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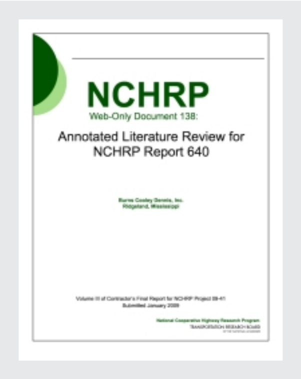
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TABLE OF CONTENTS

Task 1 - Conduct a Comprehensive Review of Worldwide Literature on PFC................. 1

1.1 National Roads Authority (NRA). Porous Asphalt in Ireland 1

1.2 Decoene, Y. "Contribution of Cellulose Fibers to the Performance of Porous Asphalts." Transportation Research Record No. 1265. Transportation Research Board. National Research Council. Washington, D.C. pp 82-86. 1990.................................... 2

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1.17 Rogge, D. and E.A. Hunt. "Development of Maintenance Practices for Oregon F-Mix – Interim Report SPR371." Oregon Department of Transportation. Salem, OR. August 1999.......... 78

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1.42 Litzka, J. "Austrian Experiences with Winter Maintenance on Porous Asphalt." Proceedings of the Ninth International Conference on Asphalt Pavements. Copenhagen, Denmark. August 2002. 175

1.43 Padmos, C. "Over Ten Years Experience with Porous Road Surfaces." Ninth International Conference on Asphalt Pavements. International Society of Asphalt Pavements. Copenhagen, Denmark. 2002. 178

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1.69 McDaniel, R. S. and W. Thornton. “Field Evaluation of a Porous Friction Course for Noise Control.” TRB 2005 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2005. 256

1.70 Scofield, L. and P. Donavan. “The Road To Quiet Neighborhoods In Arizona.” TRB 2005 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2005. 260

1.71 Sholar, G. A., G. C. Page, J. A. Musselman, P. B. Upshaw and H. L. Moseley. “Development of the Florida Department of Transportation’s Percent Within Limits Hot-Mix Asphalt Specification.” TRB 2005 Annual Meeting CD-ROM. Transportation Research Board. National Research Council, Washington, D.C. 2005. 264

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1.73 Wagner, C. and Y.S. Kim. “Construction of a Safe Pavement Edge: Minimizing the Effects of Shoulder Dropoff.” TRB 2005 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D. C. 2005 ... 270

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LIST OF FIGURES

Figure 1: Use of Cantabro Abrasion and Air Voids to Select Minimum Binder Content 21
Figure 2: Air Void Classifications 22
Figure 3: Examples of Daylighting Porous Asphalt Mixtures.. 24
Figure 4: Steps in Mix Design 32
Figure 5: Outline of the Components of an Anti-icing Program in the Context of a Winter Maintenance Program 37
Figure 6: Methods of Developing Job Mix Formulas for OGFCs.................................. 62
Figure 7: Methods for Determining Mixing Temperatures 62
Figure 8: Tack Coat Materials Used for OGFC... 68
Figure 9: Tack Coat Application Rates.. 69
Figure 10: Reported Service Lives for OGFC 70
Figure 11: Principle of Two Layer Porous Asphalt Pavements...................................... 81
Figure 12: Benefits of Open-Graded Mixes Cited by Agencies 88
Figure 13: Comparison of Air Void Contents... 113
Figure 14: Truck-Mounted Function-Recovery Machine... 152
Figure 15: High-Pressure Ejection and Vacuum Systems 153
Figure 16: Voids in Coarse Aggregate Concept for Ensuring Stone-On-Stone Contact169

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LIST OF TABLES

Table 1: Results of Basket Drainage Tests 3
Table 2: Results of Schellenberger Drainage Tests 4
Table 3: General Data on Materials and Mix.. 6
Table 4: Gradation Bands for Porous Asphalt Mixtures in Spain 10
Table 5: Typical Requirements for Porous Asphalt Mixes in the Netherlands 14
Table 6: Belgian Specification for the Composition of Porous Asphalt........................... 18
Table 7: Influence of Air Void Classification on Permeability and Noise 22
Table 8: Forms in Which Salts Are Used in Europe... 27
Table 9: Average Spreading Rates For Solid NaCl (g/m^2) 28
Table 10: Average Spreading Rates for $CaCl_2$ Flakes (g/m^2).. 28
Table 11: Average Spreading Rates for Wet Salt (g/m^2) 28
Table 12: Open-Graded Asphalt Used In Australia 30
Table 13: Mix Design of Open-Graded Asphalts in Australia 31
Table 14: Production and Laydown of Open-Graded Asphalt Mixes 33
Table 15: Performance Related Properties 34
Table 16: Weather Event: Light Snow Storm.. 41
Table 17: Weather Event: Light Snow with Period(s) of Moderate or Heavy Snow 42
Table 18: Weather Event: Moderate or Heavy Snow Storm 43
Table 19: Weather Event: Frost or Black Ice... 44
Table 20: Weather Event: Freezing Rain Storm 45
Table 21: Weather Event: Sleet Storm... 46
Table 22: Standard and Proposed New Porous Mix 48
Table 23: Problems with Porous Mixes 49
Table 24: Mixes used in the Test Sections around Amsterdam....................................... 50
Table 25: Problems with Tests... 50
Table 26: Observations from Condition Survey 51
Table 27: Details of Cyclic Tensile Test.. 51
Table 28: GDOT OFGC Gradation Requirements 53
Table 29: Summary of Mix Design Information of Research Mixes 54
Table 30: Average Friction Test Results for Test Sections 56
Table 31: Average Smoothness Values in Test Sections... 56

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Table 32: Results of Visual Distress Survey on Test Sections 57
Table 33: Average Rut Depths in Test Sections (inches) 57
Table 34: Average Permeability Results Over Time 58
Table 35: Stresses and Strains in Binder Films in Porous Asphalt........ 59
Table 36: Gradation of OGFC Mixes in Different States 63
Table 37: Asphalt Binders used for OGFCs 64
Table 38: Problems with OGFC 66
Table 39: Mix Design Practices of States with Good and Bad Experiences 67
Table 40: Materials and Mix Design 72
Table 41: Construction Practices 73
Table 42: Materials and Mix Design Criteria for OGFC Mixes Used in Florida........ 75
Table 43: Projects with Bleeding Problems........ 76
Table 44: Investigation and Follow-Up Work 77
Table 45: Materials and Mix Design for Laboratory Mixes 84
Table 46: Tests and Results 85
Table 47: In-Place Mixes 86
Table 48: Results of Visual Survey 86
Table 49: FHWA Design Gradation Band........ 88
Table 50: Summary of 9.5mm OGFC Mixture Designs used in United States........ 90
Table 51: Summary of 12.5mm and 19.0mm OGFC Mixture Designs used in the United States........ 91
Table 52: Summary of Non-North American Porous Asphalt Mixtures 92
Table 53: Gradations Used........ 98
Table 54: Summary of Mix Volumetric Properties 99
Table 55: Volumetric Properties of mixes with Difference Binders (Average Values). 100
Table 56: Results of Draindown Tests from Mixes with Different Binders........ 100
Table 57: Abrasion Loss (Aged Samples) for Mixes with Different Types of Binder... 101
Table 58: Laboratory Test Results for the Six OGFC Mixes (6)........ 106
Table 59: Laboratory Test Results for Field Produced OGFC Mixes (6)........ 106
Table 60: Laboratory Test Results for Roadway Core Samples from OGFC Test 107
Table 61: Severity and Percentage of Transverse Reflective Cracks 108
Table 62: Average Permeability and In-Place Air Void Contents for the Six Test Sections 109

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Table 63: Gradation Requirements and Design Gradation 111
Table 64: Properties of Fibers 112
Table 65: Materials and Mix Design Used in Projects in BC 116
Table 66: Suggested Gradation 118
Table 67: Results of Performance Evaluations of Different Projects 119
Table 68: Performance Life Equations from UK Trials 122
Table 69: Use of Different Materials in the UK Road Trials 123
Table 70: Summary of results from a Survey on the Use of Open-Graded Friction Course (OGFC), as Presented by Bolzan et al (based on response from forty-two states)......... 125
Table 71: Comparison of US Mixtures to European Mixtures 127
Table 72: Description of Full Scale Field Trials of Porous Asphalt Mixes in Argentina 128
Table 73: Details of Test Sections with Different Markings 133
Table 74: Testing of Durability and Retro Reflectivity for Recessed and Non-Recessed Pavement Markings 134
Table 75: Results of Durability and Retroreflectivity Evaluations 135
Table 76: Inferences from Statistical Analysis of Retro Reflectivity Results from Different Sections, for Different Conditions 136
Table 77: Use and Characteristic of Open-Graded Asphalt Concrete 143
Table 78: ODOT Specifications for Open-Graded Asphalt Mix 144
Table 79: Construction Concerns and Special Considerations for Open-Graded Asphalt Mixes 145
Table 80: Special Considerations 146
Table 81: Maintenance Issues and Recommendations 147
Table 82: Evaluation of Performance of Open-Graded Asphalt Mixes 149
Table 83: Results of Permeability Testing after Functional Recovery 152
Table 84: Results of Permeability Testing after Use of Truck-Mounted Function Recovery Machine 153
Table 85: Classification of Degree of Clogging 155
Table 86: Study Gradations 157
Table 87: Tests Conducted on Samples 157
Table 88: Road Surface Conditions during a Snowfall (Rutted Sections)...................... 163
Table 89: Road Surface Conditions during a Snowfall (Non-Rutted Sections) 164
Table 90: Frictional Data (Pennsylvania) 166
Table 91: Coarse Aggregate Requirements for OGFC 168

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Table 92: Recommended Gradation for OGFC 169
Table 93: Costs (net present value) and Effect of the Three Noise Abatement Techniques 174
Table 94: Models Relating Hydraulic Conductivity to Structural Design of Porous Pavements 181
Table 95: Properties of One Porous Mix and Two Unbound Aggregates 183
Table 96: Properties of One Porous Mix and Two Unbound Aggregates 183
Table 97: Typical Winter Maintenance Techniques for F-Mix 189
Table 98: Measurement Methods and Equations 192
Table 99: OGFC Properties 193
Table 100: Test Matrix 195
Table 101: Materials and Mixes Used in This Study 202
Table 102: Tests for Different Properties 203
Table 103: Composition of Mix Design Blends 211
Table 104: Composite Blends and Mixture Properties 211
Table 105: Summary of Deicing/Anti-icing Techniques 213
Table 106: Summary of Friction Numbers Measured on the Test Lane and Control Lane 214
Table 107: Properties of Materials and Information on Mix Design 221
Table 108: Results of Mix Design 222
Table 109: Conclusions from Comparison of SHRP Model Predicted Versus Measured Temperatures 227
Table 110: Conclusions from Comparison of LTPP Model Predicted Versus Measured Temperatures 228
Table 111: Materials and Mixes Used by Watson et al 229
Table 112: Tests for Different Properties 231
Table 113: Materials and Mixes 234
Table 114: Mix Design Information 238
Table 115: Mix Design Information Continued 239
Table 116: Mix Design Information Continued 239
Table 117: Description of Pavement Structure and Construction 240
Table 118: Details of Tests Conducted 241
Table 119: Typical Percolation Speed of Porous Asphalt in France 245
Table 120: Reduction in Noise Levels at Various Locations 250

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Table 121: Aggregate Grading Used in This Investigation 252
Table 122: Materials, Mix Design and Test Results 257
Table 123: Results of All Sound Measurements 259
Table 124: Results of Surface Texture Measurement 259
Table 125: Results of Friction Measurement 260
Table 126: Variability of Test data from OGFC Projects (25 projects, 319 data points) 265
Table 127: Specification Limits for OGFC 266
Table 128: Pay Table for Small Production 267
Table 129: General Climatic Statistics for the Netherlands 269
Table 130: Recommend Spreading Rates 269
Table 131: OGFC Placement Temperatures 277
Table 132: OGFC Temperature Limits 278
Table 133: NCAT Recommended Aggregate Properties 282
Table 134: NCAT's recommended Gradation Requirements 282
Table 135: Specifications for Selecting Optimum Asphalt Binder Content 282
Table 136: TxDOT Master Gradation Band and Binder Content 283
Table 137: Summary of Design Requirements in Australia 285
Table 138: Typical Reported OGFC Mixture Service Life 291
Table 139: Gradation Requirements in Switzerland 294
Table 140: Gradation Requirements in Japan 295

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Task 1 - Conduct a Comprehensive Review of Worldwide Literature on PFC

The project statement for NCHRP 9-41 identifies Task 1 as a comprehensive review of worldwide literature dealing with (1) design methodology; construction, maintenance, and rehabilitation strategies; safety; and performance of PFCs, conventional OGFC and similar materials and (2) their advantages and disadvantages. The project also calls for the reviews to be annotated. Because NCHRP 9-41 did not entail any field or laboratory work, the literature search and review was of paramount importance to the successful completion of this project.

Individual summaries of the various items found in the literature follow. Each of the summaries are divided into nine subheadings: general, benefits, materials and mix design, construction practices, maintenance practices, rehabilitation practices, performance characterization, structural design and limitations.

The "General" Subheading within each review provides a brief summary of the information contained in the document. The remaining eight subheadings are issues that were of particular interest to the successful completion of this project. Organizing each review in this manner may appear somewhat redundant; however, this organization assisted the researchers when developing the state of practices and guidelines.

Within some of the reviews, comments by the authors of this report are provided to clarify or to add information that was not contained within the original text but might be of interest. These comments are provided within brackets. The reviews are provided in chronological sequence. The State of Practice contained within Volume I of this report is a synthesized version of the document. The reader should be cautioned that units within this appendix are not consistent. Units within each review represent the units utilized by the respective authors.

1.1 National Roads Authority (NRA). *Porous Asphalt in Ireland* http://www.fingalcoco.ie/motorway2/ASPHALT.HTML

1.1.1 General

This website describes the use of porous asphalt in Ireland, the reasons for its use, advantages of its use, and brief information on a typical mix design.

Porous asphalt was first introduced in Ireland within the past five years [from the time of the article, which is unknown], although it has been used in other parts of Europe for the past decade, mainly France and Belgium. Because of the increased volume of traffic and a rather wet climate in Ireland, a pavement that would allow water to drain with more ease was needed.

Also, because of the high volume of traffic, a pavement that reduces noise was also a main concern.

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1.1.2 Benefits of Permeable Asphalt Mixtures
Porous asphalt allows water to drain from the surface of pavement because of its gradation and aggregate make-up. Because of the high air-void content in the mix, the surface water can drain, which reduces road spray, improves skid resistance and decreases road glare (increasing vision).

1.1.3 Materials and Mix Design
Irelands' porous asphalt has a nominal maximum aggregate size of 14mm. The aggregates are gap-graded and held together with a binder that is polymer modified. The air voids in the porous asphalt are approximately 20 percent. In the design, an inverted pavement surface texture is created which allows for acoustic absorption, which will reduce noise level.

1.1.4 Construction Practices
No specifics on construction practices were given.

1.1.5 Maintenance Practices
No specifics on maintenance practices were given.

1.1.6 Rehabilitation Practices
No specifics on rehabilitation practices were given.

1.1.7 Performance
Because of the performance of porous asphalt with respect to reducing noise levels, water drainage and friction, it is being used on many roads in Ireland.

1.1.8 Structural Design
No specifics on inclusion within structural design were given.

1.1.9 Limitations
No specific limitations were given.

1.2 Decoene, Y. "Contribution of Cellulose Fibers to the Performance of Porous Asphalts." Transportation Research Record No. 1265. Transportation Research Board. National Research Council. Washington, D.C. pp 82-86. 1990.

1.2.1 General
The use of porous asphalt mixes has increased in Belgium due to the many benefits that they provide. Many roadways have been covered with a porous asphalt wearing layer and the performance has been good. Decoene states that the performance has been good even though unmodified binders have been used. However, in some instances problems have occurred due to the draining of the asphalt binder from the aggregate structure. As a result of this draindown problem, raveling has occurred in areas with low asphalt binder contents. In order to solve the draindown problem and to allow porous asphalt to be placed with higher asphalt binder contents (for better durability), fibers can be added to

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the mixture. Small amounts of asbestos fibers had been used; however, because of the health issues related to asbestos, Belgium decided to investigate the use of cellulose (organic) fibers within porous asphalt. This paper summarizes research conducted to evaluate the use of cellulose fibers within porous asphalt.

1.2.2 Benefits of Permeable Asphalt Mixtures

Benefits associated with porous asphalt layers cited by Decoene included improved skid resistance, reduced hydroplaning, reduced splash and spray, improved visibility, reduced tire/pavement noise and resistance to permanent deformation.

1.2.3 Materials and Mix Design

Organic fibers used within the research study were characterized as gray fibers with a cellulose content of at least 75 percent and a pH of between 6 and 8.5. The length of fibers was 1.2 mm and the density was 1.5 g/cm^3. Decoene stated that the fibers were resistant to temperatures up to 180ºC (356˚F).

Laboratory evaluation of the cellulose fibers within porous asphalt included testing for draindown potential, the influence on compaction (in terms of air voids) and durability. Two draindown potential tests were utilized: a Basket Drainage test and the Schellenberger Drainage test.

During the basket Drainage testing, a total of seven porous asphalt mixes were evaluated (Table 1). Of these seven mixes, cellulose fibers were added to six of the mixes at either a rate of 0.3 percent or 0.5 percent, by total mix mass. In order to conduct the Basket Drainage test, the mixes were first compacted in Duriez molds under a pressure of 30 bars (435 psi). The molds containing the compacted porous asphalt were then placed in a grid pattern within an oven maintained at 180ºC. The samples were held at this elevated temperature for 7.5 hours. During the course of the test, asphalt binder would drain from the compacted samples. At the conclusion of the test, the percent binder loss was calculated based upon the initial asphalt binder content. Results of this Basket Drainage testing are provided in Table 1.

Table 1: Results of Basket Drainage Tests

Composition	Mix Designation						
	A	B	C	D	E	F	G
Diorite (10/14)	55.5	55.5	55.5	55.5	55.5	55.5	55.5
Diorite (6/10)	30	30	30	30	30	30	30
Sand (0/2)	13	12	12	12	12	12	12
Filler	1.5	2.2	2.2	2.2	2.0	2.0	2.0
Organic Fibers	0	0.3	0.3	0.3	0.5	0.5	0.5
Bitumen (80/100)	4.7	5.5	5.7	5.9	5.5	5.7	5.9
Binder Loss, %	13.5	1.5	2.9	3.4	0.3	1.3	1.2

Based upon the data presented in Table 1, Decoene provided several observations. First, the porous asphalt mixture without any fibers (Mix A) lost 13.5 percent of its binder

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during the test, even at the low asphalt binder content. For the six mixes that did contain cellulose fibers, the amount of binder loss was significantly lower than the mix with no fibers. There was very little difference in the drainage characteristics between mixes containing 0.3 percent fiber and those having 0.5 percent fiber.

