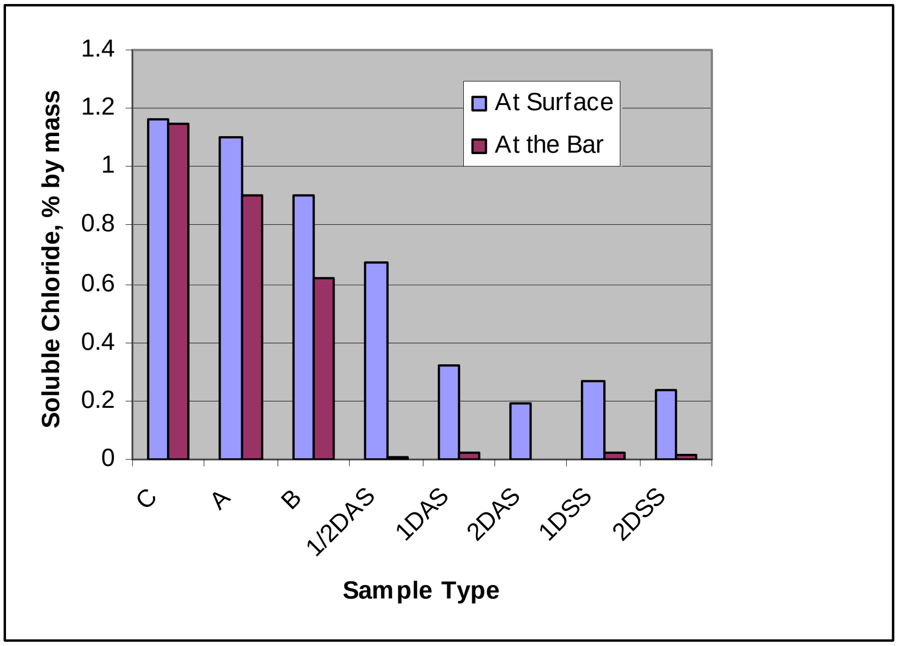

Figure 1. Soluble chlorides measured at the surface and at the level of the steel

For precracked specimens at 35 weeks, the Hycrete specimens generally were comparable to or better than inhibitor A and B (see Figure 2).

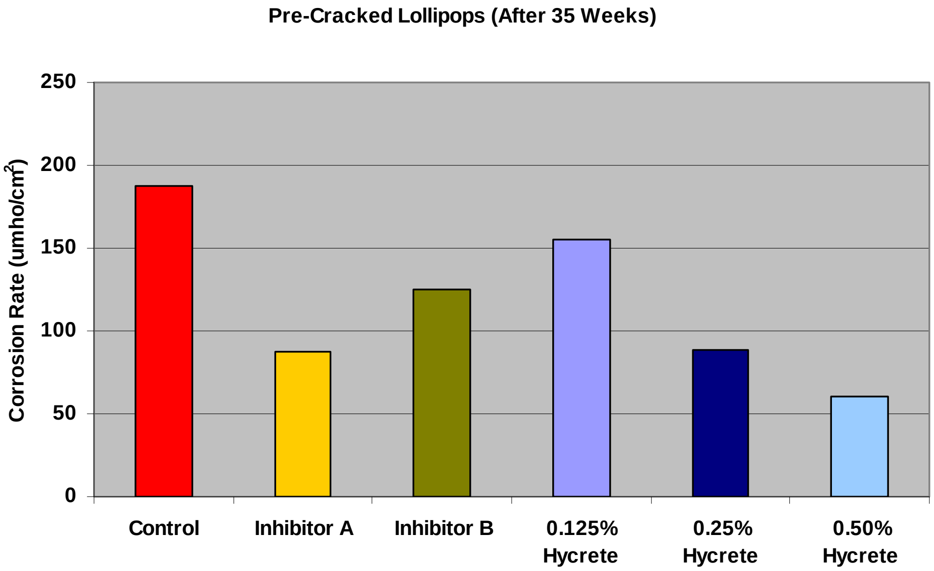

Figure 2. Corrosion rate of precracked specimens at 35 weeks

Conclusions in the report include, compared to inhibitor A and inhibitor B, the Hycrete chemicals (1) provided significantly more protection in cracked concrete (lower corrosion rates and lower corroded areas), (2) provided significantly better protection against chlorides penetration, (3) provided very good freezing and thawing protection, and (4) at a concentration of about 1/2% the compressive strength were still more than adequate for structural use.

University of Massachusetts

The University of Massachusetts conducted research for the New England Transportation Consortium, a group of New England state departments of transportation (Civjan and Crellin 2008). A number of combinations of admixtures and pozzolans were tested in a standard structural concrete mixture by ponding. These are listed below.

1. Control, w/c = 0.40
2. Calcium Nitrite, w/c = 0.40
3. 6% Silica Fume, w/c = 0.40
4. 15% Fly Ash, w/c = 0.40
5. 25% Slag, w/c = 0.40
6. **0.5% Hycrete DSS, w/c = 0.40**
7. Calcium Nitrite + Silica Fume, w/c = 0.40
8. Calcium Nitrite + Fly Ash, w/c = 0.40
9. Calcium Nitrite + Slag, w/c = 0.40
10. **Calcium Nitrite + Hycrete DSS, w/c = 0.40**
11. Calcium Nitrite + Silica Fume + Fly Ash, w/c = 0.40
12. Calcium Nitrite + Silica Fume + Slag, w/c = 0.40
13. Calcium Nitrite + Silica Fume, w/c = 0.47
14. Calcium Nitrite + Silica Fume + Fly Ash, w/c = 0.47

Slab specimens were pounded for three days and dried for four days for 12 weeks using a 15% NaCl solution. After that, they were continually ponded for 12 weeks. Specimens with 1 inch of clear cover were both intact and cracked to allow the salt solution direct access to the steel. Chloride penetration into the intact slabs after 204 weeks is shown in Figures 3, 4, and 5.

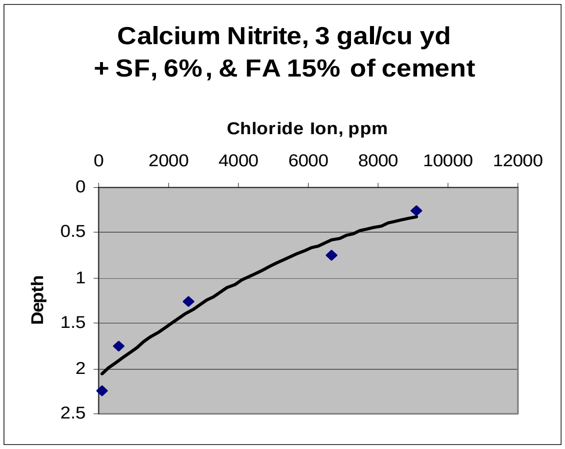

Figure 3. Control Mixture

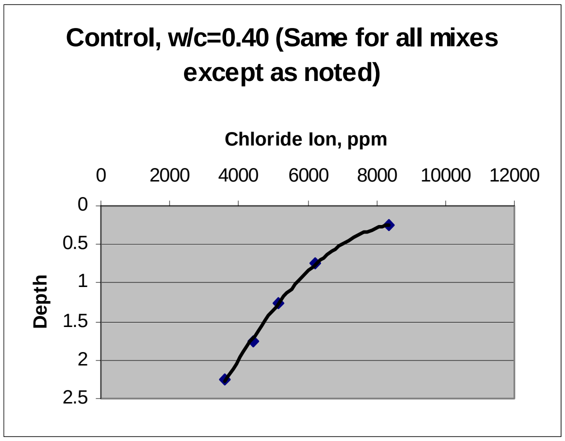

Figure 4. Calcium nitrite, silica fume, and fly ash

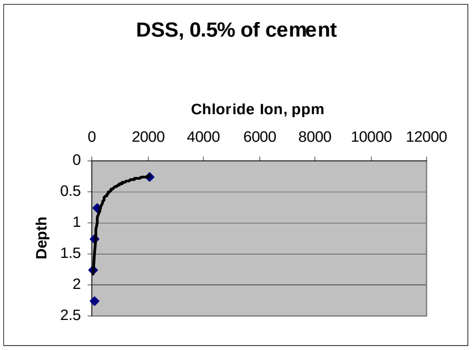

Figure 5. Hycrete

Iron loss was measured on all specimens for 102 weeks, except the Hycrete specimens (Nos. 6 and 10), which were measured at 204 weeks. Results are shown in Figure 6 below.

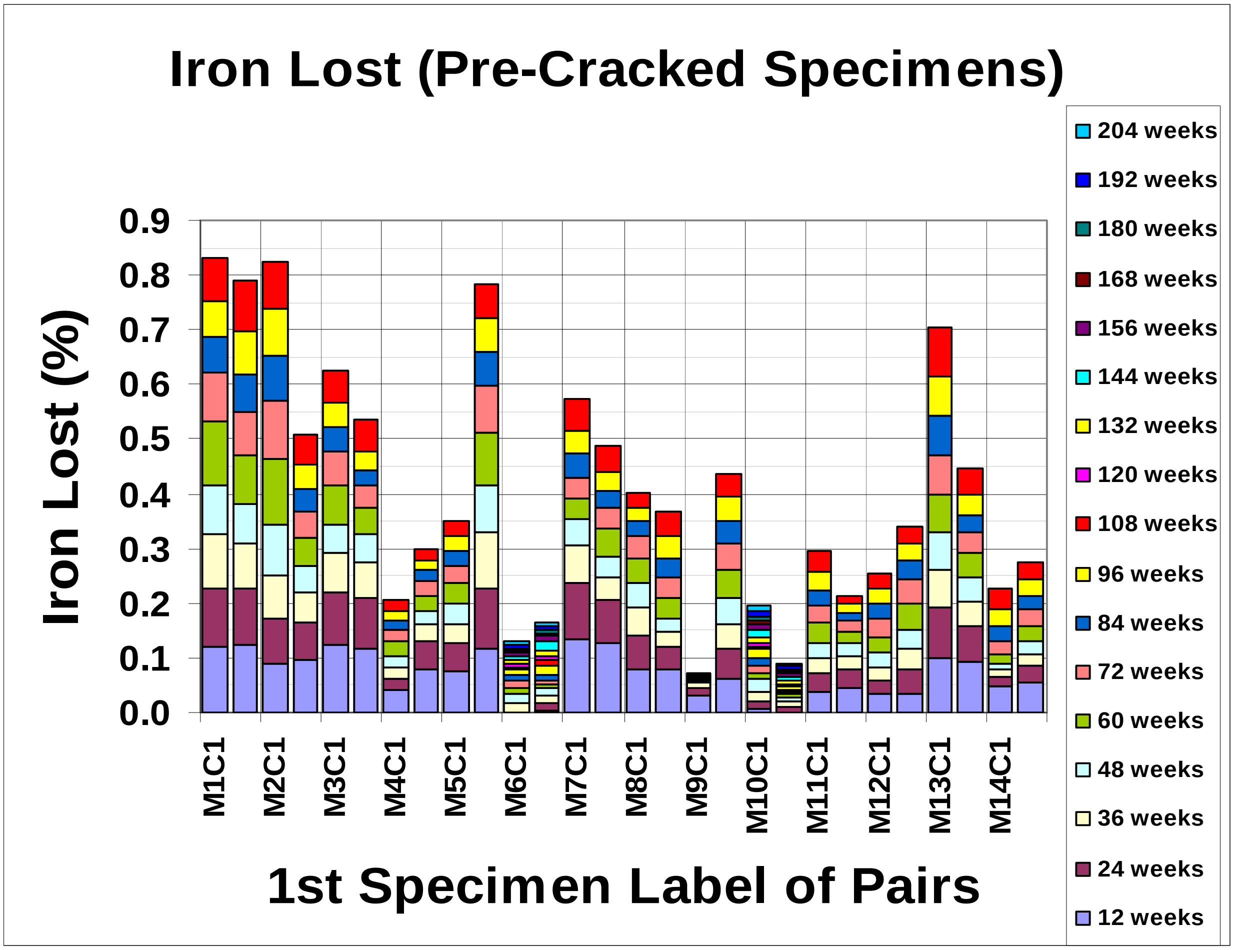

Figure 6. Iron loss in ponded cracked slabs

Hycrete was also sprayed on one cracked specimen, and the ongoing corrosion measured by macrocell (voltage across a resistor) was reduced significantly as shown in Figure 7.

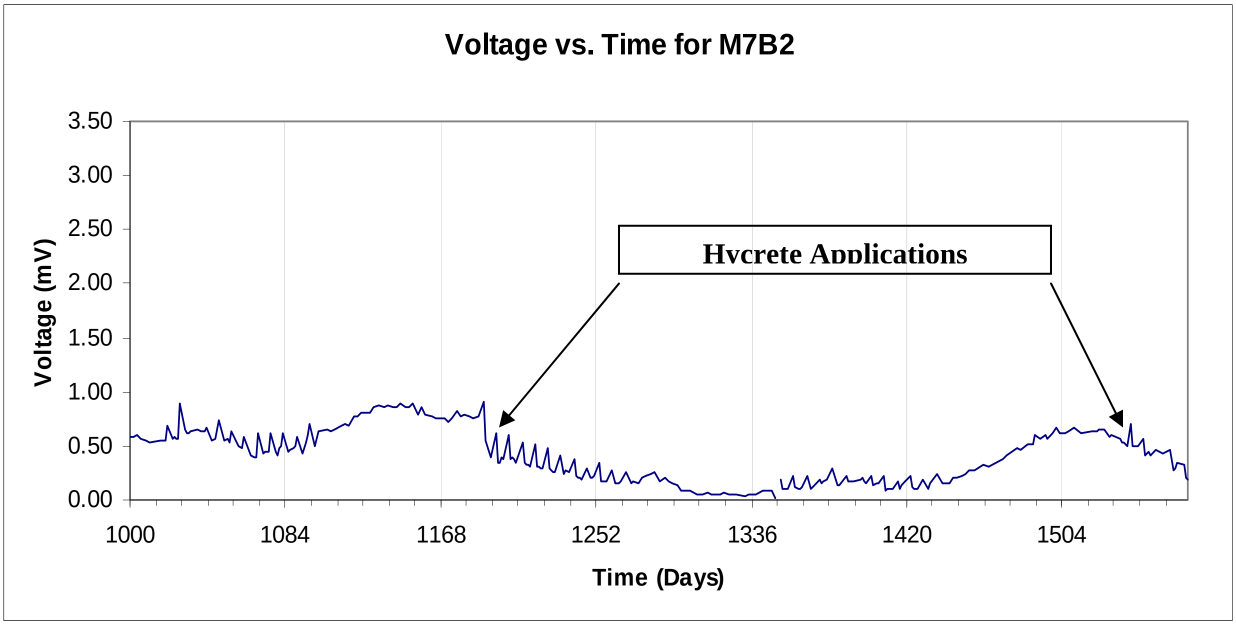

Figure 7. Effect of Hycrete admixture on corroding reinforcing steel

KANSAS DEPARTMENT OF TRANSPORTATION RESEARCH

Laboratory Testing

Methods

In 2003, the concrete research staff of the Kansas Department of Transportation (KDOT) investigated the physical effects of adding DSS corrosion inhibitor to a standard Kansas concrete mix. The following is a summary and some analysis of information given in the report of the investigation (Distlehorst and Wojakowski 2007). Four 1.70-cubic-foot batches of concrete were produced: a control mix, a mix with air-entraining admixture, a mix with Hycrete DSS, and a mixture with Hycrete DSS and a defoaming agent. See Table 1 for batch weights and proportions.