The second test method employed by the author that measured the potential for draindown was the Schellenberger Drainage test. This test entails placing 1,000 to 1,100 grams of porous asphalt into a glass beaker and then placing the sample into an oven for 1 hour. Similar to the Basket Drainage test, results are reported as the percent binder loss as a percentage of the asphalt binder content. For this experiment, a total of 12 porous asphalt mixtures were utilized as shown in Table 2. In order to evaluate different sources of cellulose fiber, Decoene utilized four different sources of cellulose fibers.

Table 2: Results of Schellenberger Drainage Tests

Composition	Mix Designation											
	A	B	C	D	E	F	G	H	I	J	K	L
Durite (10/14)	55.5	55.5	55.5	55.5	55.5	55.5	55.5	55.5	55.5	55.5	55.5	55.5
Durite (6/10)	30	30	30	30	30	30	30	30	30	30	30	30
Sand (0/2)	12	12	12	12	12	12	12	12	12	12	12	12
Filler	2.5	2.5	2.5	2.5	2.2	2.2	2.2	2.0	2.2	2.2	2.5	2.2
Fiber A	-	-	-	-	0.3	0.3	-	-	-	-	-	0.3
Fiber B	-	-	-	-	-	-	0.3	0.5	-	-	-	-
Fiber C	-	-	-	-	-	-	-	-	0.3	-	-	-
Fiber D	-	-	-	-	-	-	-	-	-	0.3	-	-
Binder 60/70	4.5	4.7	4.9	4.9	5.9	6.1	5.9	5.9	5.9	-	-	-
Binder 80/100	-	-	-	-	-	-	-	-	-	5.9	4.7	5.9
Binder Loss, %	15	16	21	17	0.1	1.1	1.1	0.3	0.6	1.1	18	0.5

Results of the Schellenberger Drainage testing yielded similar results as the Basket Drainage test. Mixtures without fibers had significantly more binder loss (draindown). This again suggested that the fibers help hold the asphalt binder to the aggregate structure of a porous asphalt. Whether 0.3 percent or 0.5 percent fiber was used in the mix did not appear to significantly influence draindown potential. The data also indicated that relatively high asphalt binder contents could be used with the porous asphalt containing cellulose fibers with little potential for draindown problems.

The Belgian Road Research Centre had recommended air void contents of Marshall compacted specimens and Cantabro Abrasion tests during the design of porous asphalt. Therefore, Decoene evaluated these properties on four mixes with and without fibers. Based on the results of this testing, there was little difference in air voids of mixes with and without fibers. Also, the Cantabro loss was deemed acceptable for all four of the mixes.

1.2.4 Construction Practices

Cellulose fibers are supplied to the plant site wrapped in polyethylene packaging. This packaging has a relatively low melting point that will dissolve if the cellulose fiber bag is added to the production process. Alternatively, cellulose fibers can also be pre-coated with some amount of asphalt binder in the form of a pellet. These pelletized fibers are specifically intended for drum mix plants.

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1.2.5 Maintenance Practices
No specific maintenance practices were given.

1.2.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.2.7 Performance
When describing a full scale pavement experiment within the paper, Decoene indicated that permeability testing was included at the time of construction as a performance indicator. No details on the test method or typical results were provided.

1.2.8 Structural Design
No specifics on inclusion within structural design were given.

1.2.9 Limitations
No limitations on use were given.

1.3 Isenring, T., H Köster and I. Scazziga. "Experiences with Porous Asphalt in Switzerland." Transportation Research Record No. 1265. Transportation Research Board. National Research Council. Washington, D.C. pp 41-53. 1990.

1.3.1 General
The first porous asphalt in Switzerland was placed in 1972 on an airport runway. For highway pavements, porous asphalt has been used since the late 1970's and early 1980's. This paper presents the results of a research program where 17 pavements were monitored during the life of the various porous asphalt layers.

1.3.2 Benefits of Permeable Asphalt Mixtures
Benefits of porous asphalt highlighted by the authors included:
- Reduction in the potential for hydroplaning.
- Reduction in splash and spray.
- Good frictional properties at higher speeds.
- Reduction in noise.
- Reduced glare at nighttime and in wet weather.
- Resistance to permanent deformation.

1.3.3 Materials and Mix Design
Table 3 presents typical properties of porous asphalt in Switzerland. The authors also stated that polymer modified binders are generally used with porous asphalt.

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Table 3: General Data on Materials and Mix

Property	Porous Asphalt (0/10)	Porous Asphalt (0/16)
Max. Aggregate Size (mm)	10	16
Layer Thickness (mm)	28-42	43-50
Binder Content (%)	4.65 – 5.82	4.23-4.99
Air Voids (Marshall, %)	10.9 – 22.5	14.9 – 17.0
Air Voids (Cores, %)	14.6 – 21.1	14.6 – 19.6

1.3.4 Construction Practices

No specific construction practices were given.

1.3.5 Maintenance Practices

For general maintenance, the authors indicate that the cleaning of porous asphalt layers that have become filled with debris can be very difficult. High pressure water with subsequent vacuuming has been used to clean porous pavements; however, this technique has not been very successful in restoring permeability. The authors state that if cleaning techniques are begun while the layer is still permeable, instead of clogged, results should be to maintain permeability for a longer period of time.

The authors used skid testing to evaluate the behavior of porous asphalt wearing layers during winter conditions. A skid trailer was used for this testing, with the wheel in a braking condition instead of a locked condition. This was done because of the possible formation of a snow or slush wedge in front of a locked wheel.

The authors state that porous asphalt wearing layers are very variable during winter conditions and can change rapidly. Generally, the frictional properties of the porous asphalt layers were similar to comparison dense-graded layers. It was also noted that porous asphalt layers that had lost the ability to drain water behaved very similar to dense-graded layers. Differences in behavior between porous asphalt and dense-graded layers mainly occurred on heavily trafficked roads where traffic does not pass (i.e., between wheel paths and edges of road).

Some advantages of porous asphalt during winter conditions include: ice does not generally form on wet porous asphalt surfaces because of the ability to drain water and the good macrotexture exhibited by porous asphalt; the high level of macrotexture is beneficial when snow and slush exist; and the tendency for ice formation within wheelpaths covered with snow is reduced due to the macrotexture, water absorption (draining of water) and limited thaw. Disadvantages of porous asphalt during winter conditions include: the need for deicing salts and other thawing products; the use of sand and small aggregates to improve frictional properties is not possible because they clog the void structure of the porous asphalt layer; snow and ice tend to stick to the porous asphalt layer sooner because the surface is generally cooler by about 0.5°C; snow and icing rain can form earlier on porous asphalt because deicing salts do not remain on the surface; preventative salting is not as beneficial because the salt penetrates into the void structure; if the porous asphalt layer drainage capacity is reduced, ice can build up within the layer and expand onto the pavement surface; and some icing problems can occur within the

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initial portion of a subsequent dense-graded surface which does not receive salt through transportation by traffic.

1.3.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.3.7 Performance
The first performance issue discussed by the authors was frictional properties. Frictional properties were measured once or twice a year with a skid trailer. This trailer allowed the testing of the wearing layer with either a braked or locked wheel using a water film. Initial testing of each pavement was conducted within two months of opening to traffic. Within this investigation, locked-wheels were used to evaluate frictional properties. Based upon the friction testing, the authors state that vehicle speed does not influence the frictional properties of porous asphalt wearing layers as much as typical dense-graded layers due to the higher level of macrotexture. On lower vehicle speed roadways where microtexture in more important, the authors indicate that porous asphalt has lower (but acceptable) frictional resistance. Because of the aggregate structure of porous asphalt, vehicle tires are more in contact with individual coarse aggregates than a pavement surface. For this reason, the polish resistance of the coarse aggregate is important.

The next performance indicator utilized by the authors was a field permeability test. Permeability was defined as a performance measure because the benefits derived from porous asphalt are related to the ability of the layer to drain water (or allow noise absorption). One of the first tasks related to the research project was to identify a method for measuring the permeability characteristics of porous asphalt. Several devices were considered, however most of the devices considered were insufficiently precise or too complicated for use. Therefore, the researchers developed a new methodology, the IVT Permeameter. This device is made of a plexiglass cylinder having an interior diameter of 190 mm and a height of 250 mm. The plexiglass cylinder contains five engraved markings that are 20 mm apart, with the “zero” marking being a height of 250 mm above the pavement surface. Special putty is used to seal the flow of water through the surface texture of the porous asphalt. Permeability is expressed as the time required for water to travel between the “zero” mark and the 80 mm marking. This will result in 2.27 liters of water flowing into the pavement. The authors state that this test of permeability is a single point measurement and that a number of tests should be conducted in order to adequately characterize the properties of the pavement.

Initial testing with the developed permeameter showed a water level decrease of 80 mm ranging from 23 to 105 seconds [shorter times reflect more permeability]. Permeability of the porous asphalt wearing layers decreased over time and the rate was pavement specific. The authors listed a number of causes for this reduction in permeability. First, dust and debris can fill the void structure of the porous asphalt layer. Secondly, a slight consolidation of the porous asphalt layer will reduce permeability from the initial values. Other factors that can affect the reduction in permeability include environment (amount of rain) and type of traffic volume. Typically, permeability will be maintained longer within the wheel paths. Wheel paths will maintain permeability longer because of the

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cleaning suction action caused by tires traveling over the layer. The authors state that some porous asphalt wearing layers will maintain permeability for more than 5 years and some will become almost impermeable within one year.

When comparing the permeability of mixes having different maximum aggregate sizes (10 mm or 16 mm), permeability values were similar at the time of construction. However, porous asphalt mixtures having a maximum aggregate size of 16 mm tended to have higher permeability values for a longer period of time. The authors listed a number of favorable conditions for maintaining permeability:

- Reduced amount of dirt and debris.
- Good drainage (daylighted pavement edge, sufficient cross slope in underlying layer)
- Layer with high percentage of larger air voids
- Cleaning action of rapid and intense traffic.

Another performance measure discussed within the paper was traffic noise. Traffic noise was evaluated in three ways: measurement of tire/pavement noise using a noise trailer, measurement of roadside noise, and measurement of sound absorption due to the pavement surface. Five values were used from these measurement techniques to compare the noise levels of porous asphalt wearing layers:

- Degree of noise reflection by the pavement surface and the quantity of sound absorbed by the pavement.
- LMA-value: Value of tire/pavement noise determined from the noise trailer.
- Coasting noise: sound level of a passenger car rolling by with the engine turned off determined by a wayside measurement.
- Traffic noise: sound level of a passenger car passing at a constant speed determined by a wayside measurement.
- Traffic noise (L_{eq}): the energy-equivalent continuous sound of a traffic stream determined by a wayside measurement.

Measurement of sound absorption, or reflection of sound, can be done either in the laboratory using an impedance tube or in the field using special equipment. The research showed that porous asphalt layers that are in good functional condition (permeability has been maintained) are capable of absorbing sound. A maximum of about 20 percent of the sound was absorbed. The authors state that layers thicker than the 50 mm (maximum) used in Switzerland had the potential for absorbing more sound. The authors also showed a relationship between the permeability of the porous layers and the ability to absorb sound. As permeability increased, sound absorption also increased. However, the authors stated that surface texture seems to be more important than permeability. Several pavements exhibiting relatively low permeability values (clogged) still provided a reduction in noise levels.

When using the noise trailer, the authors found that porous asphalt layers that were in good functional condition resulted in lower noise levels than typical dense-graded layers. At speeds less than 50 to 60 km/hr, the noise levels were similar. The difference in noise

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levels increased as speed increased above 50 to 60 km/hr. This indicated to the authors that a reduction in noise levels is primarily seen and most effective at higher speeds. Results from testing with the noise trailer also showed a relationship between permeability and noise levels. As permeability increased, noise levels generally decreased. Also, porous asphalt mixes having a smaller maximum aggregate size or more continuous grading have a lower noise level than coarser porous asphalt layers.

Wayside measurements were conducted on sections of roadway where a comparison between porous asphalt and typical dense-graded layers could be made. For single vehicle cars, a level of reduction between 1 and 5 dB(A) was observed between porous asphalt and the dense-graded layers. Noise levels for a traffic stream (Leq) showed reductions between 0 and 3.5 dB(A).

Besides the reduction in noise levels, the authors also found that porous asphalt reduces disturbing noise at higher sound frequencies. By removing the disturbing higher frequency noise, persons living near a roadway having a porous asphalt wearing layer provided positive comments.

1.3.8 Structural Design

The only specifics on structural design mentioned by the authors was that typical layer thicknesses ranged from 28 to 50 mm. Porous asphalt mixes having a maximum aggregate size of 10 mm are typically placed 28 to 42 mm thick while porous asphalt mixes having a maximum aggregate size of 16 mm are typically placed from 43 to 50 mm thick.

1.3.9 Limitations

Disadvantages of porous asphalt highlighted by the authors included:

- Reduction in the advantages listed above over time due to clogging of the layer.
- Unknown durability of porous asphalt layers.
- Special requirements (e.g., lateral drainage in urban areas).
- Unfavorable frictional properties at lower speeds.
- Different winter maintenance requirements.
- Difficulty in conducting repairs.
- Costs

1.4 Ruiz, A., R. Alberola, F. Pérez, and B. Sánchez. "Porous Asphalt Mixtures in Spain." Transportation Research Record No. 1265. Transportation Research Board. National Research Council. Washington, D.C. pp. 87-94. 1990.

1.4.1 General

This paper describes the use of porous asphalt mixtures in Spain up to 1990. Ruiz et al indicate that the first application of porous asphalt in Spain was placed in 1980 on four experimental sections in northern Spain. The purpose of the trial sections was to improve safety in rainy areas. In 1986, the use of porous asphalt began to increase because doubts about the durability of porous asphalt were eliminated. Based upon the success of porous

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asphalt mixtures, it was not only used to provide a safe wearing course but also to provide a durable surface with a smooth, safe and quiet ride in any type of weather.

Ruiz et al indicate that as of 1990, approximately 3 million square meters of porous asphalt mixture had been placed. Porous asphalt was being used for all types of roadways and all types of traffic conditions. The most common practice was to place porous asphalt at a thickness of 40 mm.

In addition to being used as the wearing course of new pavements, Ruiz et al indicate that the main application of porous asphalt mixtures had been to repair aged or slippery surfaces of existing pavements that had not shown structural problems. Porous asphalt mixtures are also used in short stretches (300 m) in areas that are difficult to drain (e.g. horizontal and vertical curves.

1.4.2 Benefits of Porous Asphalt Mixtures
Ruiz et al indicate that porous asphalt mixtures provide a durable surface with a smooth, safe and quiet ride.

1.4.3 Materials and Design
Spain utilizes two porous asphalt gradation bands, P12 and PA12 (Table 4). Ruiz et al indicate that selection of the aggregate gradation influences the water drainage capacity, resistance to particle loss (durability), resistance to rutting and the macrotexture of the pavement surface. The P12 grading band results in porous mixtures with air void contents between about 15 and 22 percent, while the PA12 results in air void contents up to 25 percent.

Table 4: Gradation Bands for Porous Asphalt Mixtures in Spain

	Gradation Bands, Percent Passing	
Sieve Size, mm	P12	PA12
20 mm	100	100
12.5 mm	75-100	70-100
10 mm	60-90	50-80
5 mm	32-50	15-30
2.5 mm	10-18	10-22
0.63 mm	6-12	6-13
0.08 mm	3-6	3-6

The aggregate gradation bands both contain a large percentage of coarse aggregate (defined as larger than 2.5 mm) in order to accommodate the other components of the porous mixture. Ruiz et al indicate that selection of the amount of fine aggregate is important. The fine aggregate content must be low enough to prevent closing up of air voids and must not separate the coarse aggregate particles. Separation of the coarse aggregate particles will increase the potential for rutting. Ruiz et al also indicate that some amount of filler (finer than 0.075 mm material) is needed to give cohesion to the mixture to help prevent particle loss.

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Ruiz et al provide recommendations for aggregate properties in the form of Los Angeles Abrasion, flakiness index, particles with two or more fractured faces and sand equivalency. [These properties can be generally characterized as toughness, shape, angularity and cleanliness, respectively.] Because aggregate breakdown can lead to particle loss, raveling and the closing up of the surface texture, Spain requires a Los Angeles Abrasion value of 20 percent maximum. For the same reasons, a flakiness index of 25 percent or less is also required. To ensure angular aggregates, Spain requires 75 percent or more coarse aggregate particles with two or more fractured faces. The sand equivalency value for fine aggregates must be higher than 50 percent. Non-polishing aggregates are also required to maintain a good, durable microtexture on the pavement surface. The Spanish specification sets the polished stone value above 0.45 for traffic volumes greater than 800 trucks a day per lane and 0.40 for all other traffic categories.

Ruiz et al indicate that stiff asphalt binders are needed for porous asphalt mixtures. Stiff binders are needed to resist against particle loss and to get longer durability through thicker asphalt binder films. In 1990, approximately 80 percent of all porous asphalt mixtures utilize polymer modified binders. The most common types of polymers are EVA and SBS.

The philosophy of designing porous asphalt mixtures in Spain is two-fold: provide a minimum binder content to assure against particle loss and a maximum binder content to avoid draindown. The resistance to particle loss is determined through the use of the Cantabro Abrasion test. This test is used to determine the minimum asphalt binder content that will provide a maximum of 25 percent particle loss. No specific test was specified at the time of this paper to evaluate the draindown potential for porous asphalt mixtures. Air void contents are calculated during mix design. The minimum air void content of 20 percent defines the maximum asphalt binder content. Ruiz et al indicate that binder contents are generally 4.5 percent.

1.4.4 Construction Practices

Ruiz et al indicate that whenever porous asphalt is placed over an existing pavement, all distressed areas must first be repaired and the surface leveled. The underlying layer must also be impermeable and structurally sound. Ruiz et al recommend a quick setting emulsion be placed over the underlying layer at a residual rate of 500 to 600 gr/m^2. Underlying layers that are open or highly polished may require a slurry seal prior to placement of the porous asphalt layer.

Porous asphalt mixtures in Spain are produced in typical HMA production facilities. Production rates should correspond to the paving equipment such that stops of the paving train are minimized. Mixing temperatures are generally 140 to 150°C (280 to 300˚F) with the temperature never exceeding 160°C (320˚F).

During transportation, trucks should be covered to prevent temperature loss. Porous asphalt should not be placed when the temperature is less than 8°C (4˚F). The mix temperature should not get below 120°C (250˚F) during compaction. Compaction of the mixture on the roadway is conducted with steel-wheel rollers, without vibration, having a

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total weight of 10 metric tons. Two steel-wheel rollers are generally used. The first roller makes 4 to 5 passes and the second roller makes 2 to 3 passes to smooth out all roller marks.