Table 1. Mix designs for four batches of concrete tested

Component	Batch Type			
	Control	AEA added	Hycrete DSS	Defoamed DSS
Cement (lb)	37.90	37.90	37.90	37.90
AEA (ml)	0	8.94	0	0
Hycrete (lb)	0	0	1.04	1.04
Defoaming agent (g)	0	0	0	4.0
Water (lb)	16.68	15.92	14.78	14.78
CA-6 (lb)	91.15	92.12	92.12	92.12
FA-A (lb)	91.15	92.12	92.43	92.43
Design w/c	0.44	0.42	0.42	0.42
Design cement content (lb/cu. yd)	602	602	602	602

An air-void analysis, total air content by volume, and slump were performed on the fresh concrete. Strength and two tests used as indicators of permeability were performed on hardened samples.

Results

The fresh concrete properties are shown in Table 2. Spacing factors and entrained air contents were obtained using the air-void analyzer.

Table 2. Results of testing of fresh concrete

Batch Type	average spacing factor (mm)	slump (in.)	total air content (%)	entrained air content (%)
Control	0.699	2.75	3.75	2.2
AEA added	0.140	2.75	8.75	6.2
Hycrete DSS	0.179	2.25	13.75	5.8
Defoamed DSS	0.213	2.50	6.40	4.5

Results of the tests on hardened concrete specimens are given as follows in Tables 3, 4, and 5.

Table 3. Average compressive strength test results from three samples

Results of Compressive Strength Tests (psi)				
Sample Age (days)	Control	AEA added	Hycrete DSS	Defoamed DSS
4	3,977	2,490	2,087	2,837
7	4,377	3,330	2,700	3,377
28	6,007	4,547	3,633	4,287
56	7,263	6,007	4,610	5,027

Table 4. ASTM C 1202 "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration," average results from two samples

ASTM C 1202 results (Coulombs)				
Sample Age (days)	**Control**	**AEA added**	**Hycrete DSS**	**Defoamed DSS**
7	7,645	6,681	13,648	5,075
28	5,769	4,784	3,983	3,970
56	3,679	2,596	3,148	2,332
90	4,053	1,435	2,948	1,803

Relative resistance to capillary flow of water through the concretes (Table 5) was measured using the Kansas Evapo-transmission Test (Jayaprakash et al. 1979), in which a desiccant on one side of a one-inch-thick sample draws water from the other side. Lower numbers are better.

Table 5. Average Evapo-transmission test results from two samples

Results of Evapo-transmission Permeability Tests (ml/min x 100,000)				
Sample Age (days)	**Control**	**AEA added**	**Hycrete DSS**	**De-foamed DSS**
7	65	130	30	15
28	45	25	25	8
56	45	50	35	35
90	80	95	77	30

Discussion

Since most Kansas structural concrete is air entrained (AE), a comparison of the Hycrete concrete with the defoamer to the air-entrained standard concrete would be useful. The air-entrained standard concrete, however, had about 2% more total air than the defoamed concrete and could be expected to be about 10% stronger at a comparable air content. Adjusting for that, a comparison of the relative strength of the defoamed Hycrete concrete to the standard AE concrete is shown below in Table 6.

Table 6. A comparison of expected strengths

Comparison of Compressive Strength Test Results, Actual and Adjusted				
Sample Age (days)	**Standard AE Concrete (psi)**	**Adjusted Strength of AE Concrete (psi)**	**Hycrete Concrete with Defoamer (psi)**	**Hycrete Concrete/AE Concrete (%)**
4 days	2,490	2,739	2,837	104
7 days	3,330	3,630	3,377	93
28 days	4,547	5,002	4,287	86
56 days	6,007	6,608	5,027	76

Somewhat comparable strengths are seen at young ages, but lower, longer age strength is indicated for the Hycrete concrete with defoamer. Its mature strength, however, would exceed the design strength of 4,000 psi. Other researchers that have investigated the effect of Hycrete on physical properties of concrete have noticed similar effects on the strength (Civjan et al. 2003).

For the defoamed Hycrete concrete, improved results from ASTM C 1202 “Standard Test Method for Electrical Indication of Concrete’s Ability to Resist Chloride Ion Penetration” were not expected since the Hycrete molecule has an electrically polar end that can facilitate, to some degree, the passage of electricity in moist concrete. Testing results bore that out.

The results from the Kansas Evapo-transmission Test, however, did show a good improvement for the defoamed Hycrete concrete. These results are from concrete that is saturated under the test conditions.

Field Placement of Two bridges

Control Bridge and Experimental Bridge with Hycrete in Elk County, Kansas

Two structures were used in this experiment in order to accurately assess corrosion in a structure. A single bridge could not be used because a structure that does not have consistent properties throughout (i.e., part with Hycrete DSS and part without) can develop a corrosion cycle in which the less-corroding part can drive the corrosion in the other part at a higher rate. Bridges were haunched slabs with a one-course deck. Data below is courtesy of KDOT.

Ideally, field corrosion tests should provide early indication of long-term performance; thus, this experiment used some additional black steel reinforcing bars in a bridge deck to test the anti-corrosion capabilities of Hycrete DSS. To allow chloride ions to reach the steel bars in a relatively short time, both bridges had additional black steel bars placed at a clear spacing of one inch (25 mm) below the top of the wearing surface. Two black steel bars spaced at 18 inches (450 mm) in the top and bottom of the slab in the shoulder area are connected electrically by bare steel stirrup bars (see Figure 8).

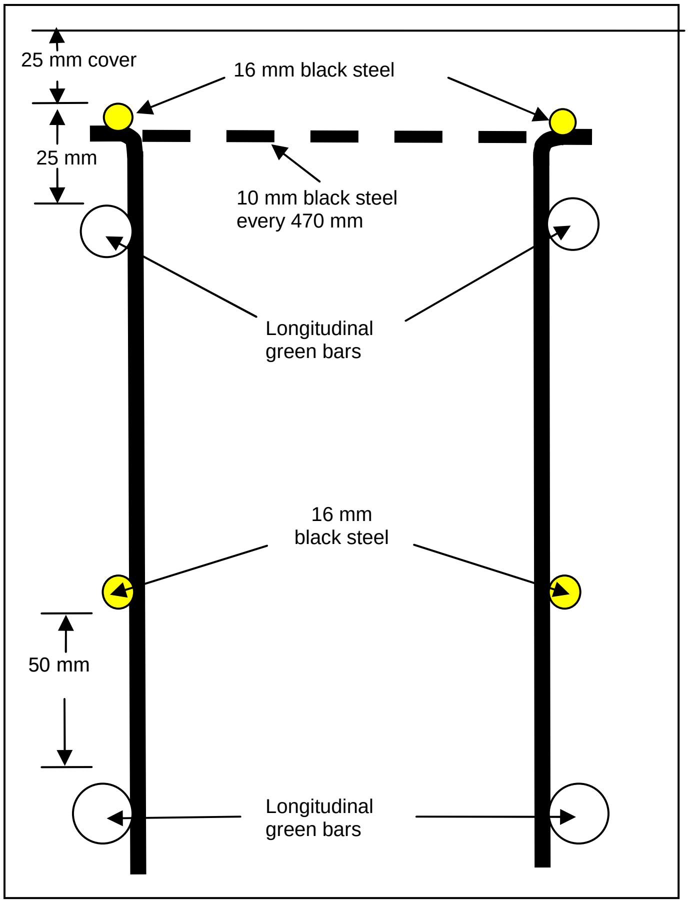

Figure 8. Schematic drawing of black steel placement

Experimental Bridge with Hycrete, Elk County

Bridge number 99-25 09.07(035) on Kansas Highway 99, south of Howard, Kansas, was selected for this experimental program. This bridge is a continuous haunched slab with spans of 13.5, 18, and 13.5 m over Mound Branch Creek, a small stream.

The bridge deck placement began early in the morning of August 25, 2004, at the north end of the bridge and proceeded somewhat slowly, though smoothly, into the early afternoon. Temperatures began in the 60s and ended in the 80s. Evaporation rates started very low and ended at about 0.10 lb/ft^2/hr. The weather was cloudy and humid most of the time, but about 8:00 a.m. there were some sprinkles of rain and about 10:00 a.m., the sky cleared briefly and the sun shone.

One of the early loads in the bridge deck was a bit stiff with a slump of two inches. The stickiness of this mixture made the finishing difficult, and the slumps of following loads were raised with the addition of more defoaming agent and small adjustments in the water content. The majority of the deck was placed at a six-inch slump. Although this is higher than normally used in Kansas bridge decks, the cohesiveness of the mixture from the Hycrete additive made this a very good mix to place and finish. No segregation was evident during placement.

Pumping of the six-inch slump concrete through a five-inch pump line gave a smooth stream of concrete. The two-inch slump concrete came out of the pump in discrete sections. Initial curing with a fogging system began immediately after placement.

Control Bridge, Elk County

The first half of the control bridge, consisting of the north bound lane and shoulder of bridge 99-25 12.72 (036), just north of Howard, Kansas, on Kansas Highway 99, was placed on September 2, 2004. This bridge is also a continuous haunched slab with spans of 15, 20, and 15 m over Paw Paw Creek, another small stream. Placement of the concrete via pump began about 5:00 a.m. at the north end of the bridge and concluded about 11:00 a.m. The pump truck was moved to the south end of the bridge to complete the placement. The 3.5- to 4-inch slump concrete tended to come out in discrete sections throughout most of the placement. The slightly higher temperatures, lower humidity, and higher wind speed contributed to overall higher evaporation rates than those experienced on the Hycrete bridge. Initial curing with a fogging system began immediately after placement.

Field Concrete Properties

Concrete was tested from each bridge for slump, unit weight, total air content, temperature, bleed rate, and air-void parameters by an air-void analyzer. Air temperature, humidity, and wind speed were measured, and an evaporation rate was calculated for the freshly placed concrete. Cylinders were made to check compressive strength and to use in testing by ASTM C 1202, “Standard Test Method for Electrical Indication of Concrete’s Ability to Resist Chloride Ion Penetration.” Maturity meters recorded the temperature of the concrete over the course of several days. The results of most of these tests are shown in Tables 7 through 10 and Figure 9 below.

Table 7. Construction testing results for concrete from Hycrete bridge

Time (AM)	Location	Spacing Factor (mm)	Unit Mass (kg/cu. m)	Slump (mm)	Air Content (%)	Air Temp (°F)	Concrete Temp (°F)	Evaporation rate (lb/ft^2/hr)	Bleed rate (lb/ft^2/hr)
6:00	North End	0.312	1910	152	7.6		86		0.02
8:35	Middle	0.227	1928	121	6.8		86		0.01
10:06						83.6	87	0.007	
11:20	South End	0.237	1890	165	8.0		90		0.00

Table 8. Construction testing results for concrete from control bridge

Time (AM)	Location	Spacing Factor (mm)	Unit Mass (kg/cu. m)	Slump (mm)	Air Content (%)	Air Temp (°F)	Concrete Temp (°F)	Evaporation rate (lb/ft^2/hr)	Bleed rate (lb/ft^2/hr)
8:05	North End	0.412	1,944	108	6.5		86		0.00
10:35	South End	0.429	2,245	85	6.8	86.7	91	0.11	

Table 9. Results of laboratory testing on field concrete cylinders

	Compressive strength (psi)			ASTM C 1202 (Coulombs)	
Age of sample (days)	**14**	**28**	**56**	**28**	**56**
Hycrete Bridge #035	3,617	4,148	5,140	5,510	3,864
Control Bridge #036	5,448	6,494	6,850	6,325	3,954

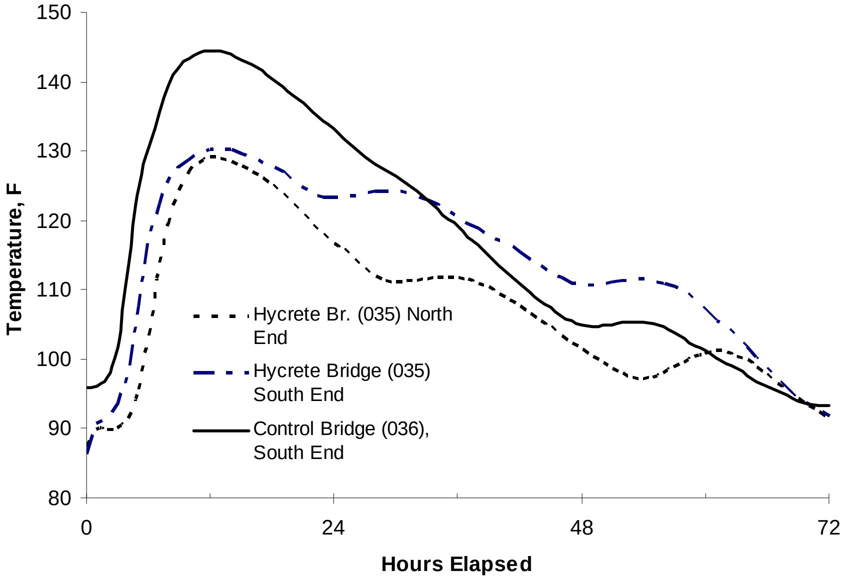

Figure 9. Internal concrete temperature vs. time

Survey Results of Bridges

In December 2004, approximately four months after the construction of these two bridges, research personnel returned to measure the corrosion potential of the black steel in each bridge. Neither bridge had ever been open to traffic at that time. The surface cracks were also measured and reported as a percent of black bars length (designed to have a 25 mm cover of concrete). The results of the corrosion potential testing and the crack survey are shown in Figure 10 and Table 10, respectively. The average depth to the black bars in the Hycrete bridge is more than that of the control bridge. The median depth (for the first 50% of the bars) in the Hycrete Bridge is about 45 mm, while the median depth in the control bridge is about 35 mm.

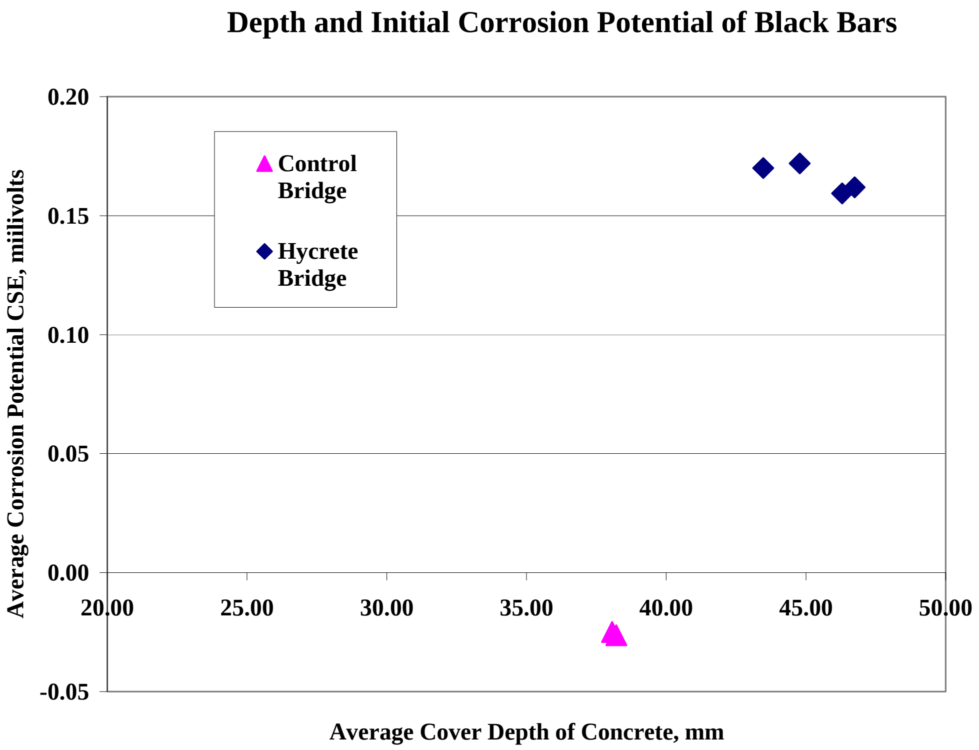

Figure 10. The average depth of concrete over the individual bars and their average initial corrosion potential

Table 10. Cracking in length of cracks as a percent of the length of the black bars

	Date		
	3/2005		**10/2006**
	Total Cracks (%)	**Longitudinal Cracks (%)**	**Longitudinal Cracks (%)**
Hycrete Bridge #035	14.4%	3.5	3.5
Control Bridge #036	12.5%	11	16

Recently, crack surveys have recorded a substantial increase in the transverse cracks in the Hycrete bridge. These cracks are primarily located over the piers in the negative moment areas. A core taken from this area showed cracking around the coarse aggregate. Information from the project office indicated that side forms were removed when the concrete was three days old. Interpolations of the strength tests indicate that the concrete strength might have been less than 2,000 psi. Additionally, forms were placed for the rail before the concrete may have attained 3,000 psi. These strengths are lower than had been expected from the laboratory testing. The location of these cracks, the strength data, and the core showing the lack of bond to the paste strongly suggest that these cracks are related to early overloading of the structure. The cracking in both bridges may prove to be advantageous for research by allowing chloride ions quicker access to the bars, and thereby shortening the time to observe active corrosion for these areas.

Recently measured corrosion rates are 1.27 micrometer per year for the control bridge and 0.91 micrometer per year for the Hycrete bridge, indicating that active corrosion is yet to occur.

CONCLUSIONS

1. Hycrete corrosion-inhibiting admixture makes concrete very impermeable to chloride ions under wetting and drying conditions.
2. Under moist conditions, Hycrete corrosion-inhibiting admixture is still effective in reducing water flow through the concrete.
3. Hycrete corrosion-inhibiting admixture greatly reduces iron loss in notched specimen testing.
4. No active corrosion of black steel bars in the control or the Hycrete.bridge has yet occurred after four winters exposure to salt.
5. Higher levels of longitudinal cracking seen in the control bridge appear to be due to the low cover specified for the experiment.
6. Higher levels of transverse cracking seen in the Hycrete bridge appear to be related to slower strength gain and early loading of the structure.
7. Continued close monitoring of the bridges should begin to provide answers as to the effectiveness of the Hycrete anti-corrosion admixture.

OTHER FIELD RESEARCH PROJECTS

Other states and agencies have also begun to use Hycrete in various experimental projects. Among these are the states of New Jersey and Ohio. The United States Army Corps of Engineers is currently beginning a project to evaluate Hycrete.

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Development of a Freight Analysis Framework for the Kansas City Metropolitan Area

Erin Wurfel
Transportation Research Institute
University of Kansas
1530 W. 15th Street, Learned Hall
Lawrence, KS 66045
erin.wurfel@gmail.com

Yong Bai
Department of Civil, Environmental, and Architectural Engineering
University of Kansas
1530 W. 15th Street, 2150 Learned Hall
Lawrence, KS 66045
ybai@ku.edu

Luke Huan
Department of Electrical Engineering and Computer Science
University of Kansas
1520 W. 15th Street, 2034 Eaton Hall
Lawrence, KS 66045
jhuan@ittc.ku.edu

Vincent Buhr
Department of Electrical Engineering and Computer Science
University of Kansas
1520 W. 15th Street, Eaton Hall
Lawrence, KS 66045
vinveb7@gmail.com

ABSTRACT

Freight transportation is the backbone of the United States economy and is critical for the daily operations of every business in the United States. Because of the United States' dependency on freight transportation, it is vital for transportation decision makers to properly maintain the transportation infrastructure to meet the growing demand for freight capacity. In order for these decision makers to identify problem areas for freight transportation, the Federal Highway Administration (FHWA) created the Freight Analysis Framework (FAF). The major constraint of the FAF is that it only concentrates on the major metropolitan areas of the United States and ignores smaller metropolitan areas. The objective of this research was to create a freight analysis framework for the Kansas City metropolitan area so that the state government agencies will be able to properly plan for future increases in freight traffic, and identify current issues and future trends regarding freight transportation. The objective was accomplished through a four-step approach including literature review, data collection, data analysis, and development of the Kansas Freight Analysis Framework (KFAF). The KFAF is a commodity-destination database that estimates tonnage and value of goods shipped by the type of transportation modes. In addition, it gives projections and forecasts of commodities in the Kansas City area. There are 43 commodities ranging from live animals to furniture and even electronics. The transportation modes include truck, rail, air, water, truck & rail, air & truck, and other intermodal. The framework of the developed KFAF is easily adaptable and can be used to develop a freight analysis model for other cities in the United States once reliable freight data become available.

Key words: commodity—framework—freight—mode—truck

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INTRODUCTION

Freight transportation is the backbone of the United State's economy and is critical for the daily operations of every business in the United States. Therefore, it is vital that there be a reliable freight transportation system. In order for decision makers to identify problem areas in freight transportation, the Federal Highway Administration (FHWA) created the Freight Analysis Framework (FAF). FAF is a database of major metropolitan areas and county-to-county freight flows over the national highway, railroad, water, pipeline, and air freight networks (FHWA 2006). However, the major constraint of the FAF is that it only concentrates on the major metropolitan areas of the U.S. and ignores smaller areas (FHWA 2004) such as Kansas City, Wichita, and Topeka, KS. This research paper explains the methodology, data collection, and development of the Kansas Freight Analysis Framework (KFAF).

RESEARCH OBJECTIVES AND SCOPE

The objective of this research is to develop a KFAF for the Kansas City Metropolitan area in order to identify major freight corridors and connectors, and collect data that will be important in creating a long-range freight transportation plan. The scope includes the nine-county metropolitan area of Kansas City. These counties are Cass, Clay, Jackson, Platte, and Ray in Missouri; and Johnson, Leavenworth, Miami, and Wyandotte in Kansas.

RESEARCH METHODOLOGY

The methodology consists of a literature review on common practices used to transport freight. Then data were collected from the 2002 and 2007 FAF, as well as from the Kansas Department of Transportation (KDOT) and Missouri Department of Transportation (MODOT). Data collected includes freight shipments in weight and value by origin/destination, destination/origin, commodities, as well as by each mode (highway, rail, water, air, pipeline, and intermodal). A summary of the data is shown in Table 1. After the data collection, the data were analyzed to create the KFAF. Lastly, conclusions and recommendations were given to help KDOT in implementing the framework.

Table 1. Freight shipments to, from, and within Kansas City metropolitan area in 2002 and 2007

	Tons (thousands)		Value (millions $)	
	2002	2007	2002	2007
Metropolitan Total	326,321	324,908	235,617	259,606
By Mode				
Truck	231,072	255,102	167,572	184,683
Truck & Rail	324	316	1,333	1,327
Air & Truck	29	43	1,207	1,697
Rail	34,446	38,959	26,425	27,118
Water	559	568	20	22
Other Intermodal	632	652	17,582	15,493
Pipeline & Unknown	59,260	29,267	21,479	29,267

KANSAS FREIGHT ANALYSIS FRAMEWORK DEVELOPMENT

Structure

The KFAF was developed as an online database. It can be used to estimate tonnage and value of goods shipped by type of commodity and mode of transportation. There are 43 commodities ranging from live animals/ fish to furniture and even electronics. The modes include truck, truck and rail, air and truck, rail, water, other intermodal, and pipeline & unknown (FHWA 2006). The KFAF can also show the through traffic in the Kansas City Metropolitan Area.

Assumptions

To develop the KFAF, a few assumptions were made. When converting commodity tonnage to trucks, it is assumed that every truck is a Class 5 truck according to the FHWA Vehicle Groups. Class 5 includes Truck/Tractor Trailers with five axles. By using the payload by commodity, it is assumed the payloads are the same across all states. When allocating trucks to the highways to determine through traffic, the assumption is that only the major highways are used from each direction. This is because the major highways are most likely faster than the smaller one-lane highways with lower speed limits. The following modes were classified as trucks for the KFAF: truck, truck and rail, and air and truck. The 2007 FAF data were used to calculate the conversion of commodity tonnage to trucks and the through traffic for the KFAF.

Conversion of Commodity Tonnage to Trucks

The number of trucks was calculated by converting the number of tons of freight into pounds and dividing it by the Class 5 average payload of each commodity from the FHWA Vehicle Class VIUS. The payloads for each commodity can be found online at: http://ops.fhwa.dot.gov/freight/freight_analysis/faf/faf2_reports/reports9/s513_14_15_tables.htm#_Toc16.9399567. The definition of trucks used for conversion included the modes Truck, Air and Truck, and Truck and Rail, all from the 2007 FAF data. The following are the classes used in the original FAF along with their definitions:

Class 1—Single Unit: 2-axle
Class 2—Single Unit: 3-axle
Class 3—Single Unit: 4-axle or more
Class 4—Truck/Tractor Trailers: 4-axle or less
Class 5—Truck/Tractor Trailers: 5-axle
Class 6—Truck/Tractor Trailers: 6-axle or more
Class 7—Combination Trucks: 5-axle or less
Class 8—Combination Trucks: 6-axle
Class 9—Combination Trucks: 7-axle or more
(FHWA 2007)

An example of the calculation formula is shown in Figure 1.

$$\frac{\text{\# of Tons X 1000 tons X 2000 lbs/1 ton}}{\text{Commodity Payload for Class 5}} = \textbf{Number of Trucks}$$

Example for Live Animals and Fish

$$\frac{3 \text{ X } 1000 \text{ X } 2000/1}{41{,}627} = 144 \text{ trucks}$$

Figure 1. Formula for converting commodity tonnage to trucks

The total number of trucks with a destination of Kansas City metropolitan area was 5,648,558 in 2007, while the total number of trucks with an origin of Kansas City metropolitan area was slightly higher with 5,697,096 trucks in 2007.

Allocation of Trucks to Highways

Trucks were allocated to the major Kansas City highways including I-70, I-35, I-29, and I-71 based on the direction the trucks are going to and from Kansas City. Some of the states were split between two directions; therefore, half of the trucks were used for each direction. The highway distributions to Kansas City from the North, South, East, and West are shown in Tables 2 and 3. The distributions from Kansas City are shown in Tables 4 and 5.