Two methodologies of constructing shoulders with porous asphalt have been used in Spain. First, the porous asphalt has been extended over the entire shoulder while the second method entails extending the porous asphalt 50 cm (1.6ft) onto the shoulder.

For quality control of the production process, samples are prepared and the air voids are controlled. The amount of compaction is controlled in the field by means of a permeability test. The permeability test uses water and was developed at the University of Santander.

1.4.5 Maintenance Practices
The primary problem with porous asphalt in Spain has been particle loss. Particle loss generally occurs shortly after traffic. Ruiz et al indicate that this problem generally originates from placing the porous asphalt too cold, not enough compaction, or from draindown problems. The maintenance activities generally consist of milling the section in question and filling with new porous asphalt mixture. In one case, the poor performing section was overlaid by a new porous asphalt mixture, with no problems as of publication date.

Spain's experience with winter maintenance has not been extensive. In areas of high snow falls, porous asphalt is generally not used. In warmer areas, when winter maintenance is needed, more salt and an increase in salting are generally used. Ruiz et al indicates that approximately double the amount of salt is needed.

1.4.6 Rehabilitation Practices
No specifics of rehabilitation were described in the paper.

1.4.7 Performance
The authors indicate that the performance of porous mixtures with more than 20 percent air voids has been much better than porous asphalt mixtures with 15 to 18 percent air voids. Better durability and less clogging has been noted in the mixes having more than 20 percent air voids.

1.4.8 Structural Design
A lift thickness of 40 mm has been established in Spain. The possibility of using thicker lifts has not been considered. Ruiz et al indicate that the water absorption capacity with 40 mm is thought sufficient; therefore, the thickness specified in Spain is based upon the volume of potential rainfall. Ruiz et al also indicate that the same structural value is assigned to porous asphalt mixtures as other open or semi-open conventional asphalt mixtures such as road bases. When porous asphalt mixtures are used above pavement structures containing cement-treated road bases, an additional 20 mm of hot mix asphalt is provided to assist in preventing reflective cracking. Reflective cracking that appears in

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a porous asphalt mixture will provide an avenue for water to penetrate into the pavement structure, thereby, increasing the potential for pavement deterioration.

1.4.9 Limitations

Ruiz et al indicate that the use of porous asphalt mixtures should be carefully studied prior to being placed in the following situations:

- Areas of frequent snow
- Urban or industrial areas
- Areas with a high potential for reflective cracking
- Bridge pavements.

1.5 Van Der Zwan, J.T., T. Goeman, H.J.A.J. Gruis, J.H. Swart, and R.H. Oldenburger. "Porous Asphalt Wearing Courses in the Netherlands: State of the Art Review." Transportation Research Record No. 1265. Transportation Research Board. National Research Council. Washington, D.C. pp 95-110. 1990.

1.5.1 General

This paper presents a detailed summary on the use of porous asphalt in the Netherlands. The Netherlands is located in northwest Europe with a temperate climate. Average temperatures during January and July are 1.7ºC (35ºF) and 17ºC (63ºF), respectively. Annual precipitation is almost 800 mm (32 in) with the precipitation distributed throughout the year. Because of the amount of precipitation, the authors indicated that road surfaces tend to be wet about 13 percent of the time.

Porous asphalt was first utilized in the Netherlands in 1972. Since that time, many pavement sections have been placed that included porous asphalt wearing layers.

The paper discusses all aspects of porous asphalt use within the Netherlands. Information provided within the paper was used to conduct a life-cycle cost analysis to compare porous asphalt wearing layers and dense-graded wearing layers. Based upon the results of the cost-benefit analysis, the Dutch Department of Works decided that porous asphalt wearing layers would be preferable on the following types of pavements:

- Busy motorways with an average of more than 35,000 vehicles per day.
- Limited access roadways that do not allow slow moving traffic.
- At discontinuities such as superelevations where excess water may cause difficulties.
- On roadways with a recognized noise nuisance problem.

1.5.2 Benefits of Permeable Asphalt Mixtures

Benefits noted by the authors centered on improvements in safety for the traveling public. These benefits were directly related to the ability of porous asphalt to remove water from the pavement surface. By eliminating water films from the pavement surface, porous asphalt layers reduce the amount of splash and spray and improve the visibility of pavement markings. Additionally, the authors state that hydroplaning is basically eliminated. Skid resistance on wet pavements is also increased.

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The authors did note that porous asphalt layers reduce tire/pavement noise by approximately 3dB(A) when compared to typical dense-graded wearing layers.

1.5.3 Materials and Mix Design

The authors state that the porous asphalt mixture being utilized in the Netherlands [as of 1990] was very similar to the OGFC mixes used in the U.S. Table 5 presents typical requirements for porous asphalt in the Netherlands. In order to improve bonding between the aggregates and asphalt binder, the authors state that a limestone filler is added during the production process. The limestone filler must have a hydrated lime content of at least 25 percent. During design, a minimum air void content of 20 percent is required. A dynamic bending test is used to evaluate the stiffness of designed porous mixtures. Wheel tracking tests are also used to evaluate resistance to rutting. The authors indicate that the results of the dynamic bending tests (in terms of modulus) are only about 20 percent of typical dense-graded mixes. However, the wheel tracking tests indicate that porous asphalt has much less potential for permanent deformation.

Table 5: Typical Requirements for Porous Asphalt Mixes in the Netherlands

Sieve, mm	Percent Mass Passing, %			
	Target	Maximum	Minimum	Tolerance
16		100	96	± 1.0
11.2		85	70	± 8.0
8		50	35	± 7.0
5.6		30	15	± 7.0
2	15			± 4.0
0.063	4.5			± 1.0
Binder Content (80/100 binder)	4.5			± 0.5

1.5.4 Construction Practices

The authors indicate that all HMA is produced in the Netherlands using a batch plant facility. This makes the production of porous asphalt much more straightforward than the use of other plant types.

During construction, the authors state that handwork is very difficult and should be avoided. Compaction of porous asphalt is best achieved using static steel-wheel rollers. The temperature of the mixture during compaction is critical. If temperatures are too high, the mortar may drain from the aggregate structure. If the temperatures are too low, the mixture will be difficult to compact. Ideal placing and compaction temperatures are between 140 and 170ºC (280 to 340ºF). The authors indicate that the costs for placing and compacting porous asphalt are comparable to placement and compaction of dense-graded mixes.

1.5.5 Maintenance Practices

The authors provide issues with dealing with porous asphalt during wintry conditions. Icy roads are a recurrent problem in the Netherlands; however, snowfall is not a major

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problem in the temperate climate of the Netherlands. Salting operations are the typical method of dealing with icy roads. Electronic monitoring systems were being installed along the main road network [at the time of this paper] to assist in selecting the appropriate time for conducting winter maintenance. Field measurements have shown that porous asphalt layers remain below 0ºC (32˚F) longer than dense-graded wearing layers. As a result, ice problems are likely to develop sooner and last longer on porous asphalt wearing layers.

Due to the open nature of porous asphalt, salt applications will disappear into the void structure of the layer. This will be further exacerbated by the salt being removed from the pavement surface by melting ice. As a result, the time that salt remains on the pavement surface is relatively short compared with dense-graded wearing layers.

The authors note that special attention must be paid to transitions between porous asphalt and dense-graded wearing layers. At these transitions, there is little salt transport. Experience has shown, however, if pre-wetted salt is applied that these transition points are not as big of a problem.

In conclusion, the authors state that porous asphalt wearing layers are considered as safe as dense-graded wearing layers during the winter, provided that timely measures are taken into account for the different behaviors of the two wearing layers.

1.5.6 Rehabilitation Practices
Minor rehabilitation strategies are similar to conventional dense-graded layers; however, the authors state that the inherent drainage characteristics of the porous asphalt should be maintained. The preferred method of rehabilitating porous asphalt layers is to mill the existing layer and replace with a new wearing layer.

1.5.7 Performance
The authors state that the service life of porous asphalt layers is about 10 years. This is compared to the expected 12 year service life of dense-graded wearing layers. The most common distress encountered on porous asphalt layers is raveling, with the most potential for raveling being within the first year.

1.5.8 Structural Design
Porous asphalt layers are placed at a thickness of 50 mm. This thickness was selected based upon the typical rainfall rates experienced within the Netherlands.

The pavement design methodology in the Netherlands entails designing to prevent classical bottom-up fatigue cracking. When designing pavement thicknesses, dense-graded mixes are assigned a dynamic modulus value of 7,500 MN/m^2 and the mixture must also meet specific fatigue properties. The authors provided a discussion on the structural contribution of a porous asphalt layer within pavements by comparing dense-graded and porous asphalt.

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One area that dense-graded and porous asphalt mixes were compared was in terms of dynamic modulus. The authors indicated that the dynamic modulus of porous asphalt is generally 5,400 MN/m^2, or about 70 to 80 percent of dense-graded mixes. This value of dynamic modulus was input into their pavement design models and the results indicated that 10 to 20 percent more thickness was required to maintain a specific fatigue strain at the bottom of the pavement layer when using porous asphalt as compared to dense-graded mixes.

The authors also evaluated the effect of aging and stripping on pavement design. Due to the open nature of porous asphalt, the asphalt binder coating aggregates is susceptible to accelerated oxidative aging. Oxidative aging of the asphalt binder results in an increase in stiffness within the porous asphalt layer which would reduce pavement thickness. Alternatively, the authors state that water within the porous asphalt layer can lead to a loss of adhesion between the porous asphalt layer and the underlying layer. This loss of adhesion impairs the load transfer characteristics of the structure. The authors state that there is no evidence that the loss of adhesion between layers [delamination] has taken place in the field; they conservatively assumed a loss of adhesion to evaluate the net effect on pavement structure using the BISAR program. When delamination occurs, the effective bearing capacity of the debonded layer is reduced to between 2 and 10 percent of the original value. By applying Miner's modified linear damage law, the authors stated that the combined effect of aging and stripping [delamination] would result in about 35 to 40 percent effective contribution of porous asphalt when compared to dense-graded layers.

Because porous asphalt has different thermal properties, the authors also evaluated the effect of temperature on pavement structure when comparing dense-graded and porous asphalt wearing layers. Van der Zwan et al provided a hypothesis that the suction and pumping action of tires passing over porous asphalt surface, coupled with wind action, promotes continuous air circulation within a porous asphalt layer. As a result, the temperature of the porous asphalt layer will tend to be lower than for comparable dense-graded layers. In order to investigate this hypothesis, the authors conducted experiments on newly placed and 8-year old porous asphalt layers to compare the temperatures of pavements with porous asphalt and dense-graded wearing layers at the surface and at depth. Results from these experiments, which included 1-year of data, indicated that the weighted average temperature over a year was found to be 1ºC lower in pavements containing porous asphalt wearing layers. Due to the viscoelastic properties of asphalt, the lower effective temperature in pavements including a porous asphalt wearing layer means that the stiffness (modulus) of layers within these pavements is higher. The net result being that less thickness is required to resist fatigue cracking.

The combined effect of the above mentioned factors suggests that porous asphalt can be expected to contribute about 50 percent of the equivalent structural capacity compared to a dense-graded layer. However, if adhesion between the porous asphalt layer and the underlying layer is not lost (as was conservatively assumed), then the effective contribution of porous asphalt can be 100 to 110 percent of conventional systems.

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1.5.9 Limitations

No limitations on the use were given.

1.6 Van Heystraeten, G. and C. Moraux. "Ten Years' Experience of Porous Asphalt in Belgium." Transportation Research Record No. 1265. Transportation Research Board. National Research Council. Washington, D.C. pp 34-40. 1990.

1.6.1 General

Porous asphalt has been used in Belgium since 1979. Based upon the success of initial trials, more sections were placed starting in 1981. As of 1990, a total of about 70 projects encompassing approximately 2 million square meters of porous asphalt had been placed. The authors define porous asphalt as a bituminous road mixture designed so that after construction, the mixture will form a surface having an air void content of about 22 percent. These mixes must be designed so that water is drained through the layer laterally; therefore, the underlying layer must have some cross-slope to promote drainage through the layer laterally. Also, the underlying layer must be impervious so that water cannot infiltrate into the layer.

Porous asphalt is most commonly used in Belgium where water will tend to stagnate, like superelevations, wide pavements (motorways and airfield runways) and sags in horizontal curves. Another location that the authors indicate porous asphalt can be used is within tunnels. Any water that may seep up through the pavement would be carried by the porous laterally to a drainage system to remove the water. Other areas where porous asphalt is commonly utilized would be areas where traffic noise is an issue. The authors state that porous asphalt will provide a reduction in noise of 6 to 10 dB(A) compared with concrete pavements. In some instances, porous asphalt is used on noisy urban arterials, even curbed sections. When used on curbed sections, adequately designed drainage systems at the pavement edge are a necessity.

1.6.2 Benefits of Permeable Asphalt Mixtures

Porous asphalt makes it possible for the tires on vehicles to stay in contact with the pavement surface during wet weather, thus avoiding hydroplaning. Porous asphalt also eliminates splash and spray behind vehicles and avoids reflections from the pavement surface during the day and night.

The authors state that porous asphalt accounts for a great part of the success considered in reducing traffic noise both inside and outside vehicles. This reduction in traffic noise is due to the absorption of sound that occurs in the voids of the porous asphalt layer. Also, the void structure eliminates the air pumping at the tire/pavement interface.

1.6.3 Materials and Mix Design

The philosophy of designing porous asphalt in Belgium is to provide sufficient coarse aggregate in order to provide the gap-grading required of porous asphalt. Typically, 80 percent of the aggregate would be larger than 2 mm. The gap-grading is generally provided by omitting the 2/7mm or 2/10mm fraction from a 0/14 mm gradation size. Mix designers also try to minimize the amount of asphalt binder so that the voids are not filled

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with the asphalt binder. Table 6 presents typical requirements for porous asphalt in Belgium.

Table 6: Belgian Specification for the Composition of Porous Asphalt

Property	Specification
Grading	0/14 mm gap
Stones (≥ 2mm)	83 %
Crushed Sand (0.080 mm – 2 mm)	12 %
Filler (< 0.080 mm)	5 %
Binder	
- 80/100 bitumen	4 to 5 %
- Modified bitumen	4 to 5 %
- Rubber bitumen	5.5 to 6.5 %
Air Void Content	
- Average	19 to 25 %
- Individual	16 to 28 %

The mix design method used in Belgium includes to first determine the voids in coarse aggregate for the plus 2 mm aggregate fraction [though no test method was given]. The next step entails compacting mixture with the Marshall hammer [no blow count given] and evaluating air voids and the percent wear using the Cantabro Abrasion test method. Optimum asphalt binder content is selected such that the granular materials are coated correctly but not excessively while meeting all other criteria.

1.6.4 Construction Practices

Conventional batch plants are used to produce porous asphalt in Belgium. The authors note that care must be taken to ensure that the temperature of the mix does not exceed 170°C (338°F) in order to avoid draindown problems. Haul distances must also be minimized. As haul distance increases, the potential for draindown also increases. Paver operations during the placement of porous asphalt are very similar to that of typical dense-graded layers.

Static steel-wheel rollers are the recommended equipment for compacting porous asphalt. Vibrator rollers are not recommended because of the potential for degrading the aggregate within the mix. Pneumatic tire rollers are generally not used as there is generally a pick-up problem when compacting porous asphalt with these rollers. Care must be taken at longitudinal joints. The joints should not be tacked as this will obstruct the drainage of water across the layer.

Finished porous asphalt tends to stick to the tires of vehicles. Therefore, it is common practice in Belgium to place fines (<0.080 mm) onto the porous asphalt surface at a rate of 50 g/m^2.

1.6.5 Maintenance Practices

The authors state that porous asphalt and dense-graded HMA wearing layers do not behave differently in snowy weather if deicing salts are “intensively” spread on the pavement surface. If the deicing salts are not “intensively” spread, snow may remain on

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the surface of porous asphalt longer because the brine that is created due to the melting snow will penetrate into the void structure of the layer. Accidents have been reported on porous asphalt due to icy conditions when nearby dense-graded surfaces were not icy [no explanation of conditions were given].

1.6.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.6.7 Performance
During construction, a permeability test is used to evaluate the drainage characteristics of porous asphalt. A criterion of 1.4 liters of water flowing into the pavement in less than 60 seconds is used. The authors also state that porous asphalt will slowly clog with silts in places where traffic is not intense.

1.6.8 Structural Design
The authors state that there are two thicknesses used with porous asphalt: 25 and 40 mm. In order to maintain the drainage characteristics and noise reducing attributes for a longer period of time, the authors indicate that the 40 mm layer thickness is best.

The authors also stated that based upon modulus testing, porous asphalt constructed with an 80/100 penetration graded asphalt binder will contribute 73 to 79 percent of the structural capacity of typical dense-graded mixes.

1.6.9 Limitations
The authors listed several circumstances where porous asphalt should not be used. Roads that have a high potential for debris, such as near farming operations, should not include porous asphalt because of the high potential for rapid clogging. Another location where porous asphalt should not be used is on low volume, low speed pavements. When traffic volumes or traffic speed is low, the self cleaning attributes of porous asphalt are negated. Self cleaning occurs because of the pumping and suction of the tires of numerous fast moving vehicles. The final location the authors recommended not using porous asphalt included areas subjected to very high shearing forces at the tire/pavement interface.

1.7 Lefebvre, G. "Porous Asphalt." Permanent International Association of Road Congresses. 1993.

1.7.1 General
This comprehensive document provides an overview of the state-of-art for the use of porous friction courses in Europe [as of 1993]. Complete chapters are dedicated to: 1) criteria for the selection of porous asphalt for a specific project; 2) desirable material properties; 3) mix design; 4) production and construction; 5) field performance; and 6) maintenance and repair. Experiences from many countries are combined to discuss each of the above topics.

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1.7.2 Benefits

Lefebvre mentions numerous benefits when using porous asphalt. These benefits were categorized based upon safety, driving comfort and environmental impact.

Benefits related to safety include hydroplaning, skid resistance, splash and spray, light reflection, driving speed, stability of construction and the effects on accidents. Hydroplaning occurs when a layer of water builds up between a tire and the pavement surface. This layer of water breaks the contact between the tire and the road. There are two aspects of porous asphalt that help prevent the occurrence of hydroplaning. First, because water drains into porous asphalt, the film of water is not available to break the bond between the tire and pavement surface. The second aspect is the macrotexture provided by porous asphalt. Even when clogged, porous asphalt provides a significant amount of macrotexture. This macrotexture provides small channels for water to be dissipated as a tire crosses over the pavement. Therefore, in wet driving conditions, the skid resistance of porous asphalt wearing layers is generally very good. Lefebvre does mention; however, that in dry driving conditions the skid resistance of porous asphalt pavements is generally about the same as traditional dense-graded wearing surfaces.