Table 2. Highway distributions to Kansas City metropolitan area from the north/south

Highway Distributions To Kansas City from the North using I-29 & I-35 South	
Locations	Number of Trucks
Alaska	23
Canada	17,248
Idaho 1/2	851
Illinois 1/2	61,227
Iowa	98,457
Minnesota	19,765
Montana	1,167
Nebraska	43,254
North Dakota	1,078
Oregon 1/2	693
South Dakota	4,384
Washington 1/2	1,244
Wisconsin	15,608
Wyoming 1/2	1,464
Total	**266,462**

Highway Distributions To Kansas City from the South using I-35 and 71 North	
Locations	Number of Trucks
Arizona	797
Arkansas	77,784
Kansas 1/2	1,243,994
Louisiana	24,757
Mexico	6,288
Mississippi 1/2	3,470
New Mexico	2,091
Oklahoma	57,226
Texas	79,503
Americas	7,349
Total	**1,503,259**

Table 3. Highway distributions to Kansas City metropolitan area from the west/east

Highway Distributions To Kansas City from the East using I-70 West	
Locations	Number of Trucks
Alabama	13,019
Connecticut	953
DC	47
Delaware	876
FL	6,597
Illinois 1/2	61,227
GA	13,161
Indiana	45,414
Kentucky	26,251
Maine	6,397
Massachusetts	1,091
MD	3,255
Michigan	48,915
Mississippi 1/2	3,470
Missouri	2,159,633
NC	11,048
New Hampshire	1,665
NJ	5,906
NY	29,245
Ohio	52,418
Penn	19,674
Rhode Island	2,650
SC	6,872
Tenn	20,108
VA	5,420
VT	852
W. VA	548
Asia and Europe 1/2	7,123
Total	**2,553,836**

Highway Distributions To Kansas City from the West using I-70 East	
Locations	Number of Trucks
CA	19,087
Colorado	48,544
Hawaii	0
Idaho 1/2	851
Kansas 1/2	1,243,994
Oregon 1/2	693
Utah	1,141
Nevada	284
WA 1/2	1,244
Wyoming 1/2	1,464
Asia and Europe 1/2	7,123
Total	**1,324,425**

Table 4. Highway distributions from Kansas City metropolitan area to the north/south

Highway Distributions From Kansas City To the North using I-29 & I-35 North	
Locations	Number of Trucks
Alaska	193
Canada	23,702
Idaho 1/2	821
Illinois 1/2	36,459
Iowa	78,810
Minnesota	24,133
Montana	1,629
Nebraska	104,338
North Dakota	442
Oregon 1/2	1,510
South Dakota	9,677
Washington 1/2	1,314
Wisconsin	14,314
Wyoming 1/2	2,324
Total	**299,666**

Highway Distributions From Kansas City to the South using I-35 and 71 South	
Locations	Number of Trucks
Arizona	8,205
Arkansas	147,328
Kansas 1/2	1,121,290
Louisiana	14,821
Mexico	8,657
Mississippi 1/2	7,310
New Mexico	2,861
Oklahoma	99,248
Texas	133,690
Americas	4,600
Total	**1,548,009**

Table 5. Highway distributions from Kansas City metropolitan area to the east/west

Highway Distributions From Kansas City to the East using I-70 East	
Locations	**Number of Trucks**
Alabama	34,082
Connecticut	1,753
DC	351
Delaware	310
FL	16,485
Illinois 1/2	36,459
GA	15,067
Indiana	31,760
Kentucky	9,068
Maine	964
Massachusetts	2,096
MD	4,040
Michigan	9,534
Mississippi 1/2	7,310
Missouri	2,347,495
NC	19,139
New Hampshire	3,430
NJ	7,059
NY	5,145
Ohio	18,114
Penn	13,223
Rhode Island	595
SC	2,595
Tenn	24,738
VA	4,451
VT	57
W. VA	5,220
Asia and Europe 1/2	6,547
Total	**2,627,088**

Highway Distributions From Kansas City to the West using I-70 West	
Locations	**Number of Trucks**
CA	32,848
Colorado	35,471
Hawaii	0
Idaho 1/2	821
Kansas 1/2	1,121,290
Oregon 1/2	1,510
Utah	13,466
Nevada	1,953
WA 1/2	1,314
Wyoming 1/2	2,324
Asia and Europe 1/2	6,547
Total	**1,217,545**

The total number of trucks allocated from each direction is summarized in Tables 6 and 7**.**

Table 6. Highway distributions from Kansas City metropolitan area

Highway Distributions From Kansas City	
From KC to the East using I-70 East	2,627,088
From KC To the North using I-29 & I-35 North	299,666
From KC to the South using I-35 and 71 South	1,548,009
From KC to the West using I-70 West	1,217,545

Table 7. Highway distributions to Kansas City metropolitan area

Highway Distributions To Kansas City	
To KC from the East using I-70 West	2,553,836
To KC from the North using I-29 & I-35 South	266,462
To KC from the South using I-35 and 71 North	1,503,259
To KC from the West using I-70 East	1,324,425

Through Trucks

The through traffic is calculated by adding the number of trucks into Kansas City and the number of trucks out of Kansas City, then subtracting this number from the truck counts given by KDOT and

MODOT. The total through traffic is found to be 23,158,050 trucks. Table 8 shows the calculated through traffic per year along with the highway distributions of trucks to and from Kansas City Metropolitan Area.

Table 8. Kansas City metropolitan area through traffic per year

Intersection	Trucks from KDOT/MODOT	Trucks to KC + Trucks from KC	Through Traffic Per Year
	A	B	A-B
I-70E before K-7 (West of KCK)	10,475,500	2,541,971	7,933,529
I-35N at Miami Co. Line & I-71N at Cass Co. Line (South of KC)	12,373,865	3,051,268	9,322,597
I-29S at Platte Co. Line & I-35S at Clay Co. Line (North of KC)	6,440,425	566,128	5,874,297
I-70W at Jackson Co. and Lafayette Co. Borders (East of KC)	5,208,550	5,180,924	27,626
Total	34,498,340	11,340,290	**23,158,050**

Projections

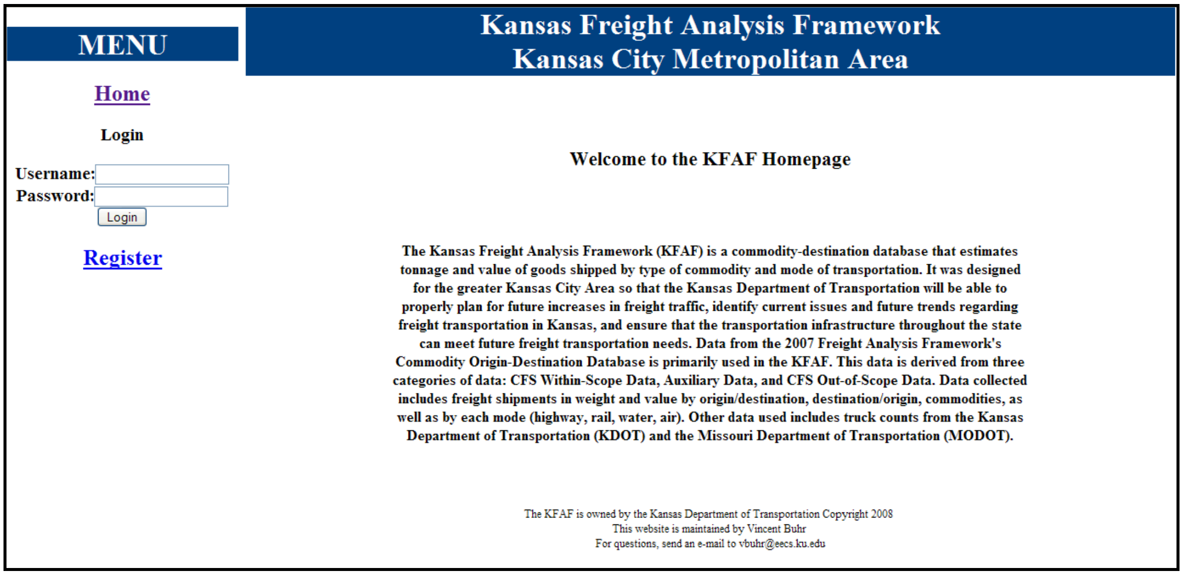
Two projections/ forecasts methods were developed. One method allows a user to enter a percentage increase and another utilizes the 2002 and 2007 FAF data. In the first method, the KFAF user is able to enter a percent increase or decrease for all commodities or select different percentages for up to four commodities. Then, commodity, mode, or truck traffic views are shown in a single table. In the second method, the 2002 and 2007 FAF data is used to find an average increase for one year. Then the years 2011, 2013, and 2018 are forecasted.

User Manual

The KFAF is a web-accessible, commodity-destination database that allows registered users to quickly view collected data from past years along with estimations of future shipments to and from the greater Kansas City Metropolitan Area. The KFAF website is currently being hosted at www.ittc.ku.edu/~vbuhr/kfaf2.html. It contains data from the 2002 and 2007 versions of the FAF, which can be found online at http://ops.fhwa.dot.gov/freight/freight_analysis/faf/index.htm. The following information highlights the central features of the KFAF and goes into detail on its technical implementation.

Features

At the introduction page of the KFAF website, a user sees a paragraph introducing KFAF. At the right part of the page is a panel for user login/registration. This page is shown in Figure 2. Only registered users may have access to contents of the KFAF website. New users may register themselves using the registration panel and the system administrator needs to approve new registered users before the users are allowed to log in.



Kansas Freight Analysis Framework
Kansas City Metropolitan Area

Welcome to the KFAF Homepage

The Kansas Freight Analysis Framework (KFAF) is a commodity-destination database that estimates tonnage and value of goods shipped by type of commodity and mode of transportation. It was designed for the greater Kansas City Area so that the Kansas Department of Transportation will be able to properly plan for future increases in freight traffic, identify current issues and future trends regarding freight transportation in Kansas, and ensure that the transportation infrastructure throughout the state can meet future freight transportation needs. Data from the 2007 Freight Analysis Framework's Commodity Origin-Destination Database is primarily used in the KFAF. This data is derived from three categories of data: CFS Within-Scope Data, Auxiliary Data, and CFS Out-of-Scope Data. Data collected includes freight shipments in weight and value by origin/destination, destination/origin, commodities, as well as by each mode (highway, rail, water, air). Other data used includes truck counts from the Kansas Department of Transportation (KDOT) and the Missouri Department of Transportation (MODOT).

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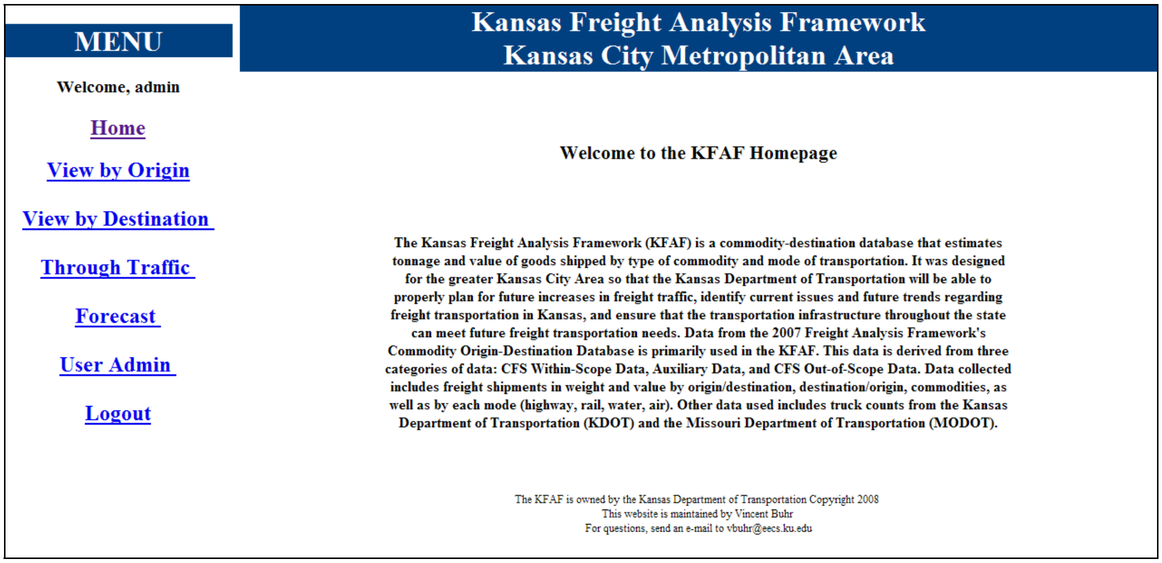
This website is maintained by Vincent Buhr
For questions, send an e-mail to vbuhr@eecs.ku.edu

Figure 2. KFAF homepage

Once users have logged in, they have access to a menu on the left side of the screen that allows them to navigate between KFAF features as shown in Figure 3. First off, they can view data on shipments to or from Kansas City based on the type of commodity shipped, mode of shipment, or an estimation of the number of trucks used in shipping. The user selects the destination/origin state first and then city, the areas available for selection are taken from the FAF data as shown in Figure 4.



Kansas Freight Analysis Framework
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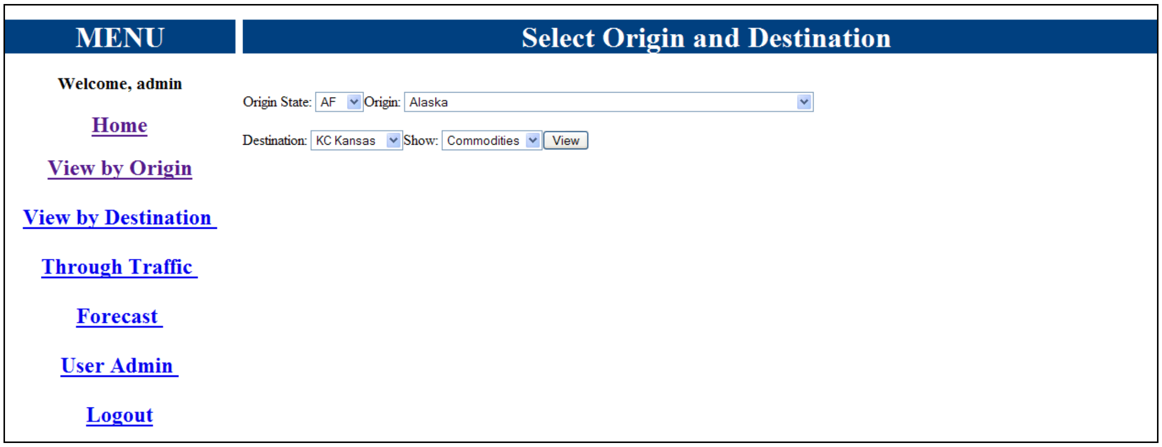
For questions, send an e-mail to vbuhr@eecs.ku.edu

Figure 3. Navigation menu

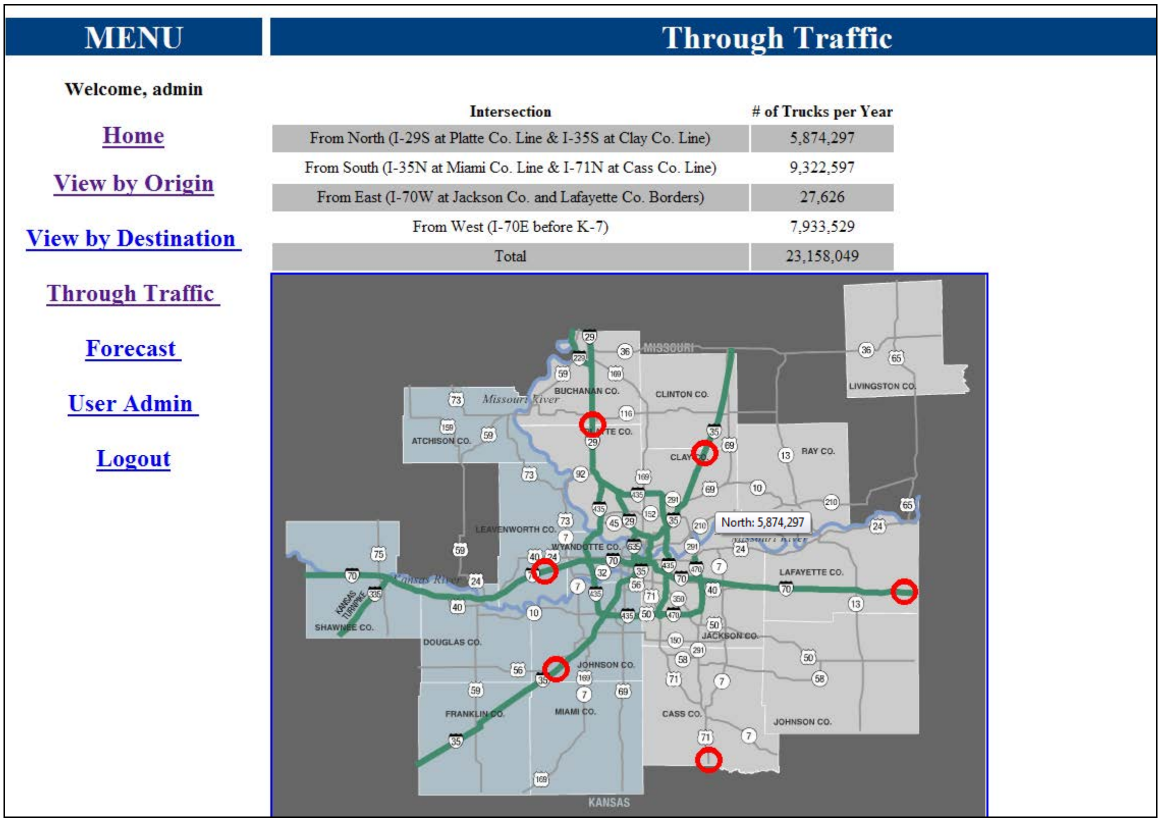

MENU
Select Origin and Destination
Welcome, admin
Home
View by Origin
View by Destination
Through Traffic
Forecast
User Admin
Logout
View

Origin State: AF Origin: Alaska

Destination: KC Kansas Show: Commodities

Figure 4. Origin/destination selection screen

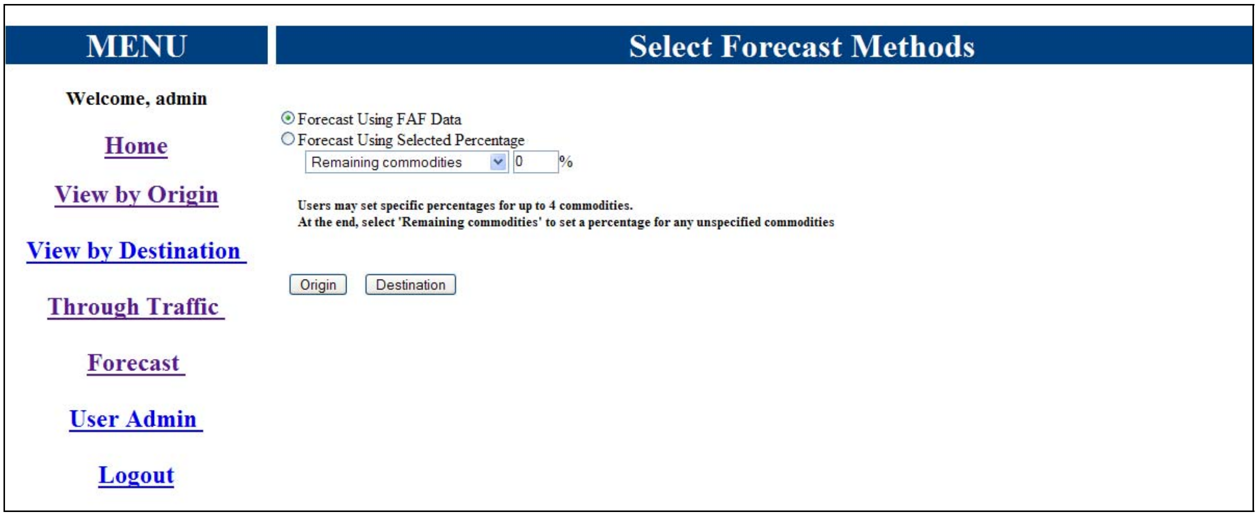
The next selection in the navigation menu is “Through Traffic” which shows a table of the estimated number of trucks that pass through Kansas City from each direction on their way to other cities. Additionally, there is a map of the Kansas City area that allows the user to mouse over the applicable intersections to view the truck counts shown in Figure 5.


MENU
Through Traffic
Welcome, admin
Home
View by Origin
View by Destination
Through Traffic
Forecast
User Admin
Logout
MISSOURI
BUCHANAN CO.
CLINTON CO.
LIVINGSTON CO.
ATCHISON CO.
CLAY CO.
RAY CO.
LEAVENWORTH CO.
WYANDOTTE CO.
LAFAYETTE CO.
SHAWNEE CO.
DOUGLAS CO.
JACKSON CO.
JOHNSON CO.
FRANKLIN CO.
MIAMI CO.
CASS CO.
JOHNSON CO.
KANSAS
Missouri River
Kansas River
KANSAS TURNPIKE
North: 5,874,297

Intersection	# of Trucks per Year
From North (I-29S at Platte Co. Line & I-35S at Clay Co. Line)	5,874,297
From South (I-35N at Miami Co. Line & I-71N at Cass Co. Line)	9,322,597
From East (I-70W at Jackson Co. and Lafayette Co. Borders)	27,626
From West (I-70E before K-7)	7,933,529
Total	23,158,049

Figure 5. Through traffic view

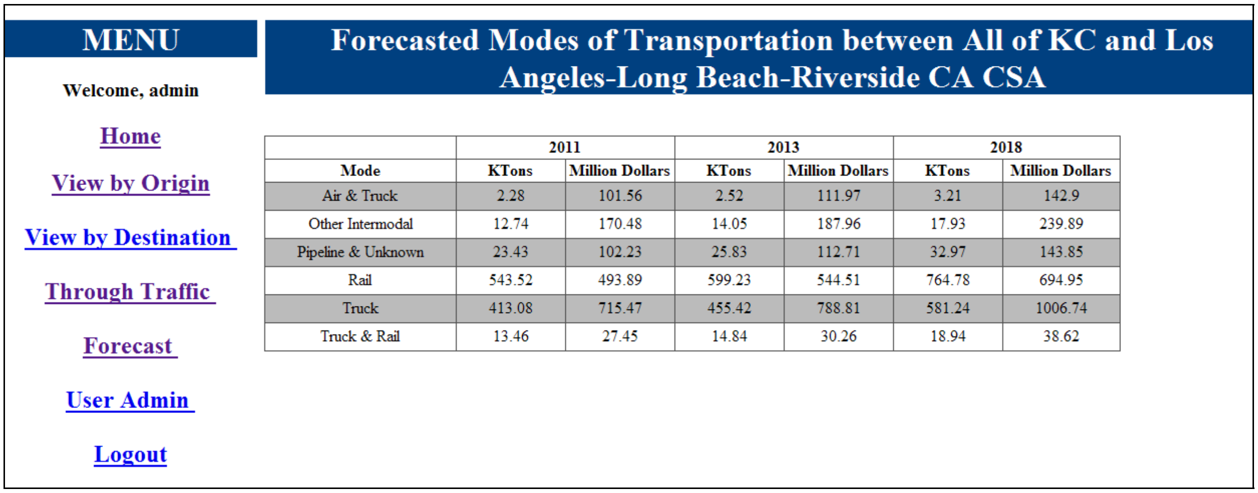
The “Forecast” feature gives the user multiple options to estimate the future traffic that will come to or go from Kansas City, which is shown in Figure 6. The first option “Forecast Using FAF Data” compares the 2002 and 2007 truck traffic data and estimates traffic for 3, 5, and 10 years in the future. The second option, “Forecast Using Selected Percentage” allows the user to set a percent increase or decrease in traffic for all commodities to propagate to the future estimations. Additionally, the user can select up to four commodities to specify a percentage independently of the rest for special consideration. As with the origin/destination views, forecasting allows the user to select from “Commodity,” “Mode,” or “Truck Traffic” views, and the forecasted results are all in a single table, as shown in Figure 7.


MENU
Welcome, admin
Home
View by Origin
View by Destination
Through Traffic
Forecast
User Admin
Logout
Select Forecast Methods
Remaining commodities
Origin
Destination

● Forecast Using FAF Data
○ Forecast Using Selected Percentage
Remaining commodities 0 %

Users may set specific percentages for up to 4 commodities.
At the end, select 'Remaining commodities' to set a percentage for any unspecified commodities

Figure 6. Forecast view selection


MENU
Welcome, admin
Home
View by Origin
View by Destination
Through Traffic
Forecast
User Admin
Logout

Forecasted Modes of Transportation between All of KC and Los Angeles-Long Beach-Riverside CA CSA

	2011		2013		2018	
Mode	KTons	Million Dollars	KTons	Million Dollars	KTons	Million Dollars
Air & Truck	2.28	101.56	2.52	111.97	3.21	142.9
Other Intermodal	12.74	170.48	14.05	187.96	17.93	239.89
Pipeline & Unknown	23.43	102.23	25.83	112.71	32.97	143.85
Rail	543.52	493.89	599.23	544.51	764.78	694.95
Truck	413.08	715.47	455.42	788.81	581.24	1006.74
Truck & Rail	13.46	27.45	14.84	30.26	18.94	38.62

Figure 7. Forecast view

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The following conclusions were reached at the end of this research project:

1. In 2007, the top five commodities shipped to the Kansas City Metropolitan Area by weight include cereal grains, gravel, nonmetal mineral products, waste/scrap, and unknown goods.

2. In 2007, the top five commodities shipped from the Kansas City Metropolitan Area by weight include cereal grains, gravel, nonmetal mineral products, waste/scrap and other agricultural products.
3. In 2007, the top five commodities shipped to the Kansas City Metropolitan Area by value include machinery, mixed freight, motorized vehicles, pharmaceuticals, and electronics.
4. In 2007, the top five commodities shipped from the Kansas City Metropolitan Area by value include machinery, mixed freight, motorized vehicles, pharmaceuticals, and textiles/leather.
5. The KFAF is a web-accessible, commodity-destination database that allows registered users to quickly view collected data from past years along with estimations of future shipments to and from the greater Kansas City Metropolitan Area. Currently, it contains data from the 2002 and 2007 versions of the FAF, which can be found online at http://ops.fhwa.dot.gov/freight/freight_analysis/faf/index.htm.
6. The KFAF can be used by KDOT planners when making decisions for maintaining an adequate infrastructure in Kansas.
7. The framework of the KFAF can be used to develop a freight analysis model for other cities in the State of Kansas once reliable data become available.

Recommendations

The results of this research also led the researchers to certain recommendations in order to improve the KFAF. Based on the results of this research project, the following recommendations are made:

1. There is a need to improve the accuracy of the data and determine if a more accurate data source could be developed for the Kansas City Area.
2. There is a need to apply more specific assumptions to the types of trucks used. 18 wheelers were assumed to ship all commodities in this study. However, in reality, a combination of trucks was used to ship commodities in and out of Kansas City.
3. The through truck calculations could be improved with a more accurate way of choosing in and out locations.
4. There is a need to consider the future intermodal facilities and the new manufacturing warehouses in the projections and forecasts of truck numbers and commodity shipments.
5. MODOT and KDOT need to work together to provide a transportation plan for the Kansas City Metropolitan Area.
6. There is a need to study the effects of the new light rail plan on future transportation issues.
7. There is a need to study the impact of the through truck traffic on the Kansas City highways, such as highway capacity, road conditions, and maintenance costs.

ACKNOWLEDGEMENTS

The authors would like to express their gratitude to Mr. John Maddox and Mr. John Rosacker from Kansas Department of Transportation (KDOT) for their valuable advice during the course of this study. Special thanks also go to Kansas University Transportation Research Institute, KDOT, and Federal Highway Administration (FHWA) for providing generous financial support.