Another benefit derived from the use of porous asphalt is the reduction in splash and spray. Rolling wheels will throw water into the air from pools on the pavement surface (splash) as well as mist the surface water (spray). These water droplets reduce driver visibility. Water that comes from the roadway will also generally be contaminated with dirt and debris which can be smeared by windshield wiper blades, further reducing visibility. Through the infiltration of water into a porous asphalt layer, the pools of water will not be available to create splash and spray under rolling wheels. Lefebvre indicates that in many European countries, counteracting splash and spray was the primary reason for applying porous asphalt layers.

Another benefit related to safety is the light reflectivity of pavement markings. Drivers observe a pavement at a glancing angle of about 1 degree or less. When surfaces are very smooth, the reflection of light at this angle will look similar to a mirror in the distance. This is especially true when water is on a pavement. Porous asphalt will diffuse the reflection of light due to the high macrotexture even when observed from a glancing angle.

Lefebvre identifies increased driving speed during wet weather as a benefit related to safety. During rain events, the decreased potential for hydroplaning and splash/spray allows drivers increased confidence that results in increased speeds. This results in less traffic moving at slower speeds.

Environmental benefits listed in this document include noise reduction, pavement smoothness and use of waste rubber. Lefebvre indicates that the average noise level resulting from traffic on porous asphalt layers is about 3dB(A) lower than on dense-graded pavement layers. Another benefit is related to pavement smoothness. Lefebvre indicates that porous asphalt layers are typically constructed smoother than dense-graded

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layers. Under certain conditions, this has resulted in a 1 to 2 percent reduction in fuel consumption.

1.7.3 Materials and Mix Design

Lefebvre provided a review of general desirable characteristics of materials needed for porous asphalt. For coarse aggregates, particles should be crushed having the proper shape and texture. Coarse aggregates should be resistant to polishing. Coarse aggregates should also be hard and resistant to the effects of freeze/thaw cycles. Fine aggregates should also be the result of crushing.

Both neat and modified binders have been used in PFCs. If neat asphalt binders are utilized, stabilizing additives are needed. If modified binders are used, stabilizing fibers are used at times. Typical asphalt binder modifiers have included polymers and recycled materials. Both elastomers and plastomers have been used with success. Recycled rubber is the most common recycled material used.

The document provides general overviews/concepts of various mix design methodologies used in Europe. Spain developed a mix design method based upon the Cantabro Abrasion Test. The method entails specifying a minimum air void content of 21 percent and a maximum Cantabro Abrasion loss of 30 percent (tested at 18˚C). Figure 1 illustrates the concept. The minimum asphalt binder content is selected as the binder content that meets both the air void and Cantabro loss requirements. The document indicates that Belgium utilizes a similar concept. Another concept used to design porous asphalt in Europe involves a draindown test. The maximum asphalt binder content is selected based upon draindown testing.

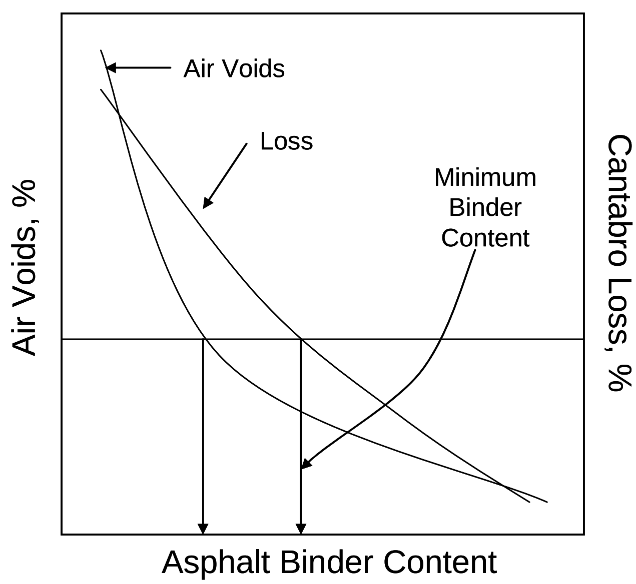

Figure 1: Use of Cantabro Abrasion and Air Voids to Select Minimum Binder Content

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Each of the benefits cited earlier are related to the drainage capacity of PFCs. PFCs are designed for high air void contents in order to enhance the ability of the layer to remove water from the pavement surface. However, only interconnected voids open from the pavement surface are of benefit. Figure 2 and Table 7 illustrate the types of air voids with PFC and their relative importance in effectively removing water and enhancing noise absorption.

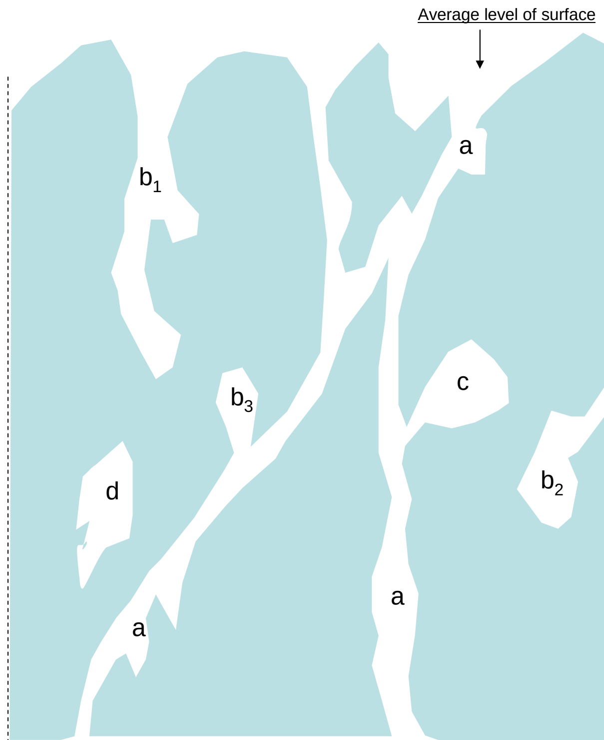

Figure 2: Air Void Classifications

Table 7: Influence of Air Void Classification on Permeability and Noise

Description of voids			Type	Effectiveness with regard to: water permeability	Effectiveness with regard to: noise absorption
"open" (accessible)	continuously connected		a	+	+
	not continuous	accessible from the surface	b1	-	+
		entering the section	b2	-	?
		"blind canal"	b3	-	?
"closed"	not accessible to water under atmospheric pressure		c	-	-
	fully closed		d	-	-

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1.7.4 Construction Practices

Construction practices are provided for production, transportation, laydown and specifications. During production, the plant operator must pay closer attention to mix temperature than for dense-graded mixes. Higher mix temperatures will increase the potential for draindown. When using modified asphalt binders, the supplier's recommendations for temperature should be used. If the binder modifier is to be formulated on-site, preliminary testing should be conducted to ensure the blended material meets requirements.

During transport, the potential for draindown increases as haul time increases. Also, vibrations transmitted to the porous asphalt through the truck bed can also increase the potential for draindown. Every effort should be made to maintain the temperature of the mix during transportation. This is especially true when modified asphalt binders are used.

Laydown of porous asphalt mixes is no more difficult than for dense-graded HMA. Handwork can be especially hard, however. Once placed, vibratory rollers should not be used to compact porous asphalt. The energy produced by vibratory compactors can fracture aggregates. The preferred method for compacting porous asphalt in Europe includes the use of a 10 to 12 ton steel-wheel roller making 2 to 3 passes. Pneumatic tire rollers are generally not used and prohibited in some countries.

When placing the porous asphalt layer, it must be daylighted at the edge. Figure 3 provides three examples of how pavement edges can be constructed. Obstructions at the pavement edge will prevent water from flowing through the porous asphalt layer. Also, longitudinal joints should not be sprayed with a tack coat. Again, this prevents water from flowing through the porous asphalt layer. Related to laydown, porous asphalt should never end in the middle of a vertical or horizontal curve. Porous asphalt layers should end in a flat region of the roadway.

With respect to construction specifications, porous asphalt should only be placed on impervious layers. In order to promote drainage of water, the underlying layer should have a sufficient cross-slope and daylighted on the pavement edge. A tapered edge can be used if the PFC is not carried out to the pavement edge (Figure 3). Pavement marking materials should be such that they don't penetrate into the pavement layer.

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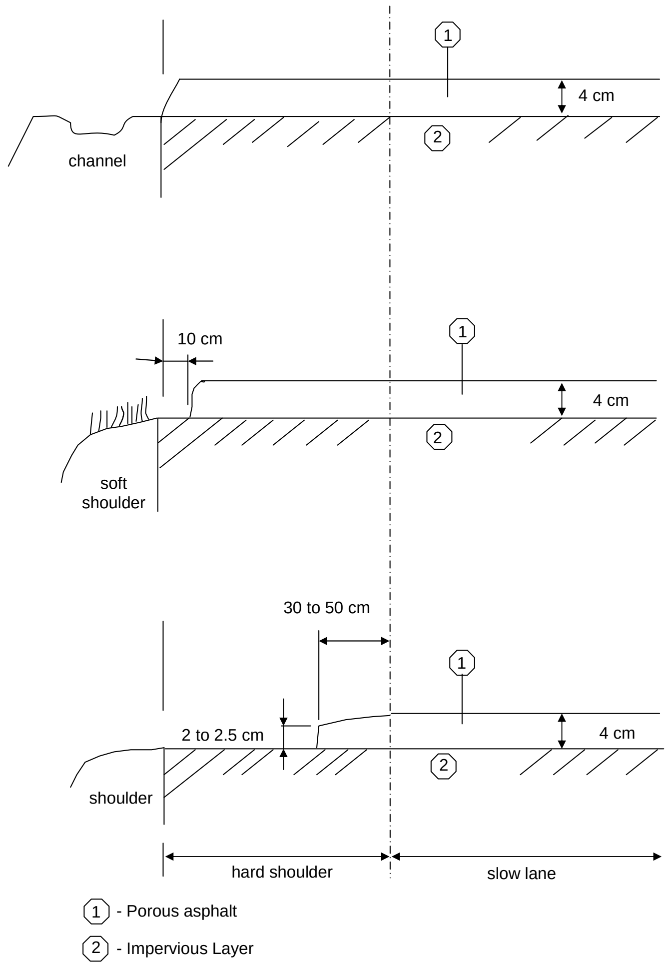

Figure 3: Examples of Daylighting Porous Asphalt Mixtures

1.7.5 Maintenance Practices

General maintenance of porous asphalt layers has many similarities to dense-graded pavement layers in that skid resistance, smoothness, cracking, rutting and/or deterioration must all be monitored. However, when dealing with porous asphalt, permeability and noise reducing capabilities must also be considered. During the life of a porous asphalt layer, dirt and debris can clog the layer. This clogging decreases the ability to drain water through the layer. Common methods for de-clogging porous asphalt layers include the use of highly pressurized water or the use of a "suck-sweep" cleaning truck.

The role of winter maintenance is to clear roads from ice and snow at an acceptable cost, so that drivers can use the major roads almost normally under all but exceptional winter conditions. Lefebvre indicated that each country appears to handle winter maintenance in a different manner.

The effects of three winter conditions on a porous asphalt layer were discussed. The first is a freezing fog/hoar frost. This condition occurs at certain temperature-humidity combinations and results in a very thin layer of ice on the pavement surface due to condensation and near freezing temperatures. Both France and the Netherlands have

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shown that porous asphalt layers are generally 1 to 2˚ C lower than dense-graded layers. Research in Austria indicated that porous asphalt behaves differently at temperatures in the range of 0 to -5˚C. Below this temperature range, the porous asphalt acts similar to dense-graded layers. At these lower temperatures, preventative salt spreading is not as effective. An increased frequency is often needed.

The second winter condition cited in the document was frozen wet surfaces. This condition is ice building up on the pavement surface due to rain on the frozen pavement surface. In this condition, preventative salting is not very effective. Also, during the precipitation, an increased frequency in salting is required.

The final winter condition is snow fall or hail. Preventative salting in this situation is also less effective. More frequent salting is needed to melt the snow; however, too much salt can lead to formation of ice within the porous asphalt layer due to the melted snow re-freezing within the layer.

Lefebvre provided typical chemicals used as deicing salts in Europe. Table 8 presents typical materials used in different countries and Tables 9 through 11 provide typical dosage rates.

1.7.6 Rehabilitation Practices

Lefebvre indicates that a distinction needs to be made between minor and major rehabilitation of porous asphalt layers. Minor rehabilitation entails small local repairs necessary because of small damage or distress with the rest of the pavement layer in good condition. Major rehabilitation is conducted when the entire layer is in need of repair.

Local distresses can be repaired by replacing the area with dense-graded mix. A discontinuity in the drainage path will occur; however, the discontinuity should not be considered severe when dealing with small localized areas.

Major rehabilitation techniques include replacement of the entire layer or refurbishment of the entire layer. Replacement of the porous asphalt would include completely removing the layer and replacing with a new layer. Refurbishment of the layer would include in-situ recycling. It was noted that the Netherlands have been using hot in-place recycling to rehabilitate porous asphalt. Largely, this experiment has been unsuccessful.

1.7.7 Performance

Several performance measures were discussed. Lefebvre indicated that rutting has not been experienced within porous asphalt layers in Europe. This is likely because of the relatively thin lifts and the stone-on-stone contact in the aggregate structure of the mix.

Porosity is another important performance characteristic. Porosity can be measured using either air voids or in-place permeability. Porosity should be evaluated at the time of construction to ensure the porous asphalt is properly constructed and during the life of the pavement to evaluate the level of clogging. It was pointed out that permeability will vary across the lane transversely. This variation is mainly due to the pressure-suction action

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of tires over the pavement during a rain. Loss of permeability is related to the amount of traffic. More heavily trafficked pavements will maintain permeability longer. In this respect, design lanes are generally more permeable than passing lanes.

Raveling is another performance characteristic of porous asphalt. In the Netherlands, they expect raveling to begin in 10 years compared to 12 years for dense-graded layers. When raveling occurs, the acoustical benefits of porous asphalt are diminished.

As stated previously, porous asphalt layers generally provide good wet weather friction.

However, skid resistance is generally low just after construction because of the asphalt binder film coating the aggregates at the pavement surface. Once the asphalt binder film has worn away, skid resistance will improve.

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Table 8: Forms in Which Salts Are Used in Europe

	Austria	Belgium	Denmark	France	Germany	Italy	Japan	Netherlands	Sweden	Switzerland	United Kingdom
Solid NaCl	VC	VC	VC	VC	VC	VC	VC	VC	VC	VC	VC
NaCl brine	RE			RE		RE	VC		RE		
$CaCl_2$ flakes		VC		RE	RE	VC	VC			LC	
$CaCl_2$ brine		RE				VC	VC				
Solid mixture NaCl/$CaCl_2$	LC			LC		RE				VC	
Wet salt method NaCl + $CaCl_2$ solution	VC	RE		LC	VC	RE		VC		LC	RE
Wet salt method NaCl + NaCl solution	LC		VC					LC	VC	RE	RE

VC = very commonly used
LC = less commonly used
RE = rather exceptionally used
Blank = never used

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Table 9: Average Spreading Rates For Solid NaCl (g/m^2)

Country	Normal Treatment	Preventive Treatment
Germany	10-20	
	20-30	-
Belgium	20-30	7-20
Denmark	>10	5-10
France	20-30	10-15
Italy	15-30	10-15
Japan	<100	>10
Netherlands	5-20	-
United Kingdom	20-40	10-20
Sweden	20	5-10
Switzerland	15-20	10-15

Table 10: Average Spreading Rates for $CaCl_2$ Flakes (g/m^2)

County	Normal Treatment	Preventive Treatment
Belgium	20-30	7-20
France	20-30	-
Italy	10-20	5-10
Japan	10-50	10-50
Switzerland	15-40	15-30

Table 11: Average Spreading Rates for Wet Salt (g/m^2)

County	Normal Treatment	Preventive Treatment
Germany	10-30	10-15
Austria	-	10
Belgium	20-30	7-20
Denmark	>10	5-10
France	-	10-15
Netherlands	5-20	5-7
Sweden	15	5
Switzerland	-	5-15

1.7.8 Structural Design

No specifics were given on inclusion within structural design.

1.7.9 Limitations

No specific limitations were provided; however, three disadvantages were given. First, porous asphalt generally costs more than dense-graded layers. This is a result of requiring high quality, polish resistant aggregates and polymer modified asphalt binders. Also, pavement markings have to be adapted for porous asphalt. Special impervious layers specifically placed below porous asphalt also increase construction costs. Another disadvantage of using porous asphalt is the relatively shorter economic life. Finally, maintenance is generally more expensive, especially winter maintenance.

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1.8 Alderson, A., "The Design of Open Graded Asphalt." Australian Asphalt Pavement Association. CR C5151. November 1996.

1.8.1 General

Alderson provides a comprehensive report on the design and construction practices for using open graded asphalt mixes in Australia. Starting with a review of expected benefits of open graded asphalt mixes, Alderson indicates which properties are critical for achieving desired performance and then describes the mix design and construction practices required for obtaining those properties. The author provides test data from different types of pavement which show the noise reduction benefit of open-graded mixes.

Regarding mix design, Alderson mentions that the design asphalt binder content is determined on the basis of air voids, abrasion loss and draindown criteria, and provide some recommended values. The important properties are sufficient binder content and proper type of binder for resisting abrasion loss, and binder draindown. The air voids criterion is important for achieving the noise reduction and water removal properties. The use of polymer modification and fiber for enhancement of binder properties and content has been mentioned.

Alderson mentions that for production and construction, a lower temperature is necessary to avoid draindown, and the use of static steel-wheel drum rollers with a few passes is necessary for proper compaction. Alderson recommends using layer thicknesses of more than 2.5 times and less than 4 times the nominal aggregate size. Alderson cautions that to avoid damage in underlying layers, open-graded mixes must be placed only on impervious mixes, such as those with less than 5 percent air voids.

1.8.2 Benefits of Permeable Asphalt Mixtures

Alderson lists the following benefits for open-graded asphalt mix: 1) Provide better wet weather skid resistance, 2) Reduce both external and within vehicle noise, 3) Reduce splash and spray, 4) Improve visibility of road surface markings, specifically in wet weather; and 5) Provide a smooth riding surface.

1.8.3 Materials and Mix Design

Alderson provides information on the two types of open-graded asphalts used in Australia, and their applicability. These are summarized in Table 12.

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Table 12: Open-Graded Asphalt Used In Australia

Grade	Description	Pavement traffic volume	
		Commercial vehicles/lane/day	Equivalent standard axles
Type I	For modest level of performance at minimum cost; higher binder content or modified binders not used	< 500	$< 5 * 10^6$
Type II	Best performance; High binder contents, polymer modified binders and fibers used; (*mix designs described in Alderson's report are mostly applicable to this type.)*	> 500	$> 5 * 10^6$

Alderson then provides information on nominal aggregate gradation and mix design, which are summarized in Table 13.