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An Analysis of Emergency Message Delivery Scheme in Inter-Vehicular Networking

Weidong Xiang
Department of Electrical and Computer Engineering
University of Michigan
4901 Evergreen Rd.
Dearborn, MI 48128
xwd@umd.umich.edu

Hong Nie
Department of Industrial Technology
University of Northern Iowa
Industrial Technology Center 45
2900 Campus Drive
Cedar Falls, IA 50164-0178
hong.nie@uni.edu

EXTENDED ABSTRACT

Inter-vehicle networking has been proved to be an efficient and enabling system for the enhancement of road safety. With the help of inter-vehicle networking, a driver can be notified with a warning message for a traffic jam or collision in a real-time mode, and thus, take a positive action promptly to avoid entering jam areas or multiple-vehicle collisions. In this paper, we study the performance of an emergency message delivery scheme in an inter-vehicle network. When a critical safety-related event occurs in a vehicle, such as a sudden brake or the emerging of a road hazard, an emergency message is automatically generated and broadcast to the neighboring vehicles within a safety area through an one-hop or multiple-hops delivery. If the message is received by all the vehicles with the safety area before the deadline, the safety is guaranteed. However, the emergency message delivery scheme encounters significant challenges to meet the high requirements on both latency and reliability. At first, the topology of an inter-vehicle networking is normally unknown and time varying. Moreover, wireless coverage and channel fading under diverse road situations seriously impact the connectivity among the vehicles within the safety area. In this paper, we build up a simplified model to study the relation between the size of safety area and the message delivery distance per hop and the number of hops. Through theoretical analysis and computer simulations, the statistical characteristics of the size of safety area (D_s) as a function of the message delivery distance per hop (R), the number of hops (N), and the vehicle arrival rate (γ) will be carefully investigated. The obtained research results can be used to estimate the required number of hops and possible latency to deliver emergency messages within a designed safety area.

Key words: emergency message delivery—inter-vehicle networking—safety

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INTRODUCTION

As a fundamental element of contemporary society, road transportation has been playing a vital role in commercial activities and the daily lives of the majority of U.S. citizens. Yet road transportation in the United States is presently in difficult circumstances in terms of two aspects: safety and efficiency. The National Highway Traffic Safety Administration (NHTSA) has reported that there are 6.2 million crashes annually, resulting in more than 43,000 fatalities and a cost to society of more than $230 billion [1]. Safety systems widely adopted by automakers are typically based on individual vehicle implementations, such as seat belts, air bags, and anti-lock brakes. Thus, beyond some substantial initial gains, those single-vehicle systems can hardly further alleviate fatalities or injuries. Figure 1 shows that death number has been gradually increased in recent years, even though most of the states have laws strongly supporting highway safety. Road accidents are often caused by drivers' carelessness or ignorance, simple misconduct, or lack of experience, and it is extremely difficult to eliminate these "human" factors due to the inherent limits of human sensing and reaction speeds. All other measures applied failed to reduce the number of fatalities for the last 17 years, and more people in the United States give their lives in transportation-related accidents than any other single cause.

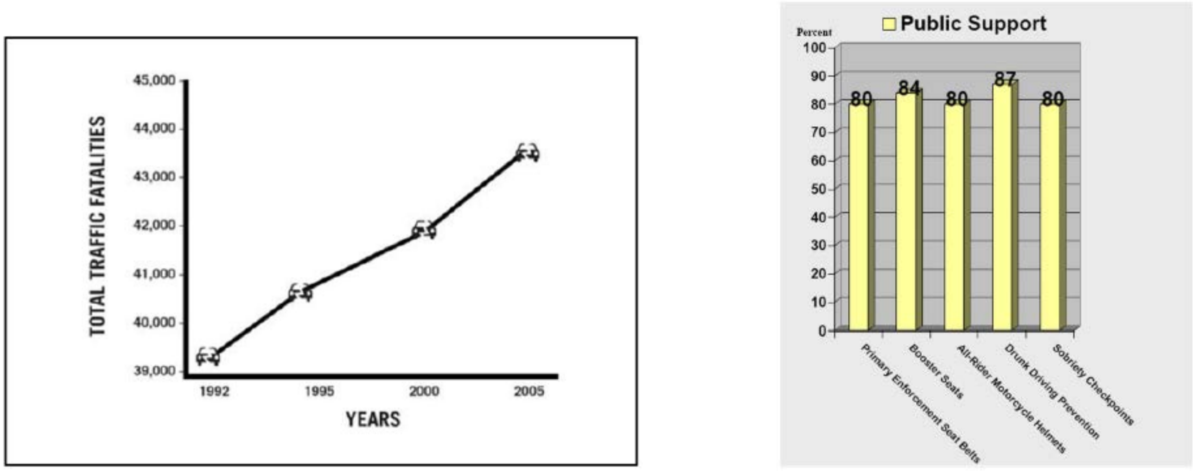


Figure 1. The U.S. traffic fatalities in recent years (left) and percentage of the states with laws supporting various highway safety efforts (right)

Besides crashes, due to aging and increasing usage of the road transportation infrastructure in the United States, congestion has emerged as another critical issue that negatively impacts our lives in multiple ways: it creates inefficiencies in roadway use, wastes fuel, causes widespread pollution and noise, and reduces personal "quality time." For example, traffic congestion costs Chicago $7.3 billion per year [2]. The average commuting time increased 14% in the last 10 years from 22.4 minutes in 1990 to 25.5 minutes in 2000 [3]. In many areas of the country, traffic congestion has become a major quality of life issue that impacts decisions as fundamental as where to buy a home or where to work [3]. "We are experiencing increasing congestion on our nation's highways, railways, airports and seaports. And we're robbing our nation of productivity and our citizens of quality time with their families" [4]. However, it is an expensive strategy to relieve congestion solely through major road infrastructure expansion.

Since changing human behavior and/or significantly enhancing human abilities critical for safe driving is unlikely and major road infrastructure expansion for congestion relief is expensive, technological solutions to improve road safety and efficiency are thus essential. For example, Mr. Paul Brubaker, the administrator of the Research and Innovative Technology Administration (RITA) of the U.S. Department of Transportation (U.S. DOT), has challenged the nation to reduce transportation accident-related deaths in the United States by 90% over the next decade through the use of better information technologies. With the wireless access for vehicular environment (WAVE) technology, the wireless transceivers installed on vehicles and roadsides form a local road areas network supporting inter-vehicle (V2V and V2I) communications and provide drivers perception, early warning, and assistance through accurate and fast sensing, surveillance, and information sharing among vehicles. As shown in Figure 2 [5], the local road area network will help drivers:

1. Maintain local vehicular awareness of surroundings in a real-time manner
2. Extend perception from local and transient to global and long-term to make prediction and preemptive responses possible
3. Translate situational information to appropriate actions, and develop multiple and collaborative automatic vehicle safety control strategies

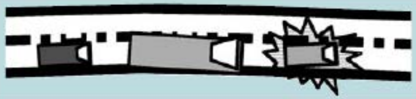
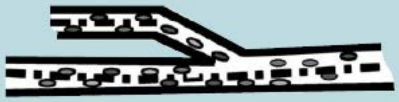
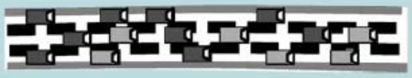
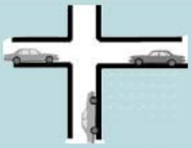
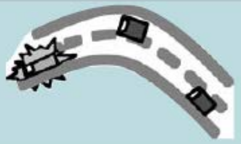
Example	Description	Image
Obstacle Warning	Stopped/skidding/slowing down vehicle warning, road obstacle/object-on-road warning	
Lane Merge/Lane Change Assistance	Merging/lane-changing vehicle communicates with vehicles in lane to safely and smoothly merge	
Adaptive Cruise/Cooperative Driving	Automatically stop and go smoothly, when vehicles are in heavy roadway traffic; cooperative driving by exchanging cruising data among vehicles	
Intersection/Hidden Driveway Collision Warning	Vehicles communicate to avoid collision at intersection without traffic light or hidden driveway	
Roadway Condition Awareness	Vehicles communicate to extend vision beyond line of sight (e.g., beyond a big turn or over a hill)	

Figure 2. Major functions of a local road area network

Furthermore, as depicted in Figure 3, when many local road areas networks are connected with each other through dedicated networking or the Internet in either wired or wireless links, a vehicle integrated information infrastructure is formed, where vehicles work as information probes and report timely traffic and road condition information to transportation agencies, and then the obtained information can be shared by a large community to reduce transportation accidents and relieve congestion. It is expected that with the aid of the vehicle-integrated information infrastructure for delivering real-time, local and global traffic messages, and further assuming that most drivers will be acting rationally, U.S. transportation systems will become more controllable and predictable.

Vehicles have been operating on roadways on this planet for more than a century in an isolated way. We are at the right time to connect these vehicles together and bring our society into a new age. Similar evolution happened on computer networks: when millions of computers were connected to share resources and information, the Internet emerged, and some profound positive changes were made to the way of our life and work. Once vehicle infrastructure integration (VII) based on the WAVE technology is implemented, it will radically improve road transportation environments in terms of safety, efficiency, and information access; fundamentally smooth the progress of intelligent transportation systems (ITS) advocated by the U.S. DOT; and turn driving and riding into a completely new experience—safer and more pleasant than ever before. Except for the WAVE-based VII, no other solution has the promise to save so many lives by actively avoiding crashes, improve the nation's productivity by relieving road congestions, promote an eco-friendly environment by reducing vehicle induced crude oil usage, and enhance the nation security by enabling the usage of vehicle recording and tracking. Furthermore, the enormous potential market for WAVE-related products can sustain several tier-one automobile suppliers and many supporting companies. Thousands of workshops will be needed to install WAVE systems to existing billions of vehicles. Moreover, the services supported by the vehicle-integrated information infrastructure could foster the generation of several service operators, the size of which could be comparable to that of cellular carriers. In summary, the WAVE-based VII has the potential to generate a fresh information technology industry based upon roadway vehicles, and it will bring the United States an opportunity to grow the high-tech sector of its economy and enhance its international economic competitiveness. The magnitude and breadth of the impacts of the WAVE-based VII on the economy of the United States are substantial, multi-layered, and profound.

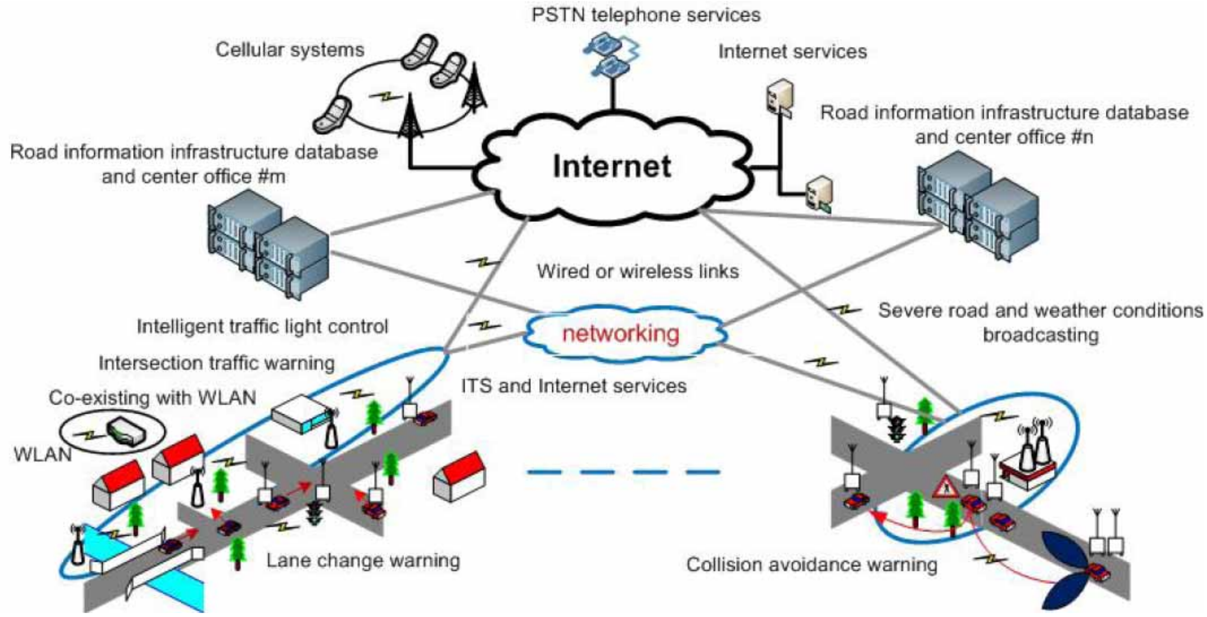

PSTN telephone services
Cellular systems
Internet services
Road information infrastructure database and center office #n
Road information infrastructure database and center office #m
Internet
Wired or wireless links
Severe road and weather conditions broadcasting
Intelligent traffic light control
networking
Intersection traffic warning
ITS and Internet services
Co-existing with WLAN
WLAN
Lane change warning
Collision avoidance warning

Figure 3. The vehicle integrated information infrastructure based on the WAVE technology

EMERGENCY MESSAGE DELIVERY SCHEME

As one of important functions of the WAVE-based VII, when an emergency event or a danger is detected by a vehicle, a warning message is automatically generated and broadcasted to neighboring vehicles within a defined safety area. If messages are surely received by all the vehicles within the safety area before a deadline, which relates to the reasonable action time of a driver, emergency events or dangers can be known in a real-time mode and earlier actions can be taken by drivers to avoid collisions, and thus, safety can be significantly enhanced. However, the delivery of emergency messages is very challenging because the topology of the inter-vehicle network varies rapidly with time, which is related to the distributions and movements of vehicles, and guaranteed latency is required for the delivery of emergency message. Some research activities on emergency message delivery schemes have been done recently [6]. In this paper, we develop a simplified model to study the relation between the size of safety area and the message delivery distance per hop and the number of hops.