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Table 13: Mix Design of Open-Graded Asphalts in Australia

<table>
<tr><th>Parameter</th><th>Selection</th><th>Recommendation</th></tr>
<tr><td colspan="3">Aggregate</td></tr>
<tr><td>Nominal size</td><td>Based on performance and layer thickness; available in 10, 14 and 20 mm nominal sizes, with 10 mm being most common</td><td>For proper compaction and to avoid instability, should be placed in layer thickness at least 2.5 times and maximum 4 times the nominal size of the mix economic to consider the largest nominal size consistent with performance and layer thickness; design thickness can be considered on the basis of accommodating run-off from periodic high intensity rain storms.</td></tr>
<tr><td>Quality/ Properties</td><td>Aggregates meeting standards for wearing courses are applicable.</td><td>Require high quality aggregates – strong, cubical, good microtexture, affinity for asphalt, clean and with crushed faces</td></tr>
<tr><td>Mineral fillers</td><td>Durability problems can be avoided by preventing loss of adhesion of asphalt binder to aggregates; hydrated lime improves binder-aggregate adhesion and reduces potential of stripping; also reduces draindown</td><td>Hydrated lime is preferred; Portland cement and ground limestone can be used.</td></tr>
<tr><td>Gradation</td><td>Selected as a balance between highly permeable (highly gap graded) <u>and</u> a strong aggregate skeleton (less gap graded); minimum air void content should be 20 %</td><td>
<table>
<tr><th rowspan="2">Sieve size, mm</th><th colspan="4">Nominal mix size</th></tr>
<tr><th>10 mm</th><th>14 mm</th><th>20 mm</th><th>Production tolerance</th></tr>
<tr><td>26.5</td><td></td><td></td><td>100</td><td>Nil</td></tr>
<tr><td>19.0</td><td></td><td>100</td><td>95</td><td>±6</td></tr>
<tr><td>13.2</td><td>100</td><td>95</td><td>55</td><td>±6</td></tr>
<tr><td>9.5</td><td>83</td><td>50</td><td>30</td><td>±6</td></tr>
<tr><td>6.7</td><td>18</td><td>27</td><td>20</td><td>±6</td></tr>
<tr><td>4.75</td><td>14</td><td>11</td><td>10</td><td>±5</td></tr>
<tr><td>2.36</td><td>6</td><td>9</td><td>8</td><td>±5</td></tr>
<tr><td>1.18</td><td>4</td><td>8</td><td>6</td><td>±5</td></tr>
<tr><td>0.6</td><td>4</td><td>6.5</td><td>4</td><td>±5</td></tr>
<tr><td>0.3</td><td>3</td><td>5.5</td><td>3</td><td>±3</td></tr>
<tr><td>0.15</td><td>3</td><td>4.5</td><td>3</td><td>±3</td></tr>
<tr><td>0.075</td><td>2</td><td>3.2</td><td>2</td><td>±1</td></tr>
</table>
</td></tr>
<tr><td colspan="3">Asphalt Binder</td></tr>
<tr><td>Binder type, modifier and fiber</td><td>Binder film should provide sufficient cohesion to resist traffic loads; binder should be resistant to oxidation or present in sufficient thickness to prevent oxidative hardening; fibers reduce draindown</td><td>Modified asphalt binders for improved cohesion and durability, and possible reduction in draindown; Polymers include SBS, SBR, EVA and crumb rubber modifier (CRM); fibers used in 0.3 to 0.5 percent by mass of total mix.</td></tr>
<tr><td>Binder content</td><td>Effective binder content (with consideration of absorption) should be such that criteria for recommended air voids, maximum permissible abrasion loss and draindown are met</td><td>Use step by step procedure to select gradation, based on recommended asphalt content; change gradation and/or asphalt content to provide optimum air voids; check abrasion loss and binder draindown; check with recommended asphalt binder contents.</td></tr>
</table>

Alderson provides a description of the steps necessary for conducting mix design. These steps are summarized in this review in Figure 4.

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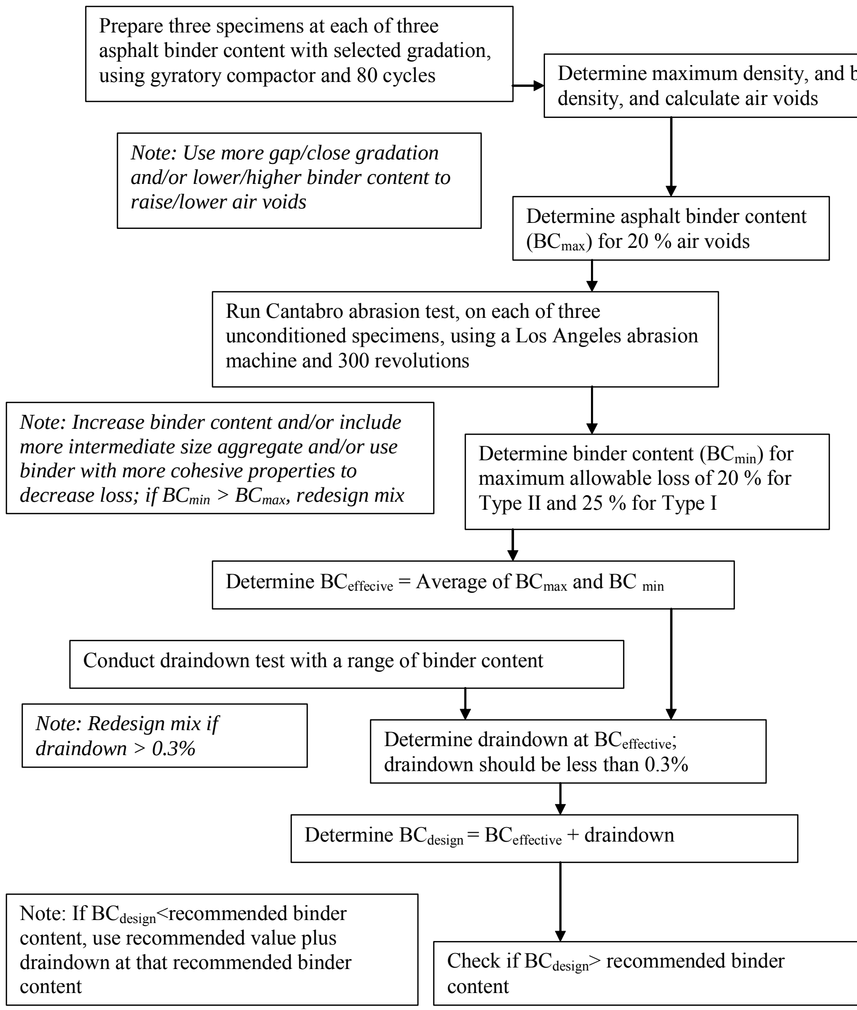

Note: All binder contents are effective binder contents, determined on the basis of absorption of aggregates

Figure 4: Steps in Mix Design

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1.8.4 Construction Practices

Alderson provides a set of discussions and recommendations for production and laydown of open-graded asphalt mixes. These discussions and recommendations are summarized in Table 14.

Table 14: Production and Laydown of Open-Graded Asphalt Mixes

Property	Open-graded asphalt mix	Factors/Recommendations
Production	Risk of draindown during production and transportation	Lower mixing temperature by 10 to 20^{o}C; follow mix design guidelines.
Construction	Equipment similar to those use for dense graded asphalt; thin layers and high air void content; joints should allow drainage	Rollers should be close behind pavers to compact mix while it is hot; use limited passes with static wheel rollers to avoid crushing or over-compaction; avoid vibratory roller to avoid forcing of binder to surface (and closing voids), and crushing of aggregates, and avoid rubber tired rollers to prevent sticking of materials; tack coating of cold longitudinal joints should be avoided to prevent blocking of drainage; recommend using hot joints.

Alderson mentions that to avoid damage to underlying layers, open-graded mixes should be placed on impervious layers only. He indicates that pavements can be considered to be impervious if they have less than 5 percent air voids and have been trafficked for more than a year. If such conditions are not available before the placement of an open-graded mix, Alderson recommends the use of a very heavy tack coat, a fog coat or a 7 mm seal.

1.8.5 Maintenance Practices

No information has been provided on maintenance practices.

1.8.6 Rehabilitation Practices

No information has been provided on rehabilitation practices

1.8.7 Performance

Alderson discusses each performance related property along with specific factors affecting this property. His discussions are summarized in Table 15.

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Table 15: Performance Related Properties

Property	Open graded asphalt	Factors
Skid resistance	Better than dense-graded asphalt; potential of hydroplaning is reduced; improved visibility	Initially has lower skid resistance due to thicker asphalt binder film; wearing by traffic exposes the aggregates within a time period depending on traffic and environment; precaution needed when placed on low volume high speed roads in cold weather; skid resistance can be measured by the British Pendulum tester
Design life	Limited life, 8-10 years in terms of structural life, but adequate in terms of retention in surface characteristics	Porous structure exposes binder films between aggregates to actions of UV, moisture and oxidation, leading to raveling and failure; important to have binders with adequate properties and binder films of adequate thickness to prevent failures.
Water dispersion	Significantly better than dense-graded mixes; porosity can get reduced with time due to clogging; oil spills or leakage can lead to destruction of cohesive bonds between aggregates and asphalt binder and mud from tires can clog up pores	Important to drain out water that percolates through the surface; underlying surface should be impervious, there should be adequate cross slope and side drains; high speed traffic can help in cleaning the pores; not recommended for intersections to avoid oil spills and also not recommended near quarries or farms to prevent clogging by dust or mud.
Noise reduction	Reduction of 3dBA is generally made possible by the use of open-graded asphalt; reduction is proportional to air void content and layer thickness; reduction caused by expulsion of air between tire and surface into pores and surface texture; study reports noise levels for two open-graded asphalts as 79.3 and 77.5 dBA, compared to 87.2 dBA for deep grooved Portland cement concrete and 83.4 dBA for dense-graded asphalt.	Optimum thickness for noise reduction is 40 mm; noise reduction due to surface texture is reduced with wearing of surface.
Strength	Structural strength not considered during design of pavement; about half to two thirds of strength of dense- graded asphalt; current practice is to consider modulus of 800 to 1200 MPa.	Tests in laboratory should be conducted under confined conditions; wheel tracking tests are suitable for evaluation of rutting potential; fatigue testing can be done with four point bending beam test at low temperatures.

1.8.8 Structural Design

No information has been provided on structural design.

1.8.9 Limitations

No information has been provided on limitations.

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1.9 **Ketcham, S.A., L.D. Minsk, R.B. Blackburn, and E.J. Fleege. "Manual of Practice for an Effective Anti-icing Program. A Guide for Highway Winter Maintenance Personnel." FHWA-RD-95-202. U.S. Department of Transportation. Federal Highway Administration. June 1996.**

1.9.1 General

This manual of practice provides information for the successful implementation of an effective highway anti-icing program. As this infers, the manual of practice is for all highway surfaces and does not specifically deal with anti-icing operations for permeable friction courses. Prior to discussing the specifics of the manual, a number of terms related to winter maintenance are provided.

<u>Black Ice</u> Term for a very thin coating of clear bubble-free, homogeneous ice which forms on a pavement surface having a temperature at or slightly above 0˚C (32˚F) and when the temperature of the air in contact with the ground is below the freezing point of water. This causes small slightly supercooled water droplets to deposit on the pavement surface and flow together before freezing.

<u>Dry Chemical Spread Rate</u> This term refers to the anti-icing chemical application rate for solid chemical applications with the spread rate being the mass of the chemical applied per lane kilometer (or mile). For liquid chemical applications, the spread rate is the mass of dry chemical in solution applied per lane kilometer (or mile).

<u>Freezing Rain</u> Supercooled droplets of liquid precipitation that falls onto a pavement surface whose temperature is below or slightly above 0˚C (32˚F). These droplets result in a hard, slick, generally thick coating of ice. This formation of ice is generally called glaze or clear ice.

<u>Frost</u> Dew or water vapor cause the formation of ice crystals in the form of scales, needles, feathers or fans on surfaces when the temperature of the pavement surface is at or below freezing. This is also called hoarfrost.

<u>Light Snow</u> Snow falling at a rate less than 12.5mm (1/2 in.) per hour. Visibility is generally not reduced.

<u>Liquid Chemical</u> A chemical solution in the form of a liquid.

<u>Moderate or Heavy Snow</u> Snow falling at a rate greater than 12.5mm (1/2 in.) per hour. Visibility may be adversely affected.

<u>Sleet</u> A mixture of rain and/or snow that has been partially melted due to falling through an atmosphere that is slightly above freezing.

<u>Slush</u> Accumulation of snow that lies on an impervious surface and is saturated with water in excess of its drainage capacity. Slush will not support any weight but will "squish" until the base support is reached.

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Anti-icing defined is the preventing of formation or development of bonded snow and ice to a pavement surface. This is accomplished by applying chemical freeze-point depressant chemical to the pavement surface at the proper concentration and at the proper time. Anti-icing is different than deicing in that anti-icing is designed to prevent the buildup of snow and ice, while deicing is the process of weakening or destroying the bond between snow/ice and the pavement.

1.9.2 Benefits of Permeable Asphalt Mixes

No specific benefits were given.

1.9.3 Materials and Mix Design

No specifics on materials and mix design were given.

1.9.4 Construction Practices

No specific construction practices were given.

1.9.5 Maintenance Practices

There are two distinct winter maintenance strategies that make use of freezing-point depressants: anti-icing and deicing. Even though both of these strategies use chemicals to maintain roadways during winter events, the fundamental objective of each is different. Anti-icing operations are conducted to prevent the formation of development of bonded snow and ice to the pavement surface. Deicing operations are performed to break the bond of already bonded snow and ice. Therefore, anti-icing operations are preventative and deicing operations are reactive.

A winter maintenance program is a very complex endeavor. One primary element of a successful program is to select the level of service (LOS), or road conditions, for the pavement during the winter event. The LOS will likely depend upon many factors, but primarily the importance of the road and the amount of traffic. Figure 5 illustrates the components of an anti-icing program based upon the LOS. As illustrated in Figure 5, an effective anti-icing program includes two components to support the anti-icing program and operations.

Tools to support an effective anti-icing program include three tool boxes: operations, decision making and personnel. The operations toolbox includes the materials and techniques that are available for an anti-icing program. The materials include solid chemicals, chemical solutions, pre-wetted chemicals and plowing. Dry solid chemicals can be an effective anti-icing treatment in many instances; however, some form of moisture must be available for two reasons: prevent the loss of the dry chemicals from a dry pavement surface and to trigger the solution of the chemical. For this reason, dry solid chemicals are generally applied after sufficient precipitation has fallen, but before bonding between the pavement and snow occurs. The most common form of dry solid chemical is sodium chloride.

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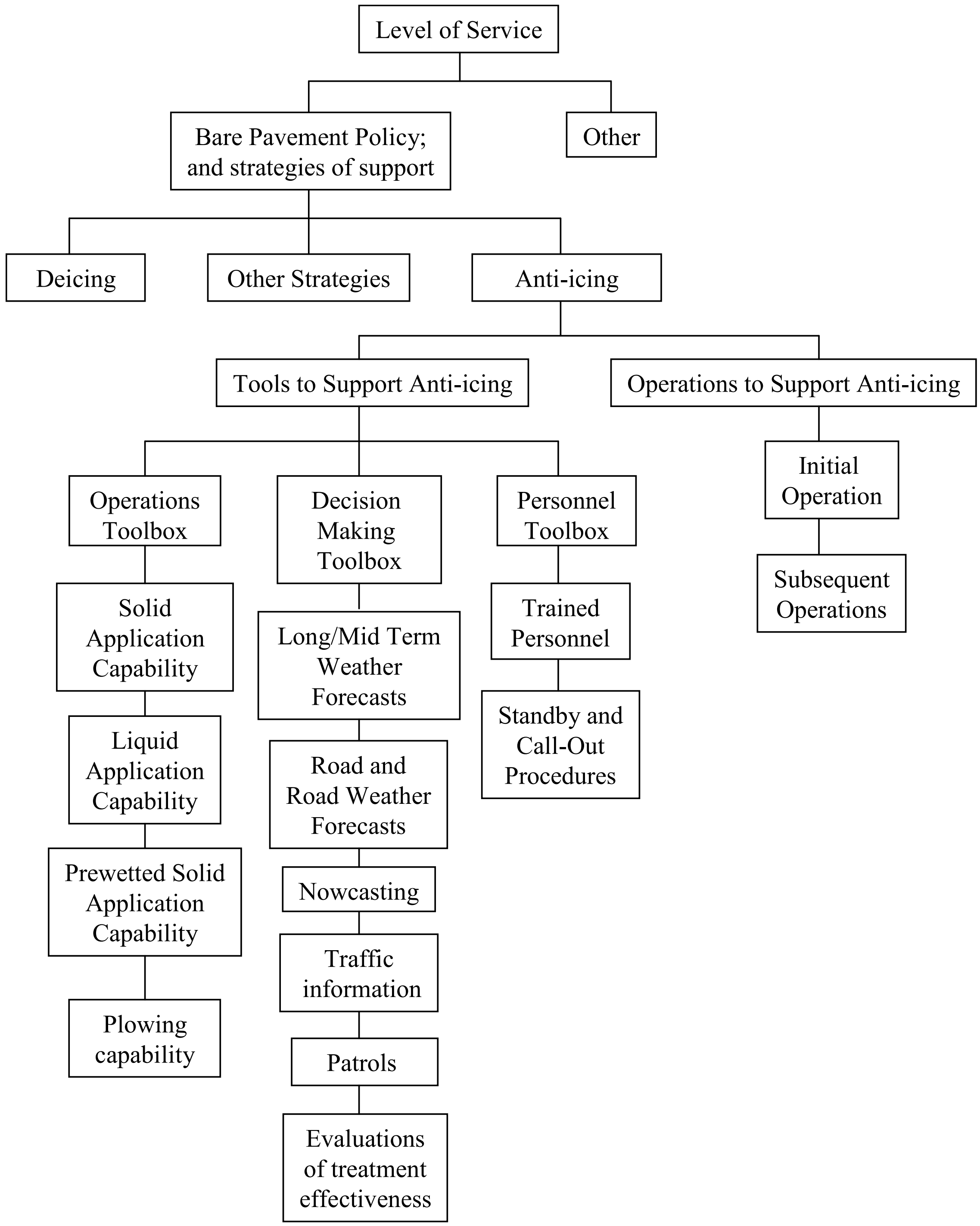

Figure 5: Outline of the Components of an Anti-icing Program in the Context of a Winter Maintenance Program

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Liquid (solids in solution) anti-icing measures can be advantageous in small amounts for some conditions at pavement temperatures of about -5˚C (23˚F) and above. Liquid applications can be uniformly and quickly applied to a dry pavement as a pre-storm treatment. Liquid applications can be used at temperatures below -5˚C (23˚F) by increasing the application rate. There are five chemicals that are generally used for liquid anti-icing treatments: sodium chloride, magnesium chloride, calcium chloride, calcium magnesium acetate and potassium acetate.