System Model

For simplicity, a one-dimensional road model is used in this paper. As shown in Figure 4, vehicles travel along a lane independently. The number of vehicles, n, within a given zone of x follows a Poisson distribution given by

$$P(n,x)=\frac{(\gamma x)^n e^{-\gamma x}}{n!}, \tag{1}$$

where γ is vehicle arrival rate. As a consequence, the distance d_i between the ith and the $(i+1)^{\text{th}}$ vehicles follows an exponential distribution, and its probability density function (pdf) can be expressed as

$$P(d_i)=e^{-\gamma d_i}. \tag{2}$$

Therefore, the distance from the nth vehicle to the first vehicle that generates an emergency message is given by

$$D_n = d_1+\cdots+d_{n-1}, \quad n\geq 2 , \tag{3}$$

and after simple derivations, the pdf of D_n can be expressed as

$$P(D_n)=\frac{\gamma^n D_n^{n-1}}{(n-1)!}e^{-\gamma D_n} . \tag{4}$$

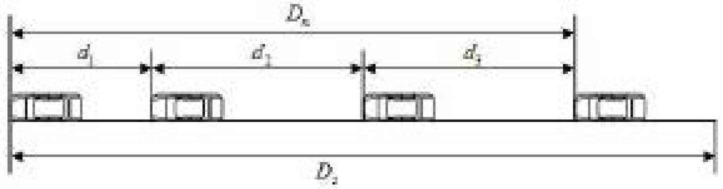


Figure 4. One-dimensional road model for emergency message delivery system

When the emergency message is delivered through multiple hops, in order to avoid messages flooding everywhere, it is assumed that only the vehicle that satisfies the following condition will relay the emergency message: the vehicle is the furthest vehicle from the vehicle transmitting the emergency message but still within the message delivery distance per hop. Therefore, if the message delivery distance per hop equals R and the number of hops equals N, the relation between the size of safety area, D_s, which becomes a distance in a one-dimensional road model, and R and N can be summarized as follows:

i) when $N=1$, $D_s=R$
ii) when $N=2$, $D_s=D_{n1}+R$ with $D_{n1}\leq R$ and $D_{n1+1}>R$
iii) when $N=3$, $D_s=D_{n2}+R$ with $D_{n1}\leq R$, $D_{n1+1}>R$, $D_{n2}-D_{n1}\leq R$, and $D_{n2+1}-D_{n1}>R$

iv) when $N=i$, $D_s=D_{n(i-1)}+R$ with $D_{n1}\leq R$, $D_{n1+1}>R$, $D_{n2}-D_{n1}\leq R$, $D_{n2+1}-D_{n1}>R$, …, $D_{n(i-1)}-D_{n(i-2)}\leq R$, and $D_{n(i-1)+1}-D_{n(i-2)}>R$

In this paper, through theoretical analysis and computer simulations, the authors will carefully investigate the statistical characteristics of the size of safety area (D_s), as a function of the message delivery distance per hop (R), the number of hops (N), and the vehicle arrival rate (γ) based upon the above described relation. The obtained research results can be used to estimate the required number of hops and possible latency to deliver emergency messages within a designed safety area.

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Dynamic Modulus of HMA: Preliminary Criteria to Prevent Field Rutting of Asphalt Pavements

Zhanping You
Department of Civil and Environmental Engineering
Michigan Technological University
1400 Townsend Drive
Houghton, Michigan 49931-1295
zyou@mtu.edu

R. Christopher Williams
Department of Civil, Construction, and Environmental Engineering
Iowa State University
482A Town Engineering
Ames, Iowa 50011
rwilliam@iastate.edu

Shu Wei Goh
Department of Civil and Environmental Engineering
Michigan Technological University
1400 Townsend Drive
Houghton, Michigan 49931-1295
sgoh@mtu.edu

ABSTRACT

The objective of this paper is to present preliminary criteria for dynamic modulus of hot mix asphalt (HMA) in order to control field rutting performance of asphalt pavements. In this study, 14 field-produced mixtures at various traffic levels and aggregate sizes were evaluated and compared with field rutting performance. Some of the selected pavements were constructed 10 years ago and some were built recently. These mixtures were collected from the job sites and compacted with a Superpave gyratory compactor to the common air-void level used in Michigan, which is 4%. Dynamic modulus, E*, testing was conducted at temperatures ranging from -5°C to 39.2°C and frequencies ranging from 0.1 Hz to 25 Hz. The results show that dynamic modulus values increased when the design traffic level for HMA mixtures increased and no significant difference in dynamic modulus when comparing mixtures designed with various nominal maximum aggregate size. Preliminary dynamic modulus criteria for Michigan were proposed and evaluated.

Key words: criteria—dynamic modulus—field rutting performance—hot mix asphalt—master curve—simple performance test

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Mechanical Properties of Asphalt Mixtures with Recycled Concrete Aggregates

Zhanping You
Department of Civil and Environmental Engineering
Michigan Technological University
1400 Townsend Drive
Houghton, Michigan 49931-1295
zyou@mtu.edu

Julian Mills-Beale
Department of Civil and Environmental Engineering
Michigan Technological University
1400 Townsend Drive
Houghton, Michigan 49931-1295
jnmillsb@mtu.edu

EXTENDED ABSTRACT

The urge sustainable asphalt highway design and construction towards is necessitated by the high diminishing rate of construction materials, pressing demand on existing landfill sites, rising dumping fees, and reducing emissions into the environment. Recycled asphalt pavement (RAP), warm mix asphalt (WMA), fly ash, bottom ash, and roof shingles are some of the sustainable transportation materials that are currently gaining prominence for the design and construction of asphalt pavement highways. A new waste aggregate material in recycled concrete aggregate (RCA) is being investigated in this research. RCA can be described as waste materials from portland cement concrete construction structures and pavements without the elements of organic matter and steel reinforcement. In spite of its abundance, RCA has seen limited use as a material in asphalt pavement design and construction.

The limited use of RCA is due to a lack of in-depth research knowledge about the effect of RCA on the performance of asphalt pavements. This project, therefore, seeks to evaluate the possibility of utilizing RCA for low equivalent standard axle load (ESAL) asphalt pavement highways. The mechanical performance of asphalt mixtures, in which natural aggregates are substituted with RCA, is determined to prove the extent of use of RCA in asphalt pavements. The choice of a low ESAL asphalt pavement is based on the hypothesis that RCAs are lower in strength than natural aggregates based on the basic composition of the RCA—an underlying natural aggregate and a cement concrete paste coating.

In the project, a traditional asphalt mixture suitable for a low ESAL road is selected. This mixture is considered the control mix, with the aggregate constituent being natural aggregates. A 4E1 control mix suitable for low ESAL Michigan roads is thus chosen. By convention and the standards of the Michigan Department of Transportation (MDOT), the 4E1 asphalt mixture has design traffic between 0.3 and 1 million ESAL.

With the 4E1 control mix having 0% of RCA, substitutions of RCA are conducted at the rates of 25, 35, 50 and 75% to produce a hybrid asphalt mixture of the RCA and the original natural aggregates. The natural aggregate is a typical Michigan trap rock virgin aggregate (VA). The hybrid VA-RCA HMA is then assessed using the Superpave™ mix performance specifications. The Superpave™ performance

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specifications are an embodiment of industry- and academia-accepted standard test methods and protocol in the United States currently used to evaluate the laboratory performance of asphalt binder and mixtures, while correlating it to identify the potential field behavior of the asphalt binder and mixture.

The hybrid asphalt mixtures were prepared and tested to evaluate their potential to withstand the 0.3 to 1 million ESAL traffic damage, moisture damage, and compaction energy resistance. The Superpave™ specifications require tests such as the dynamic modulus (E*), tensile strength ratio (TSR) for moisture susceptibility, asphalt pavement analyzer (APA) for permanent deformation damage. Additionally, the construction energy index (CEI) was determined to predict the extent of energy conservation during the paving of an asphalt pavement on the roadway.

All four hybrid VA-RCA HMA mixes passed the minimum rutting specification of 8 mm, which suggests the mixes can be satisfactorily used without due consideration on damage due to repeated traffic loads (0.3 and 1 million ESAL) along the direction of the traffic wheels. In terms of moisture susceptibility, the TSR for moisture susceptibility predicted that as the RCA content in an asphalt mixture decreases, the tensile strength increased—a trend which also suggested that a maximum limiting percentage value of 75% of RCA may be necessary if moisture damage to an asphalt pavement is to be avoided. The construction or compaction energy index (CEI) proved that using RCA would save some amount of compaction equipment energy. The dynamic modulus increased with increasing RCA in the HMA based on an analysis conducted using the master curves obtained from time-temperature superposition graphs. The master curves showed that the RCA will significantly improve the low- and high-frequency rheological properties of the RCA-VA hybrid asphalt mixture.

The research showed that even though RCA has great potential for replacing portions of natural aggregates in asphalt mixtures, a preliminary test protocol is, however, required to guide the use of RCA in HMA. It is recommended that a certain amount of RCA in HMA is acceptable for low-volume roads. Producing asphalt paving mixtures with RCA can: (1) save energy resources of our environment, (2) limit extensively the use of dumping sites for cement concrete waste, (3) reduce the demand, cost, and energy resources used for obtaining natural aggregates, and (4) contribute to low emissions to the environment due to less energy use for asphalt pavement compaction and limited use of natural aggregates.

Key words: asphalt mixtures—low-volume roads—mechanical properties—recycled concrete aggregates

Warm Mix Asphalt: Laboratory Evaluation and Pavement Design

Zhanping You
Department of Civil and Environmental Engineering
Michigan Technological University
1400 Townsend Drive
Houghton, Michigan 49931-1295
zyou@mtu.edu

Shu Wei Goh
Department of Civil and Environmental Engineering
Michigan Technological University
1400 Townsend Drive
Houghton, Michigan 49931-1295
sgoh@mtu.edu

EXTENDED ABSTRACT

This presentation is based upon a research project partially funded by the Michigan Department of Transportation. In this presentation, an overview of warm mix asphalt (WMA), in terms of the environment, will be provided through literature reviews and lab and field experiences with Sasobit® technology. Transportation officials are rapidly becoming aware of WMA due to its uniqueness compared with hot mix asphalt (HMA). Warm mix asphalt is a new paving technology that originated in Europe that appears to allow a reduction in the temperature at which asphalt mixes are produced and is a promising alternative to traditional HMA.

A field demonstration project with WMA and HMA was held at M-95, north of US 2 at Iron Mountain, Michigan. In this demostration, Sasobit® (one of the WMA technologies) was added at a rate of 1.5% (by mass of binder) to the mixture. The mixing temperature used for WMA was 260°F (126.7°C) and for HMA was 320°F (160°C). During the WMA production, it was observed that the emission was significantly reduced compared to HMA production. Figure 1 shows the comparison of truck load out emissions between HMA and WMA during the production. A reduction of 14% in NOx, a 5% decrease in CO_2, and a slight decrease in volatile organic compounds (VOCs) was reported when compared to HMA .

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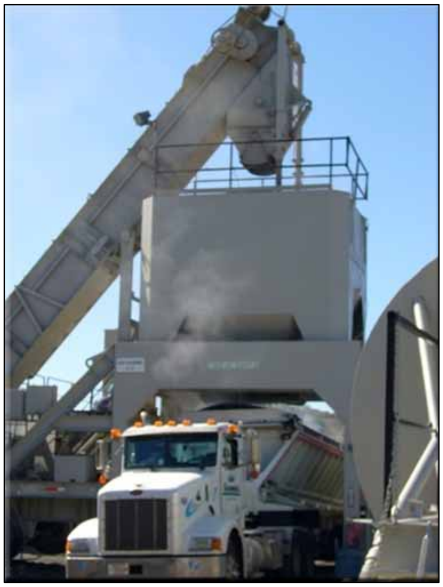

Figure 1(a). Hot mix asphalt

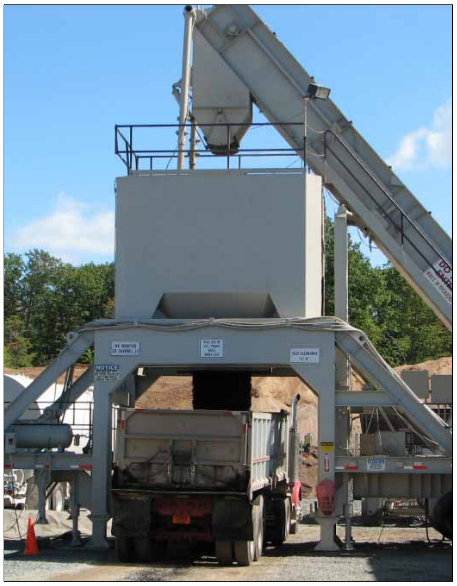

Figure 1(b). Warm mix asphalt

Reference*: Goh, S.W.and You, Z. (2008),” Warm Mix Asphalt using Sasobit®: A Brief Field and Laboratory Experience,” Proceedings of the 2008 Mid-Continent Transportation Research Forum*, Madison, Wisconsin, August 2008

Samples collected from the field were evaluated using Asphalt Pavement Analyzer (APA) rutting and tensile strength ratio (TSR) tests. The results of the APA and TSR tests are presented in Figures 2 and 3. Based on the results conducted, it was found that WMA has a similar rutting depth compared to the control mixture. It is noteworthy that WMA was compacted at 126.7°C (260°F), which is about 25°C (45°F) lower than traditional HMA (compacted at 152°C). The results also indicated that WMA with a reduction of 25°C (45°F) in compacting temperature has a similar rutting performance to HMA. For the TSR test, it was found that WMA is compatible with the HMA (control mixture), which could indicate there are no significant differences between WMA made with Sasobit® and HMA in terms of moisture damage.

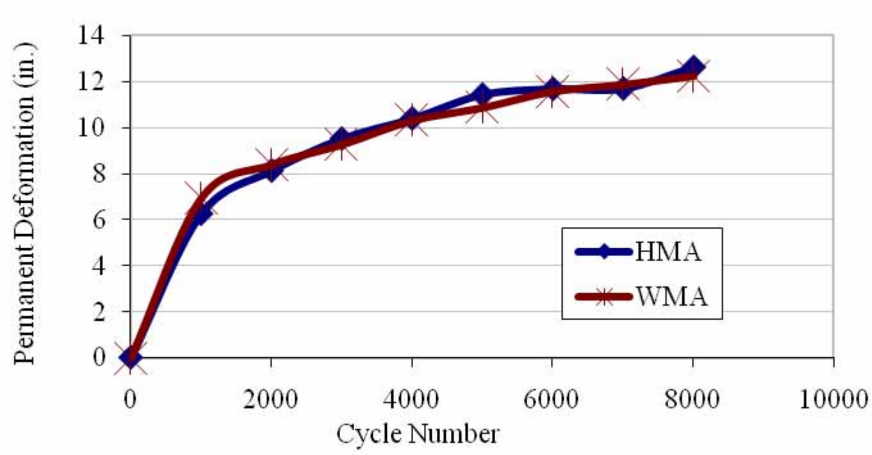

Figure 2. Rutting results for HMA and WMA made with Sasobit® tested at 58°C

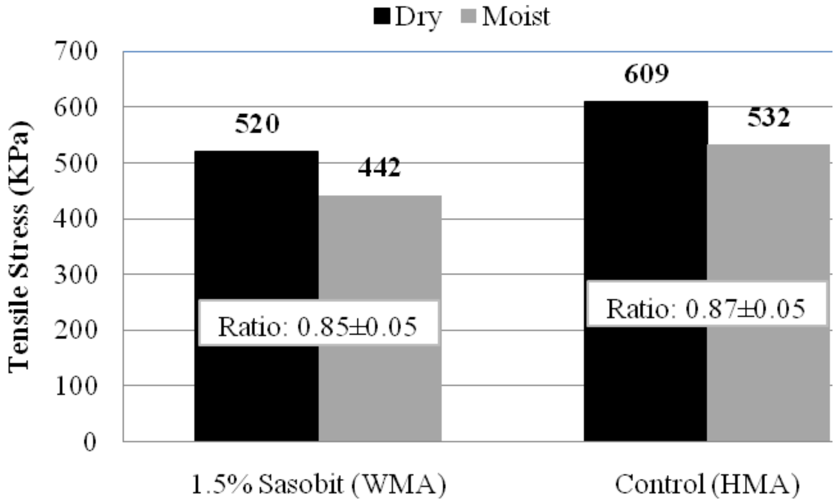

Figure 3. Tensile strength ratio result for control and WMA mixtures

Key words: laboratory evaluation—Sasobit®—traditional hot mix asphalt—warm mix asphalt

Effectiveness of Driver Improvement Programs

Wei Zhang
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
weizh@iastate.edu

Shashi Nambisan
Institute for Transportation
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
shashi@iastate.edu

Konstantina “Nadia” Gkritza
Center for Transportation Research and Education
Iowa State University
2711 S. Loop Drive, Suite 4700
Ames, IA 50010
nadia@iastate.edu

ABSTRACT

Driver improvement programs have been pursued by many states in the United States to reduce the public safety risk posed by repeat violators of traffic rules and regulations. These programs include issuing warning letters, providing educational material and courses, conducting diagnostic reexaminations, individual counseling, and license suspension/revocation. Iowa’s Driver Improvement Program targets drivers who have received multiple citations for moving violations. Under this program, in lieu of driver’s license suspension, such drivers may be required to attend and successfully complete a driver improvement program at the person's own expense. Currently, there are 17 community colleges across the state of Iowa that offer the approved program.

A number of studies have been reported over the past three decades in the literature on the effectiveness of driver improvement programs. Most studies have been initiated by state motor vehicle divisions and, hence, offer state-specific results, while a few studies constitute a meta-analysis and offer quantitative reviews of existing research. This presentation will provide a synthesis of practices in the United States related to driver improvement programs and driver education (in-class training and online driver education courses). It also will offer a review of the existing literature (both in the United States and internationally) on the effectiveness of driver improvement programs (or select driver interventions) in reducing crashes and violations. The presentation will discuss data and methodological approaches used to evaluate such programs. These are key steps to develop recommendations of educational materials and strategies for adoption in driver education programs in Iowa and other states so as to enhance transportation safety.

Key words: driver improvement program—effectiveness—safety

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Application of Wireless Sensor Networks in Infrastructure Monitoring: A Preliminary Study

Jin Zhu
Department of Industrial Technology
University of Northern Iowa
Industrial Technology Center 44
Cedar Falls, IA 50613
Jin.zhu@uni.edu

M.D. Salim
Department of Industrial Technology
University of Northern Iowa
Industrial Technology Center 33
Cedar Falls, IA 50613
Md.salim@uni.edu

EXTENDED ABSTRACT

There has been a growing interest to deploy sensors for monitoring the conditions of infrastructure systems such as buildings, bridges, and highways. Traditionally, infrastructure inspection is performed via infrequent periodical visual inspection in the field. Wireless and sensor technology provide an alternative cost-effective approach for real-time monitoring of infrastructures.

Although applying sensing technology in structure health monitoring is not a new concept, the installation and maintenance cost of such monitoring systems is high. The typical structure health monitoring (SHM) system consists of sensors, a data acquisition instrument, and a PC, which are connected together via cables. It is necessary to deploy multiple sensors over the structure to collect adequate information, and thus, the cables have to run all over the structure. The installation of sensor cables is labor intensive, and the maintenance is not an easy job. In addition, the equipment cost of commercially available SHM systems is also high. These drawbacks limit SHM systems to be deployed widely for infrastructure monitoring.

This research studies the feasibility of applying the wireless sensor networks to implement cost-efficient infrastructure monitoring. Wireless sensor networking technologies have advanced quickly in the past few years. By integrating the sensing, processing, and wireless communication capability together, the low-cost wireless sensor nodes provide a good platform for infrastructure monitoring. A sensor node consisting of temperature/light/humidity sensors, a microcontroller, and an IEEE 802.15.4 transceiver could cost less than $100. Small-size, low-cost, and cable-free installation of wireless sensor nodes not only make the installation easier but also reduce both maintenance and installation costs significantly. The data processing can be done along the sensor nodes, and only abstract information will be transmitted back wirelessly. Decentralized monitoring mode may also help to improve the monitoring reliability. A test bed of the wireless sensor monitoring module is in the progress of implementation. The module will be tested in the lab and preliminary data will be presented.

Key words: infrastructure monitoring—sensor networks—wireless

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Web-based Collaboration for Iowa DOT Bridge Construction

James S. Nelson
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010
James.S.Nelson@dot.iowa.gov

Aaron C. Zutz
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
394 Town Engineering
Ames, IA 50011
aczutz@iastate.edu

Charles T. Jahren
Department of Civil, Construction, and
Environmental Engineering
Iowa State University
394 Town Engineering
Ames, IA 50011
cjahren@iastate.edu

EXTENDED ABSTRACT

Bridge construction projects are becoming increasingly complex as the demand for context sensitive solutions, aesthetic designs, and accelerated bridge construction becomes more prevalent. In addition, the Iowa Department of Transportation (Iowa DOT) is entering a phase of design and construction of large border bridges, such as the I-80 (Let 2008 for $56 Million) and US 34 bridges over the Missouri River and I-74 over the Mississippi River.

Compared to typical construction projects, these bridges generate more contractor Requests for Information, Value Engineering Proposals, Requests for Changes, and shop drawings. Management of these submittals is a significant challenge for Resident Construction Engineers and other department of transportation (DOT) staff. Additionally, some submittals require cross-departmental review and review from project consultants as well. Implementation of a web-based collaboration solution is intended to speed construction submittal review time, reduce incidence of delay claims, and free up DOT staff from project management administrative tasks.

Researchers from Iowa State University working with the Iowa DOT conducted a multi-pronged approach to indentify a web-based collaboration solution for Iowa DOT bridge projects. An investigation was launched to determine the functional needs of the Iowa DOT. Commercially available software programs were also evaluated to find what functionality is currently available. By comparing the needs of the DOT to what is currently available, a recommendation was made for a solution to be used on a pilot project by the Iowa DOT. The results of this research should give the Iowa DOT a tool to assist in the successful completion of future complex bridges.

Key words: bridge—collaboration—construction—web-based

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PROBLEM STATEMENT

Bridge construction projects are becoming increasingly complex as the demand for context sensitive solutions, aesthetic designs, and accelerated bridge construction becomes more prevalent. In addition, the Iowa Department of Transportation (Iowa DOT) is entering a phase of design and construction of large border bridges, such as the I-80 (Let 2008 for $56 Million) and US 34 bridges over the Missouri River and I-74 over the Mississippi River.

Compared to typical construction projects, these bridges generate more contractor Requests for Information (RFI), Value Engineering (VE) Proposals, Requests for Changes (RFC), and shop drawings. Management of these submittals is a significant challenge for Resident Construction Engineers (RCE) and other department of transportation (DOT) staff. In addition, some submittals require cross-departmental review and review from project consultants as well. Commercially available software exists for managing submittals and project collaboration teams; in-house solutions may also be possible. Implementation is intended to speed construction submittal review time, reduce incidence of delay claims, and free up DOT staff from project management administrative tasks.

RESEARCH OBJECTIVES

Researchers from Iowa State University working with the Iowa DOT conducted a multi-pronged approach to indentify a web-based collaboration solution for Iowa DOT bridge projects. An investigation was first launched to determine the functional needs of the Iowa DOT. Researchers sought to determine the current needs and practices of the Iowa DOT and other potential users of the collaboration solution. Researchers also needed to investigate what would promote or hinder the success of the proposed solution.

Concurrently, commercial software programs were evaluated to find what functionality was available. Researchers then worked to determine if commercially available solutions met the Iowa DOT’s functionality requirements. In many cases, commercially available solutions have capabilities beyond the functionality requirements identified by Iowa DOT. Such excess functionality might be valuable but overlooked by potential users because they are unfamiliar with the capabilities of commercial solutions. Therefore, researchers also investigated these capabilities and considered them for possible additions to the list of functional requirements.

A comparison of required functionality and available functionality would be used to make a recommendation to the Iowa DOT for an electronic collaboration solution to be used on pilot projects. Successful utilization of the selected solution should serve as a validation for the research and also provide lessons learned for future wide-scale implementation. Ultimately, this research will help provide the knowledge necessary for the Iowa DOT to successfully implement a long-term solution to assist all project participants in the management of Iowa DOT bridge projects.

RESEARCH METHODOLGY

To investigate the functionality required by the Iowa DOT for a web-based collaboration solution, interviews were conducted with users that would be affected by the proposed system. Interviews were conducted with Iowa DOT employees, consultants, contractors, and suppliers. Additionally, due to the inexperience of potential users, interviews were conducted with construction professionals from other construction sectors. Finally, a survey was developed and carried out to learn the processes of other state DOTs with respect to their collaborative practices.

Interviews were conducted in a relatively ad hoc format. A questionnaire was developed based on research done by others, initial contacts with Iowa DOT personnel, and information on commercial solutions. The questionnaire utilized primarily open-ended questions as to not limit the response of the interviewees and gain the most information. Researchers also probed deeper after asking some questions to obtain additional useful information.

The comparison of commercially available software programs was done by viewing demonstrations for a variety of solutions in order to ascertain information to fill out a set of questions for each solution. All of these responses were then combined into a matrix to facilitate the comparison of the programs.

To develop a set of questions to evaluate commercially available solutions, an initial round of demonstrations was conducted to determine a baseline for what was commercially available. Researchers then used the results of these initial demonstrations, interviews, internet research, and the criteria of other researchers to develop the set of questions. These questions were primarily closed ended to facilitate a direct comparison between solutions. Using a broad range of tactics, researchers identified over 30 web-based collaboration solutions. This list was deemed to be too large for an in-depth analysis of each solution, so researchers short listed a dozen solutions that best met the Iowa DOT's requirements prior to conducting the in depth examinations.

KEY FINDINGS

Based on the interviews conducted, it was determined that the Iowa DOT needed a web-based collaboration solution to manage four main areas: contract documents, shop drawing submittals, RFIs, and meeting minutes. A primary concern for the use of a web-based collaboration solution was the user friendliness of the solution. The ability for all participants to easily access the information would be paramount to the success of the solution. To accomplish this, a solution that did not distract the user with unneeded functionality was considered to be desirable. Additionally, features such as email alerts along with "Dashboard" and "Ball in court" features to alert users of new information or items requiring their attention would be helpful. Also, it was necessary to find a solution that could preserve the DOT's current workflow to help ease the transition. Finally, data security and the security of the solution were identified as critical to a solution's success.

Based on the required functionality, a number of solution attributes were determined to be critical for pilot project. First, it was determined that the solution used for the pilot project should be used "Software as a Solution." By having a vendor host the software, the solution could be much more easily and rapidly deployed. The software also needed to have the functionality to meet the DOT's four main areas. Additionally, to allow easy access for project participants, the solution needed to be accessible with only an Internet browser. Research showed that there were a number of commercially available solutions that meet the Iowa DOT's requirements. Due to the quantity of solutions meeting the Iowa DOT's requirements, researchers didn't recommend a specific program for the Iowa DOT but recommended a category of solutions meeting these requirements. Ultimately researchers worked with the Iowa DOT to issue a Request for Proposals (RFP) in order to most objectively select the solution to be pilot tested.

CONCLUSIONS

Iowa DOT is facing an increasing volume and level of complexity of bridge construction; these circumstances necessitate assistance in the management of these projects. Commercially available web-based collaboration solutions have the functionality required to meet the DOT's needs. A web-based collaboration solution has the potential to improve project success for all parties involved in the project by promoting accountability, increasing transparency, and decreasing the review time of documents. By pilot testing a web-based collaboration solution, the Iowa DOT will be able to learn valuable lessons that can be applied to future projects. This should help the Iowa DOT become better prepared to manage future complex bridge projects. Finally, the results of this project could assist other government agencies, including cities and counties in the State of Iowa move toward web-based collaboration on their construction projects.

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