Presetting solid chemical treatments can improve the effectiveness of the treatment. Prewetting also accelerates the process of the solid chemical going into solution. Other benefits of prewetting include a more uniform application rate because particles stick to the pavement surface, spreading speed can be increased, and in some instances, the road surface dries more quickly. Sodium chloride has been the most common solid chemical when applying prewetted chemicals. Finer graded (smaller particle sizes) solid chemicals are more appropriate for the prewetting process. Because of the smaller particle sizes and, therefore, larger surface area, the time for solution to form and cover the pavement surface will decrease.

The final tool in the Operations tool box is plowing. The primary purpose of plowing within an anti-icing program is to remove as much snow or loose ice prior to applying the chemical treatment.

As shown in Figure 5, the next tool box within the tools to support an anti-icing program is decision making. The decision making toolbox includes weather forecast information, road and road weather information, nowcasting, traffic information, patrols and evaluations of treatment effectiveness. Accurate weather forecast information is essential to deciding whether or not to initiate a treatment, when to start and what treatment to apply. Weather forecasts would include when precipitation is expected to start, what form the precipitation will be, the probable air temperatures and the temperature trend during and after the storm, and the wind direction and speed.

Road and road weather information includes real-time knowledge of the state of the pavement surface in order to make informed decisions on the appropriate anti-icing treatment. Information required would include pavement temperature, whether the pavement is wet or dry and some indication of the concentration of a treatment. The most important of these is pavement temperature. The solubility of all chemicals varies with temperature, the lower the temperature the less solubility. An effective method of monitoring pavement temperature is to place pavement sensors. Pavement temperature sensors can be used to generate a forecast of pavement temperature. Some pavement sensors can also provide a relative indication of how much chemical concentration is left on the pavement from the previous anti-icing treatment. A road weather information system (RWIS) is also an effective method of providing real-time information on weather conditions. A RWIS is a network of roadside weather data-gathering and road condition monitoring systems that can be remotely accessed and analyzed.

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The third component of the decision making toolbox is nowcasting. Nowcasting refers to the use of real-time data for short-term forecasting. Nowcasting can use RWIS stations, radar, patrols and/or any other data sources to make an educated judgment on the probable condition of a roadway over the short-term.

Another component of the decision making toolbox is traffic information. Vehicles can affect the condition of the pavement surface through tires compacting, abrading, displacing or dispersing snow and heat created by tire, engines and exhaust systems. Vehicles can also disperse dry solid chemicals in the absence of any moisture on the pavement.

Patrols are also an important component in the decision making toolbox. Patrols provide visual observations of weather conditions and pavement conditions. Trained personnel prepared to judge the severity of conditions and to make recommendations or corrective actions are invaluable.

The final component of the decision making toolbox is the evaluation of treatment effectiveness during an event. Observations from patrols will provide real-time estimates of the effectiveness of treatments. Pavement sensors that provide an indication of chemical concentration can also provide an indication of treatment effectiveness. Specialized vehicles can also measure pavement friction during storm events to provide an evaluation of treatment effectiveness.

The final toolbox needed for an effective anti-icing program is personnel. The personnel toolbox includes trained personnel as well as effective standby and call out procedures.

Operations to support an anti-icing program include initial operations and subsequent operations. The initial anti-icing operations includes assembling the necessary data to make an educated decision on the type treatment required, making the decision and then acting on the decision. Subsequent operations will require monitoring weather and pavement conditions, determining if or when additional treatments are required and acting on the required subsequent action.

Ketcham et al also provide guidance for anti-icing operations for various storm events. Tables 16 through 21 provide these recommendations.

1.9.6 Rehabilitation Practices
No specifics on rehabilitation practices were given.

1.9.7 Performance
No specific performance measures were given.

1.9.8 Structural Design
No specifics on inclusion within structural design were given.

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1.9.9 Limitation

No specific limitations were given.

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Table 16: Weather Event: Light Snow Storm

PAVEMENT TEMPERATURE RANGE, AND TREND	INITIAL OPERATION				SUBSEQUENT OPERATIONS			COMMENTS
	pavement surface at time of initial operation	maintenance action	dry chemical spread rate, kg/lane-km (lb/lane-mi)		maintenance action	dry chemical spread rate, kg/lane-km (lb/lane-mi)		
			liquid	solid or prewetted solid		liquid	solid or prewetted solid	
Above 0°C (32°F), steady or rising	Dry, wet, slush, or light snow cover	None, see comments			None, see comments			1) Monitor pavement temperature closely for drops toward 0°C (32°F) and below 2) Treat icy patches if needed with chemical at 28 kg/lane-km (100 lb/lane-mi); plow if needed
Above 0°C (32°F), 0°C (32°F) or below is imminent;	Dry	Apply liquid or prewetted solid chemical	28 (100)	28 (100)	Plow as needed; reapply liquid or solid chemical when needed	28 (100)	28 (100)	1) Applications will need to be more frequent at lower temperatures and higher snowfall rates 2) It is not advisable to apply a liquid chemical at the indicated spread rate when the pavement temperature drops below -5°C (23°F) 3) Do not apply liquid chemical onto heavy snow accumulation or packed snow
ALSO **-7 to 0°C (20 to 32°F)**, remaining in range	Wet, slush, or light snow cover	Apply liquid or solid chemical	28 (100)	28 (100)				
-10 to -7°C (15 to 20°F), remaining in range	Dry, wet, slush, or light snow cover	Apply prewetted solid chemical		55 (200)	Plow as needed; reapply prewetted solid chemical when needed		55 (200)	If sufficient moisture is present, solid chemical without prewetting can be applied
Below -10°C (15°F), steady or falling	Dry or light snow cover	Plow as needed			Plow as needed			1) It is not recommended that chemicals be applied in this temperature range 2) Abrasives can be applied to enhance traction

Notes

CHEMICAL APPLICATIONS. (1) Time initial and subsequent chemical applications to *prevent* deteriorating conditions or development of packed and bonded snow. (2) Apply chemical ahead of traffic rush periods occurring during storm.

PLOWING. If needed, *plow before chemical applications* so that excess snow, slush, or ice is removed and pavement is wet, slushy, or lightly snow covered when treated.

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Table 17: Weather Event: Light Snow with Period(s) of Moderate or Heavy Snow

PAVEMENT TEMPERATURE RANGE, AND TREND	INITIAL OPERATION				SUBSEQUENT OPERATIONS					COMMENTS
	pavement surface at time of initial operation	maintenance action	dry chemical spread rate, kg/lane-km (lb/lane-mi)		maintenance action	dry chemical spread rate, kg/lane-km (lb/lane-mi)				
			liquid	solid or prewetted solid		liquid		solid or prewetted solid		
						light snow	heavier snow	light snow	heavier snow	
Above 0°C (32°F), steady or rising	Dry, wet, slush, or light snow cover	None, see comments			None, see comments					1) Monitor pavement temperature closely for drops toward 0°C (32°F) and below 2) Treat icy patches if needed with chemical at 28 kg/lane-km (100 lb/lane-mi); plow if needed
Above 0°C (32°F), 0°C (32°F) or below is imminent; *ALSO* **-4 to 0°C (25 to 32°F)**, remaining in range	Dry	Apply liquid or prewetted solid chemical	28 (100)	28 (100)	Plow as needed; reapply liquid or solid chemical when needed	28 (100)	55 (200)	28 (100)	55 (200)	1) Applications will need to be more frequent at lower temperatures and higher snowfall rates 2) Do not apply liquid chemical onto heavy snow accumulation or packed snow 3) After heavier snow periods and during light snow fall, reduce chemical rate to 28 kg/lane-km (100 lb/lane-mi); continue to plow and apply chemicals as needed
	Wet, slush, or light snow cover	Apply liquid or solid chemical	28 (100)	28 (100)						
-10 to -4°C (15 to 25°F), remaining in range	Dry, wet, slush, or light snow cover	Apply prewetted solid chemical		55 (200)	Plow as needed; reapply prewetted solid chemical when needed			55 (200)	70 (250)	1) If sufficient moisture is present, solid chemical without prewetting can be applied 2) Reduce chemical rate to 55 kg/lane-km (200 lb/lane-mi) after heavier snow periods and during light snow fall; continue to plow and apply chemicals as needed
Below -10°C (15°F), steady or falling	Dry or light snow cover	Plow as needed			Plow as needed					1) It is not recommended that chemicals be applied in this temperature range 2) Abrasives can be applied to enhance traction

Notes

CHEMICAL APPLICATIONS. (1) Time initial and subsequent chemical applications to *prevent* deteriorating conditions or development of packed and bonded snow. (2) *Anticipate increases in snowfall intensity. Apply higher rate treatments prior to or at the beginning of heavier snowfall periods to prevent development of packed and bonded snow*. (3) Apply chemical ahead of traffic rush periods occurring during storm.

PLOWING. If needed, *plow before chemical applications* so that excess snow, slush, or ice is removed and pavement is wet, slushy, or lightly snow covered when treated.

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Table 18: Weather Event: Moderate or Heavy Snow Storm

PAVEMENT TEMPERATURE RANGE, AND TREND	INITIAL OPERATION				SUBSEQUENT OPERATIONS			COMMENTS
	pavement surface at time of initial operation	maintenance action	dry chemical spread rate, kg/lane-km (lb/lane-mi)		maintenance action	dry chemical spread rate, kg/lane-km (lb/lane-mi)		
			liquid	solid or pre-wetted solid		liquid	solid or pre-wetted solid	
Above 0°C (32°F), steady or rising	Dry, wet, slush, or light snow cover	None, see comments			None, see comments			1) Monitor pavement temperature closely for drops toward 0°C (32°F) and below 2) Treat icy patches if needed with chemical at 28 kg/lane-km (100 lb/lane-mi); plow if needed
Above 0°C (32°F), 0°C (32°F) or below is imminent; *ALSO* **-1 to 0°C (30 to 32°F)**, remaining in range	Dry	Apply liquid or pre-wetted solid chem..	28 (100)	28 (100)	Plow accumulation and reapply liquid or solid chemical as needed	28 (100)	28 (100)	1) If the desired plowing/treatment frequency cannot be maintained, the spread rate can be increased to 55 kg/lane-km (200 lb/lane-mi) to accommodate longer operational cycles 2) Do not apply liquid chemical onto heavy snow accumulation or packed snow
	Wet, slush, or light snow	Apply liquid or solid chemical	28 (100)	28 (100)				
-4 to -1°C (25 to 30°F), remaining in range	Dry	Apply liquid or pre-wetted solid chemical	55 (200)	42-55 (150-200)	Plow accumulation and reapply liquid or solid chemical as needed	55 (200)	55 (200)	1) If the desired plowing/treatment frequency cannot be maintained, the spread rate can be increased to 110 kg/lane-km (400 lb/lane-mi) to accommodate longer operational cycles 2) Do not apply liquid chemical onto heavy snow accumulation or packed snow
	Wet, slush, or light snow	Apply liquid or solid chemical	55 (200)	42-55 (150-200)				
-10 to -4°C (15 to 25°F), remaining in range	Dry, wet, slush, or light snow cover	Apply prewetted solid chemical		55 (200)	Plow accumulation and reapply prewetted solid chem.. as needed		70 (250)	1) If the desired plowing/treatment frequency cannot be maintained, the spread rate can be increased to 140 kg/lane-km (500 lb/lane-mi) to accommodate longer operational cycles 2) If sufficient moisture is present, solid chemical without prewetting can be applied
Below -10°C (15°F), steady or falling	Dry or light snow cover	Plow as needed			Plow accumulation as needed			1) It is not recommended that chemicals be applied in this temperature range 2) Abrasives can be applied to enhance traction

Notes

CHEMICAL APPLICATIONS. (1) Time initial and subsequent chemical applications to *prevent* deteriorating conditions or development of packed and bonded snow -- *timing and frequency of subsequent applications will be determined primarily by plowing requirements*. (2) Apply chemical ahead of traffic rush periods occurring during storm.

PLOWING. *Plow before chemical applications* so that excess snow, slush, or ice is removed and pavement is wet, slushy, or lightly snow covered when treated.

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Table 19: Weather Event: Frost or Black Ice

PAVEMENT TEMPERATURE RANGE, TREND, AND RELATION TO DEW POINT	TRAFFIC CONDITION	INITIAL OPERATION			SUBSEQUENT OPERATIONS			COMMENTS
		maintenance action	dry chemical spread rate, kg/lane-km (lb/lane-mi)		maintenance action	dry chemical spread rate, kg/lane-km (lb/lane-mi)		
			liquid	solid or prewetted solid		liquid	solid or prewetted solid	
Above 0°C (32°F), steady or rising	Any level	None, see comments			None, see comments			Monitor pavement temperature closely; begin treatment if temperature starts to fall to 0°C (32°F) or below and is at or below dew point
-2 to 2°C (28 to 35°F), remaining in range or falling to 0°C (32°F) or below, *and* equal to or below dew point	Traffic rate less than 100 vehicles per h	Apply prewetted solid chemical		7-18 (25-65)	Reapply prewetted solid chemical as needed		7-18 (25-65)	1) Monitor pavement closely; if pavement becomes wet or if thin ice forms, reapply chemical at higher indicated rate 2) Do not apply liquid chemical on ice so thick that the pavement cannot be seen
	Traffic rate greater than 100 vehicles per h	Apply liquid or prewetted solid chemical	7-18 (25-65)	7-18 (25-65)	Reapply liquid or prewetted solid chemical as needed	11-32 (40-115)	7-18 (25-65)	
-7 to -2°C (20 to 28°F), remaining in range, *and* equal to or below dew point	Any level	Apply liquid or prewetted solid chemical	18-36 (65-130)	18-36 (65-130)	Reapply liquid or prewetted solid chemical when needed	18-36 (65-130)	18-36 (65-130)	1) Monitor pavement closely; if thin ice forms, reapply chemical at higher indicated rate 2) Applications will need to be more frequent at higher levels of condensation; if traffic volumes are not enough to disperse condensation, it may be necessary to increase frequency 3) It is not advisable to apply a liquid chemical at the indicated spread rate when the pavement temperature drops below -5°C (23°F)
-10 to -7°C (15 to 20°F), remaining in range, *and* equal to or below dew point	Any level	Apply prewetted solid chemical		36-55 (130-200)	Reapply prewetted solid chemical when needed		36-55 (130-200)	1) Monitor pavement closely; if thin ice forms, reapply chemical at higher indicated rate 2) Applications will need to be more frequent at higher levels of condensation; if traffic volumes are not enough to disperse condensation, it may be necessary to increase frequency
Below -10°C (15°F), steady or falling	Any level	Apply abrasives			Apply abrasives as needed			It is not recommended that chemicals be applied in this temperature range

Notes

TIMING. (1) Conduct initial operation in advance of freezing. Apply liquid chemical up to 3 h in advance. Use longer advance times in this range to effect drying when traffic volume is low. Apply prewetted solid 1 to 2 h in advance. (2) In the absence of precipitation, liquid chemical at 21 kg/lane-km (75 lb/lane-mi) has been successful in preventing bridge deck icing when placed up to 4 days before freezing on higher volume roads and 7 days before on lower volume roads.

Table 20: Weather Event: Freezing Rain Storm

PAVEMENT TEMPERATURE RANGE, AND TREND	INITIAL OPERATION		SUBSEQUENT OPERATIONS		COMMENTS
	maintenance action	chemical spread rate, kg/lane-km (lb/lane-mi)	maintenance action	chemical spread rate, kg/lane-km (lb/lane-mi)	
Above 0°C (32°F), steady or rising	None, see comments		None, see comments		1) Monitor pavement temperature closely for drops toward 0°C (32°F) and below 2) Treat icy patches if needed with prewetted solid chemical at 21-28 kg/lane-km (75-100 lb/lane-mi)
Above 0°C (32°F), 0°C (32°F) or below is imminent	Apply prewetted solid chemical	21-28 (75-100)	Reapply prewetted solid chemical as needed	21-28 (75-100)	Monitor pavement temperature and precipitation closely
-7 to 0°C (20 to 32°F), remaining in range	Apply prewetted solid chemical	21-70 (75-250)	Reapply prewetted solid chemical as needed	21-70 (75-250)	1) Monitor pavement temperature and precipitation closely 2) Increase spread rate toward *higher indicated rate* with decrease in pavement temperature or increase in intensity of freezing rainfall 3) Decrease spread rate toward *lower indicated rate* with increase in pavement temperature or decrease in intensity of freezing rainfall
-10 to -7°C (15 to 20°F), remaining in range	Apply prewetted solid chemical	70-110 (250-400)	Reapply prewetted solid chemical as needed	70-110 (250-400)	1) Monitor precipitation closely 2) Increase spread rate toward *higher indicated rate* with increase in intensity of freezing rainfall 3) Decrease spread rate toward *lower indicated rate* with decrease in intensity of freezing rainfall
Below -10°C (15°F), steady or falling	Apply abrasives		Apply abrasives as needed		It is not recommended that chemicals be applied in this temperature range

Notes

CHEMICAL APPLICATIONS. (1) Time initial and subsequent chemical applications to *prevent* glaze ice conditions. (2) Apply chemical ahead of traffic rush periods occurring during storm.

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Table 21: Weather Event: Sleet Storm

PAVEMENT TEMPERATURE RANGE, AND TREND	INITIAL OPERATION		SUBSEQUENT OPERATIONS		COMMENTS
	maintenance action	chemical spread rate, kg/lane-km (lb/lane-mi)	maintenance action	chemical spread rate, kg/lane-km (lb/lane-mi)	
Above 0°C (32°F), steady or rising	None, see comments		None, see comments		1) Monitor pavement temperature closely for drops toward 0°C (32°F) and below 2) Treat icy patches if needed with prewetted solid chemical at 35 kg/lane-km (125 lb/lane-mi)
Above 0°C (32°F), 0°C (32°F) or below is imminent	Apply prewetted solid chemical	35 (125)	Plow as needed, reapply prewetted solid chemical when needed	35 (125)	Monitor pavement temperature and precipitation closely
-2 to 0°C (28 to 32°F), remaining in range	Apply prewetted solid chemical	35-90 (125-325)	Plow as needed, reapply prewetted solid chemical when needed	35-90 (125-325)	1) Monitor pavement temperature and precipitation closely 2) Increase spread rate toward *higher indicated rate* with increase in sleet intensity 3) Decrease spread rate toward *lower indicated rate* with decrease in sleet intensity
-10 to -2°C (15 to 28°F), remaining in range	Apply prewetted solid chemical	70-110 (250-400)	Plow as needed, reapply prewetted solid chemical when needed	70-110 (250-400)	1) Monitor precipitation closely 2) Increase spread rate toward *higher indicated rate* with decrease in pavement temperature or increase in sleet intensity 3) Decrease spread rate toward *lower indicated rate* with increase in pavement temperature or decrease in sleet intensity
Below -10°C (15°F), steady or falling	Plow as needed		Plow as needed		1) It is not recommended that chemicals be applied in this temperature range 2) Abrasives can be applied to enhance traction

Notes

CHEMICAL APPLICATIONS. (1) Time initial and subsequent chemical applications to *prevent* the sleet from bonding to the pavement. (2) Apply chemical ahead of traffic rush periods occurring during storm.

1.10 Kuennen, T. "Open – Graded Mixes: Better the second time around". American City & County, August 1996

1.10.1 General
With asphalt modifiers providing stability and noise suppression, open-graded mixes are receiving the applause they deserve after an initial mixed response. This article describes how open graded friction course (OGFC) (also known as "popcorn" or porous asphalt) is getting a second look by the asphalt pavement industry. According to Kuennen, the uses of the newest polymer modifiers lead to a more durable asphalt binder.

1.10.2 Benefits
According to the American City & County magazine article, the benefits from the use of OGFC are fast drainage of water from the surface as well as the virtual elimination of tire spray and hydroplaning. Researchers in the United States and Europe have stated that OGFC provides instant reduction in annoying tire/pavement noise by as much as 5 dB(A), although the effect diminishes with time. The researchers were also quick to say all the benefits come with a high cost, about 35 percent more per ton than conventional asphalt pavements.

1.10.3 Materials and Mix Design
OGFC is a HMA that incorporates a uniform aggregate size with a minimum amount of fines. Air void contents range from as low as 12 percent to as high as 15 percent, while in Europe it is even higher (17 to 22 percent). The stone-on-stone structure can hold up better to heavy traffic, especially trucks. The texture created by the use of larger aggregates without fines provides better traction.

1.10.4 Construction Practices
According to American City & County magazine article, OGFCs are constructed as a conventional asphalt mix. The major difference is the materials and structural design practices.

1.10.5 Maintenance Practices
No specifics of maintenance practices were described in this article.

1.10.6 Rehabilitation Practices
No specifics of rehabilitation practices were described in this article.

1.10.7 Performance
In addition to high drainage ability combined with minimized spray, hydroplaning, reduced glare at night and improved skid resistance; OGFC helps to attenuate highway noise. This capability has been studied extensively in Europe where the environmental impact of highway in high-density urban area has undergone close scrutiny for years.

1.10.8 Structural Design
No specifics on inclusion within structural design were given.

1.10.9 Limitations

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The clogging from debris and fines leads to the reduction in the permeability of the surface over a period of time.

1.11 Tolman, F. and F. van Gorkum, Mechanical Durability of Porous Asphalt, Eurobitume, 1996.

1.11.1 General

In this paper Tolman and Gorkum present the results of a study carried out to evaluate the cyclic tensile test to differentiate mixes with different degrees of aging. The specimens were obtained from pavements constructed with porous mixes at different times in the Netherlands. First a set of “conventional” tests were conducted, and some specimens were aged in the laboratory. These tests produced results with a significant amount of scatter and could not be reliably used to differentiate mixes with different degrees of aging.

The authors then provided the results of cyclic tensile testing and mention that the minimum creep rate model, with a relationship between slope of the secondary part of the creep versus time curve and failure time, could differentiate the mixes with different degrees of aging quite well. They pointed out the need for availability of visual survey data and project data to analyze the results further, and the need to draw more meaningful conclusions to interpret the results from the cyclic tensile tests.

1.11.2 Benefits of Permeable Asphalt Mixtures

No information has been provided on benefits.

1.11.3 Materials and Design

Tolman and Gorkum provide the description of a porous asphalt that has been developed as a result of the study of field sections constructed between the seventies and nineties. This mix, along with standard mix, is described in Table 22.

Table 22: Standard and Proposed New Porous Mix

Gradation Sieve Size	Mix Description		
	Dutch standard 0/16	Dutch standard 0/11	Proposed new mix
	Percent Passing	Percent Passing	Percent Passing
C 16	0.0-5.0	---	0.0-5.0
C11.2	15.0-30.0	0.0-5.0	18.8-28.5
C 8	50.0-65.0	60.0-85.0	50.0-70.0
C 5.6	70.0-85.0	80.0-85.0	70.0-90.0
2 mm	85.0	85.0	---
63 mm	95.5	95.5	---
Asphalt binder content, percent	4.5	4.5	5.5
Fiber, percent	---	---	0.3-0.7

1.11.4 Construction Practices

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No information has been provided on construction practices.

1.11.5 Maintenance Practices
No information has been provided on maintenance practices.

1.11.6 Rehabilitation Practices
No information has been provided on rehabilitation practices

1.11.7 Performance
In the introduction, Tolman and Gorkum mention that durability of porous asphalt can be expressed in terms of mechanical and functional durability. They list a set of damage conditions, as shown in Table 23.

Table 23: Problems with Porous Mixes

Type of Problem	Specific Problems
Functional	Clogging of pores – decreased noise reduction and discharge of water from road surface, surface polishing, unevenness caused by either loss of aggregate or deformation of underlying layers, icing in winter
Mechanical	Concentrated stress – such as due to rim of flat tire, high shear stress, such as in sharp bends, gradual loss of aggregates leading to raveled surface and/or potholes.

Tolman and Gorkum presented the results of a partial study that was started in 1990, to study the loss of aggregate and resulting damage in porous asphalt. They mention that 19 sections were built with different materials and mix types on A10 Motorway around Amsterdam. This motorway had 3 lanes and an average traffic volume of 96,000 vehicles per day (as reported in 1995). Table 24 shows the different mixes used in the study, which include sections on A10 as well as on A12 (with 3 lanes, and 72,000 vehicles per day traffic in 1995). In addition, four other test sections were monitored – one in Belgium (N5) and three in Netherlands (A16 and two on A28).

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Table 24: Mixes used in the Test Sections around Amsterdam

Materials/Mixes	Number of sections
Sections on A10, each section of 250 m length	
Standard Dutch PA 0/16, with pen 80/100 asphalt binder	1 section
Standard Dutch PA 0/16, with 4.5 % SBS modified binder	2 sections
Standard Dutch PA 0/16, with 5.5 % SBS modified binder	2 sections
Standard Dutch PA 0/16, with 4.5 % EVA modified binder	2 sections
Standard Dutch PA 0/16, with 5.5 % EVA modified binder	2 sections
Standard Dutch PA 0/16, with 4.5 % tire rubber scrap modified binder	2 sections
Standard Dutch PA 0/16, with 4.5 % tire rubber scrap modified binder	2 sections
Standard Dutch PA 0/16, with pen 160/200 asphalt binder	2 sections
Standard Dutch PA 0/16 with changed gradation and asphalt content of 5.5%	2 sections
Standard Dutch PA 0/16 with cellulose fibers	1 section
Standard Dutch PA 0/16 with mineral fibers	1 section
Sections on A12, each section of 500 m length	
Standard Dutch PA 0/16	1 section
Standard Dutch PA 0/16 with a resin modified binder	1 section
Standard Dutch PA 0/16 with a multigrade binder	1 section
Standard Dutch PA 0/16 with a high asphalt binder content, with smaller size and uniformly graded aggregates and mineral fibers	1 section
Sections monitored periodically- details of materials and mixes not provided	
N5 in Belgium	
Heavy traffic section on A16 (built in 1985)	1 section
Section on A28 (built in 1977)	1 section
Section on A28 (built in 1985)	1 section

To simulate long term aging, and then compare the aged laboratory samples to the aged field samples, the authors conducted some review and then selected the following process (in the following sequence): 16.25 hours open air at a temperature of 50^{o}C, 4 hours sprinkling of water containing sodium chloride at 40^{o}C, 1 hour of rain water at 20^{o}C, and then finally 2.75 hours of frost at -20^{o}C. The authors could confirm the aging process from the reduction in penetration of the recovered binder.

Tolman and Gorkum then mention that several tests were carried out to discriminate the different mixes, all of which failed to provide reliable results. Table 25 shows the lists of problems encountered with the specific tests.

Table 25: Problems with Tests

Test	Problem	Recommendation
Constant displacement rate indirect tensile test	- Too much scatter in the data - Not enough theoretical background	---
Cantabro test	- Too much scatter in the data - Not enough theoretical background	Provide strict control of temperature during test to reduce scatter
Wheel tracking test	- Too much scatter in the data - Not enough theoretical background	Circular track preferable to “to and fro” tracks, which suffer from end effects

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The authors mention that results from the above tests did not correlate well with results of visual condition surveys of the different sections. Some observations, as presented by the authors, are shown in Table 26.

Table 26: Observations from Condition Survey

Section	Observation	Cause/Inference
A10, A12	No damage	Relatively young sections
N5	Loss of aggregates	Asphalt binder content less than 4.5 %; high binder content reduces loss at the cost of functional properties
A18 (1977 section)	Rough surface	Loss of aggregates

Tolman and Gorkum mention that the drawbacks of the above tests were overcome with the use of a cyclic tensile test, with the use of minimum rate of the creep curve or the time to failure parameters. They contend that this test produced results which were able to differentiate binders at different stages of aging.

Tolman and Gorkum then proceeded to provide details of the cyclic test carried out on cores for the different test sections. The details of the test are provided in Table 27.

Table 27: Details of Cyclic Tensile Test

Sample/Variable	Type
Cores	40 mm high, 100 mm diameter
Loading	Repeated; frequencies
Stresses to cause failure in binder film	Tensile stress
Temperature	Increasing damage at lower temperatures
Test conditions (2)	20^{o}C, 1 Hz, 100-800 N 0^{o}C, 30 Hz, 100-6,000 N

Tolman and Gorkum mention that the minimum creep rate model using results from cyclic tensile tests under the different conditions showed good relationship between the slope of the secondary part of the creep curve and the failure time. They indicate that mixes with different stages of aging were found to produce straight lines, in log-log scale (creep rate versus failure time), which were parallel to each other, with a characteristic slope of -1.15.

Tolman and Gorkum mention that in terms of damage in porous mixes, a deviation from the average level (for any durability parameter) seems to be more important that a decrease in average level, and that damage increases rapidly once a specific damage level is reached. They also mention that it is impossible to qualify and quantify durability parameters on the basis of one method.

Tolman and Gorkum end by stating that the cyclic direct tensile test seems to be the only method capable of discriminating mixes with different degrees of aging and that it offers significant scope of testing with respect to obtaining modulus and phase angle.

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1.11.8 Structural Design
No information has been provided on structural design.

1.11.9 Limitations
No information has been provided on limitations.

1.12 Santha, L. "A Comparison of Modified Open-Graded Friction Courses to Standard Open-Graded Friction Course." FHWA-GA-97-9110. Georgia Department of Transportation. Forest Park, Georgia. April 1997.

1.12.1 General
This report documents a research study conducted by the Georgia Department of Transportation (GDOT) to determine the improvements made to existing GDOT OGFC mixes. Test sections of seven OGFC mixtures were constructed on Interstate 75 south of Atlanta, Georgia. The average daily traffic (ADT) for the interstate was 47,000 with approximately 21 percent truck traffic. The existing pavement prior to construction of the test sections consisted of 140mm of HMA overlying a 9 in. Portland cement concrete pavement that was jointed at 30 ft intervals. In the test sections, 54mm of existing pavement was milled and 35mm of dense-graded surface HMA was then inlaid. The OGFC test mixtures were then placed at a thickness of 19mm over the dense-graded surface mixture. Each of the seven test sections were constructed approximately 0.8km in length. The following lists the seven mixtures placed during the study:

Test Mixture	Description
d	Standard OGFC
D	Coarse OGFC (after improvements)
D16R	Coarse OGFC with 16% Crumb Rubber
DM	Coarse OGFC with mineral fibers
DC	Coarse OGFC with cellulose fibers
DP	Coarse OGFC with SB polymer
DCP	Coarse OGFC with SB polymer & cellulose fibers

Based upon the results of this study, Santha concluded the following:

1. "The use of coarse OGFC should be continued and the use of fine OGFC discontinued."
2. "Both fiber and polymer should be included in coarse OGFC. Neither additive appears to perform significantly better or worse than the other in coarse OGFC."
3. "The use of crumb rubber in coarse OGFC is not recommended."

1.12.2 Benefits of Permeable Asphalt Mixtures
Benefits mentioned by the author included improved wet weather driving conditions which includes reduced hydroplaning and splash/spray and improved pavement friction and surface reflectivity.

1.12.3 Materials and Mix Design
Historically, GDOT had used fog seals to help hold aggregates within OGFC when raveling problems had arisen. Fog seals fill surface voids within OGFC and, thus, reduce permeability.

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In an effort to better maintain permeability, GDOT began to require a larger nominal maximum aggregate size gradation for OGFCs. The GDOT specifications also required the addition of a thermoplastic modifier and mineral fibers in order to help hold the aggregates in-place and to eliminate draindown during construction. Table 28 presents the gradation requirements of the standard OGFC and coarser OGFC used by GDOT (and this research study).

Table 28: GDOT OFGC Gradation Requirements

	Percent Passing	
Sieve Size, mm	Standard OGFC	Coarse OGFC
19.0	---	100
12.5	100	90-100
9.5	85-100	65-85
4.75	20-40	15-25
2.38	5-10	5-10
0.075	2-4	2-4

The design of the OGFC mixtures entailed three primary steps. The first step in the mix design process was to determine the surface capacity of the aggregate fraction that is retained on the 4.75mm sieve. To determine the surface capacity, the aggregate fraction was completely immersed in S.A.E. No. 10 oil for five minutes. After draining the oil from the aggregates, the percent oil retained was determined. The oil retained on the aggregates was then used to calculate the required asphalt binder content for the mix.

The second step of the GDOT mix design process was a modified Marshall design. Mixture was compacted at varying asphalt binder contents using 25 blows per face with the Marshall hammer. For each mixture, the bulk specific gravity was determined using volumetric calculations and the mass of each sample. Using the bulk specific gravity, the volumetric properties of each sample were determined. A plot was then developed between voids in mineral aggregate (VMA) and asphalt binder content. Optimum asphalt binder content was selected as the binder content that produces the lowest VMA.

The final step in the process was to select an asphalt binder content based upon the Pyrex Bowl Method. For this method, mixture was prepared and placed into a clear glass (Pyrex) bowl. The GDOT method states to start with a binder content of 5.5 percent and then repeat the process for asphalt binder contents of 6.0, 6.5 and 7.0 percent. The Pyrex bowls were placed in an oven set at 121˚C (250°F) for one hour. Then, a visual examination of the amount of liquid that drained from the aggregate structure and left on the Pyrex bowl was conducted. An asphalt binder content was selected where ample bonding was evident without excessive draindown.

Optimum asphalt binder content for the mixture was selected as the average binder content from the three steps described above. In addition, two other tests were conducted to evaluate the designed mixes: Cantabro Abrasion and the Schellenberg Drainage test. Table 29 summarizes the designed mixes.

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Table 29: Summary of Mix Design Information of Research Mixes

Sample Type	Coarse OGFC (D)	D+16% Rubber (D16R)	D+ Mineral Fibers (DM)	D+Cellulose Fibers (DC)	D+SB Polymer (DP)	DC+SB Polymer (DCP)
Sieve Size	Total Percent Aggregate Passing By Mass					
¾ in.	100	100	100	100	100	100
½ in.	99	99	99	99	99	99
⅜ in.	75	75	75	75	75	75
No. 4	18	18	18	18	18	18
No. 8	8	8	8	8	8	8
No. 200	2	2	2	2	2	2
	Percent Asphalt Binder in Total Mix					
% AC	6.0	6.6	6.3	6.4	6.2	6.4
	Miscellaneous Test Data					
Cantabro (% Wear)	13.5	8.6	5.7	5.8	8.6	8.2
Drainage (% Loss)	0.37	0.05	0.06	0.06	0.34	0.04
Film Thickness (µm)	34.07	36.90	35.92	36.54	35.30	36.54

Little information was provided for the materials. The base asphalt binder before any modification was an AC-20 Special. Both hydrated lime and a liquid anti-strip agent were added to the mixture in order to minimize the potential for moisture damage. All aggregates were a granite-gneiss that had a Los Angeles Abrasion of 42 percent.

1.12.4 Construction Practices

A drum mix plant was used to produce the OGFC mixes used in the research study. Mix exiting the drum was carried to a silo for depositing into the haul trucks. In order to minimize the potential for draindown, a maximum of 50 tons of OGFC was stored in the silo at any given time. The production rate for the OGFC test mixes was 150 tons/hr, while the production of typical dense-graded mixes was 250 tons/hr.

Two modifications to the plant were required in order to produce the OGFC: introduction of the fibers and asphalt rubber. In order to introduce the fibers, a fiber blowing machine was used. This machine took baled fibers, fluffed the fiber, crowded the fiber to a constant density using a series of paddles and augers, and then blew the fibers to the drum using an air jet. Fibers were blown into the drum at the same location as the hydrated lime. This location was selected based upon a preliminary investigation designed to determine the best location. The two location points investigated included the point at which hydrated lime and asphalt binder were introduced. Trial mixes were produced in order to evaluate these two locations. Samples of baghouse fines were obtained and evaluated to determine if the fibers were being caught in the high velocity drum exhaust gases. Evaluation of the baghouse fines showed no appreciable amount of fibers within the baghouse fines when either of the introduction locations were utilized. Tensile strength tests were also conducted on mix produced when using the two introduction points. Santha indicated

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that tensile strengths were "slightly higher" for OGFC mix produced when the fibers were added at the asphalt binder injection point; however, it was decided to introduce the fibers at the point that the hydrated lime was added. This decision was based upon this location allowing the fibers to be dry mixed with the aggregates for a short time before the asphalt binder was introduced. The fiber hopper used to introduce the fiber into the mixing process was calibrated prior to initiation of production. The fiber was blown into a pre-weighed sealed container for one minute.

The rubber modified asphalt binder was mixed on-site using a batch type blending unit. The rubber modified binder was gravity fed into a pump which fed the mixing process.

Both the conventional and modified OGFC mixes were placed and compacted with no major problems. The following are excerpts from the report on some minor issues during placement and compaction.

"*Coarse OGFC with Mineral Fibers (DM).* During laydown, this mix behaved as if it were too cold, even though temperatures fell within job mix formula limits. During the lab design process, GDOT observed that the mix could probably be placed at a higher temperature without the risk of draindown. The JMF was set at 275°F, and the truck temperatures taken at the plant averaged 278°F."

"*Coarse OGFC with Cellulose Fibers (DC)*. This mix, which contained cellulose fibers, had a dull, flat appearance. Problems were encountered when the mix stuck to the roller drums and was picked up. Keeping the rollers back to allow the mix to cool before rolling seemed to help this problem, but adding soap or releasing agent to the drum watering system is likely to be the best alternative…"

"*Coarse OGFC with SB Polymer and Cellulose Fibers (DCP).* This mix contained SB polymer and cellulose fibers, and it was difficult to lay so that it matched the adjoining, previously placed lane. The thickness depended on paver speed, and any increase or decrease in speed would cause the mat to get thinner and thicker, respectively…"

"*Coarse OGFC with SB Polymer (DP).* This mix contained Styrelf polymer, and it was the most difficult to lay. Paver speed changes greatly affected the thickness of the mix, as with the DCP mix. Cores taken from this section had thickness variations of up to 0.4 in. AC draindown occurred in four loads, indicating that the JMF placement temperature (300°F) was probably set too high… The difficulty in laying this mix and the DCP mix is reflected in the relatively high Maysmeter values of these mixes compared with those of the other mixes…"

"*Coarse OGFC with 16% Crumb Rubber (D16R).* This mix, which contained 16% crumb rubber, produced only minor laying or compaction problems. Some pulling of the mat occurred for approximately the first 75 feet but disappeared shortly afterward. The paver screed may have been cold, since this mix was the first which was placed that day…"

1.12.5 Maintenance Practices

No specific maintenance practices were given.

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1.12.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.12.7 Performance
Each of the test sections were monitored for 3.5 years after construction. Properties of the test sections that were monitored included: friction, smoothness, visual distress, rutting and permeability.

Friction testing was conducted in accordance with ASTM E274 at the following time increments after construction: one day, two weeks, six months, and 3.5 years. Table 30 presents the results of this testing. These results show that initial friction values (one day) were relatively low compared to the other time increments. However, the friction numbers increased significantly at the two week increment and maintained relatively high friction numbers for the 3.5 years.

Table 30: Average Friction Test Results for Test Sections

Test Section	Friction Number			
	10/27/92	11/11/92	4/12/93	2/6/96
Std. OGFC (d)	42	53	52	50
Coarse OGFC (D)	41	50	52	51
D + Mineral Fibers (DM)	39	50	53	49
D + Cellulose Fibers (DC)	37	47	53	49
DC + SB Polymer (DCP)	35	46	52	50
D + SB Polymer (DP)	32	47	51	51
D + 16% Crum Rubber (D16R)	37	48	53	51

At 3.5 years after construction, smoothness testing was conducted using a new laser profiler. Table 31 presents the results of smoothness testing conducted on each of the seven test sections. An acceptable roughness in Georgia for OGFC is 750 mm/km. Results shown in Table 31 indicate that after 3.5 years, all of the sections had acceptable smoothness.

Table 31: Average Smoothness Values in Test Sections

Test Section	Smoothness Value (5/1996), HRI, mm/km
Std. OGFC (d)	427
Coarse OGFC (D)	547
D + Mineral Fibers (DM)	564
D + Cellulose Fibers (DC)	551
DC + SB Polymer (DCP)	543
D + SB Polymer (DP)	585
D + 16% Crum Rubber (D16R)	642

Following construction, visual distress surveys were conducted annually on the different test sections. After 3.5 years, the primary visual distresses consisted of reflective cracking and some longitudinal cracking. Table 32 presents the results of the distress survey on test sections.

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Table 32: Results of Visual Distress Survey on Test Sections

Test Section	Description of Distresses	# Cracks in 0.2 Mile
Std. OGFC (d)	Low severity reflective cracks	7
Coarse OGFC (D)	Medium severity reflective cracks and some longitudinal cracks at right edge of right lane	26
D + Mineral Fibers (DM)	Low severity reflective cracks	10
D + Cellulose Fibers (DC)	Low severity reflective cracks	3
DC + SB Polymer (DCP)	Low severity reflective cracks	12
D + SB Polymer (DP)	Low severity reflective cracks	4
D + 16% Crum Rubber (D16R)	Low-medium severity reflective cracks & some longitudinal cracks at right edge of right lane	20

Rut depth measurements were taken in each of the test sections at several time intervals up to 3.5 years. Rut depths were measured with a string line at 100 ft intervals in both the left and right wheel path. Table 33 presents the average rut depths for each test section in 1993, 1994 and 1995. This table shows that rut depths were relatively small in all of the sections containing the Coarse OGFC gradation with the different additives. The largest rut depths were observed within the Standard OGFC section after 3.5 years of traffic.

Table 33: Average Rut Depths in Test Sections (inches)

Test Section	11/93		9/94		2/96	
	LWP	RWP	LWP	RWP	LWP	RWP
Std. OGFC (d)	---	---	---	---	0.25	0.28
Coarse OGFC (D)	0.09	0.06	0.14	0.17	0.16	0.10
D + Mineral Fibers (DM)	0.17	0.14	0.14	0.19	0.07	0.14
D + Cellulose Fibers (DC)	0.16	0.14	0.13	0.17	0.15	0.17
DC + SB Polymer (DCP)	0.11	0.05	0.05	0.14	0.12	0.06
D + SB Polymer (DP)	0.13	0.03	0.03	0.12	0.18	0.08
D + 16% Crum Rubber (D16R)	0.07	0.04	0.04	0.14	0.16	0.17

An in-place permeability test was used to evaluate the drainage characteristics of the OGFC sections at the time of construction. Permeability tests were also conducted after 1 year and 3.5 years. Table 34 shows the average results of permeability testing conducted on the test sections. Permeability tests were conducted within the wheel paths and between the wheel paths at a minimum of three locations along the length of each test section. This table shows that the permeability of all test sections decreased substantially over time. Santha states that the decrease in permeability was due to clogging.

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Table 34: Average Permeability Results Over Time

Test Section	Average Permeability, ft/day			Permeability in 1996 as % of Permeability in 1992
	1992	1993	1996	
Std. OGFC (d)	146	21.9	7.8	5
Coarse OGFC (D)	142	31.0	28.9	20
D + Mineral Fibers (DM)	254	66.7	12.9	5
D + Cellulose Fibers (DC)	222	61.3	28.4	13
DC + SB Polymer (DCP)	220	67.4	18.6	8
D + SB Polymer (DP)	262	95.6	11.0	4
D + 16% Crum Rubber (D16R)	210	55.9	25.6	12

Another interesting observation about the permeability data was that the permeability values between the wheel paths was generally lower than the results of testing within the wheelpaths. Santha indicated that this was an indication of the action of traffic cleaning the OGFC layer.

1.12.8 Structural Design
The test sections were placed at a thickness of 19 mm.

1.12.9 Limitations
No specific limitations were given.

1.13 Tolman, F. and F. van Gorkum. "A Model for the Mechanical Durability of Porous Asphalt." European Conference on Porous Asphalt. Madrid. 1997.

1.13.1 General
The objective of the study reported in this paper was to demonstrate the use of cyclic tensile tests in explaining the difference in behaviors of different porous asphalt mixes. Starting with the premise that failure in porous asphalt occurs mostly at the interface between binder and aggregates, Tolman and Gorkum provided results from tensile tests conducted on cores from six different porous asphalt pavements in Netherlands. Using the minimum creep rate model, the authors derived parameters required for explanation of fracture and ductile failure mechanisms. They were able to derive fundamental parameters to explain the differences in behaviors between cores obtained from different pavements (of different ages).

1.13.2 Benefits of Permeable Asphalt Mixtures
No specific benefits were given.

1.13.3 Materials and Design
Tolman and Gorkum provide definition and description of porous asphalt. They mention that porous asphalt is a skeleton of stones bound by "visco-elastic" deteriorating mortar, which consists of asphalt, added filler, aggregate filler and particles embedded in the asphalt film.

Tolman and Gorkum indicate that the principal factors affecting the strength of porous asphalt are binder stiffness, film thickness and minerals in binder film. They mention that failure in porous asphalt has been seen to occur primarily due to the fracture of binder films and not due to

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separation from aggregates or fracture of aggregate particles. They mention that the separation or detachment (or stripping) has been assumed to be the result of physicochemical actions. The authors make reference to three papers and mention that there is a critical stiffness level for porous asphalts, below which the failure occurs by flow, termed as ductile failure, and above which failure occurs by fracture, termed as brittle failure.

1.13.4 Construction Practices
No information has been provided on construction practices.

1.13.5 Maintenance Practices
No information has been provided on maintenance practices.

1.13.6 Rehabilitation Practices
No information has been provided on rehabilitation practices

1.13.7 Performance
In discussing the objectives of the study, Tolman and Gorkum mention that loss of aggregates occur in porous asphalts. They indicate that this phenomenon, which is a combination of raveling, fretting and disintegration, occurs probably because of stretching of binder film due to traffic, particularly during warm weather. They contend that this stretching causes intrusion of dust and exposure to materials such as oxygen, UV and water, which in turn, leads to hardening of the porous asphalts. The authors indicate that most likely, this increased hardening, with a drop in temperature, leads to fracture, during the winter periods. Once the porous asphalt cracks, the ingress of water causes separation of asphalt binder from the aggregate.

Tolman and Gorkum mention that it is the fracture at the interface of the aggregate and asphalt binder, and the stiffness of the binder film that are important for understanding the failure mechanism in porous asphalt. They provide estimated values of tensile stress and resulting strains in binder films under different conditions, as shown in Table 35.

Table 35: Stresses and Strains in Binder Films in Porous Asphalt

Condition	Horizontal tensile stress, maximum, MPa / Stress gradient	Strains, %
Aggregates not embedded in binder	3 MPa/-	300 %
Aggregates embedded in binder	-/0.2 MPa/mm	100 %

Note: Assumptions: 1. Asphalt binders stiffness: 1 MPa, typical length of aggregate particle = 6 mm, thickness of binder film = 1mm

Tolman and Gorkum mention that high stresses (for example, 3 MPa) can overcome relatively low binder stiffness (1 MPa) and cause a high amount of strain and ultimately failure. They contend that failure can occur in one of three ways: 1. brittle failure, caused by fracture of binder layer, 2. ductile failure caused by shear failure, due to flow and contraction of the binder layer, and 3. a combination of two processes, first a flow and contraction of the binder layer and then a brittle fracture.

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Tolman and Gorkum then proceeded to describe the results of modeling using data from load controlled cyclic tensile tests carried out on cores from pavements of different ages in Netherlands. They mention several adoptions for modeling: 1) Assumption of failure of binder film between two aggregates; 2) Assumption of a continuum with randomly distributed defects; 3) Fracture model with a zone of growing defects; and 4) Flow model with elongation and contraction forces.

Through a series of equations and plots Tolman and Gorkum illustrate their approach in modeling the data from the pavement cores. They used 3 parameters from the tests, for each specimen: 1) The slope of the deformation versus time at the inflection point of the strain versus time curve; 2) The time until failure; and 3) The strain at 90 percent of the failure time. They mention that the time to failure and failure strain are functions of the load, the total binder film and the basic material parameters, which are the initial quality, the damage growth coefficient, the damage growth exponent, and the ratio of the surface energy and the fracture zone width for fracture failure or the lateral contraction coefficient for the flow failure.

Tolman and Gorkum then exhibits a series of equations on how the material parameters can be derived as functions of the time until failure from known material properties and tested properties. In a table, the authors show that the reference stress, fracture energy per fracture zone width and the contraction coefficient are correlated to the minimum creep rate parameters and the expected condition of the specimens. They contend that the references stress of the fracture mode is of the order of the fracture strength, while the reference stress of the flow model is of the order of the mortar stiffness in thin films.

Tolman and Gorkum conclude that the obtained and derived parameters from the cyclic tensile tests are meaningful, relevant, consistent and discriminating between the different types of specimens.

1.13.8 Structural Design
No information has been provided on structural design.

1.13.9 Limitations
No information has been provided on limitations.

1.14 Kandhal, P.S. and R.B. Mallick. "Open Graded Asphalt Friction Course: State of Practice." Transportation Research Circular E-C005. Transportation Research Board. Washington, D.C. 1998.

1.14.1 General
This report provides the results of a survey of states conducted to evaluate current practices on the design and construction of OGFCs. A questionnaire was sent to all 50 state highway agencies and responses were received from 43 states.

At the time of the survey, 45 percent of the responding agencies were utilizing OGFC. Similarly, 45 percent had indicated that they had discontinued use. Ten percent of the agencies indicated that OGFC had not been used.

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1.14.2 Benefits of Permeable Asphalt Mixtures

The authors mentioned improved frictional resistance, especially in wet weather, as a benefit of OGFC.

1.14.3 Materials and Design

The survey included several questions about materials and mix design procedures for OGFCs. Figure 6 shows that 76 percent of the responding states indicated that they have formal mix design procedures for OGFC, 19 percent of the states reported that they used recipe specifications, and 5 percent of the states used a combination of a mix design procedure and recipe method. Forty-two percent of the states specified a range of allowable asphalt binder contents, whereas 58 percent did not. The different aggregate gradation ranges reported by the states are shown in Table 36. Figure 7 shows that 26 percent of the states followed the FHWA procedure to establish mixing temperature to prevent draindown of asphalt binder, 37 percent of the states used other draindown tests, whereas 37 percent of the states did not use any test, but used temperatures from viscosity-temperature charts for specific binders. Table 37 shows the different grades of asphalt binders used by the different agencies. Forty eight percent of the responding states used polymer modified binders, while 52 percent did not. However, these percentages are based on total number of states surveyed, including those which do not use OGFC at present. Forty six percent of the states used some type of additive, such as cellulose fiber, hydrated lime, or some form of anti-strip agents, whereas 54 percent of the states did not use any additive other than a modifier for binders. Nineteen percent of the states using additives used fiber, 13 percent used silicone, 13 percent used crumb rubber, 31 percent used liquid anti-strip agent and 44 percent used hydrated lime. These percentages total more than 100 percent because some states used more than one type additive.

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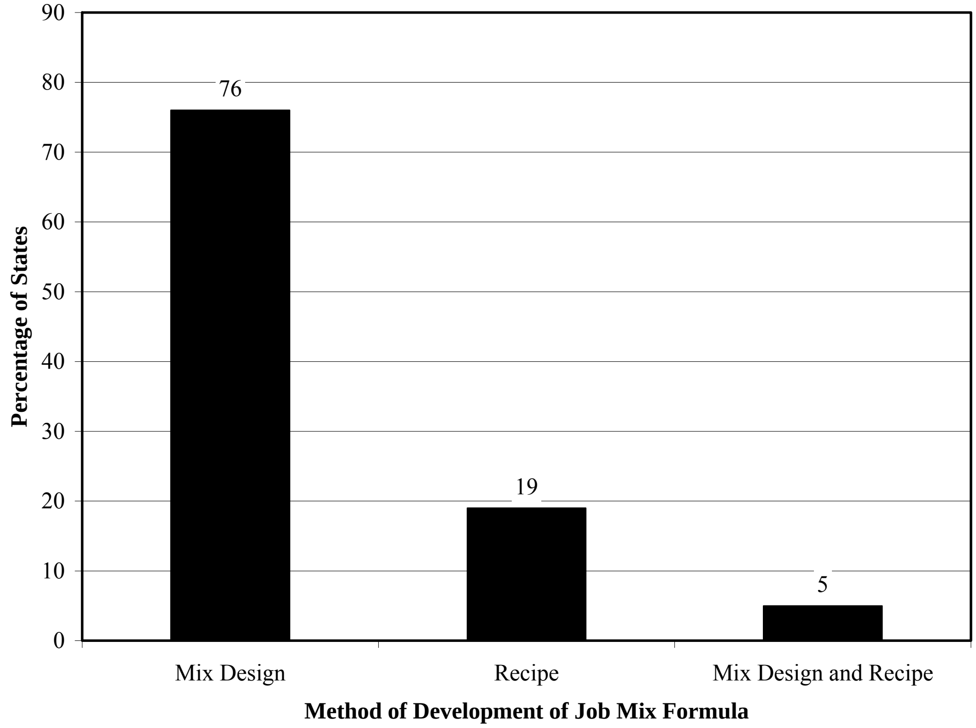

Figure 6: Methods of Developing Job Mix Formulas for OGFCs

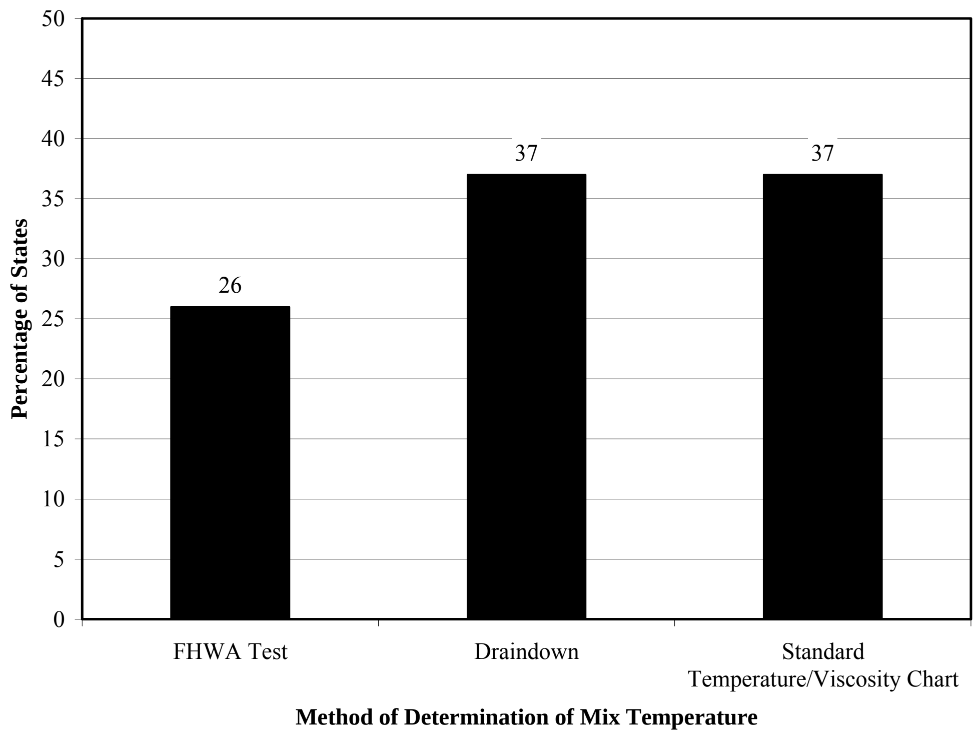

Figure 7: Methods for Determining Mixing Temperatures

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Table 36: Gradation of OGFC Mixes in Different States

State	Percent Passing Sieve (mm)												
	25	19	12.5	9.5	6.3	4.75	2.36	2	1.18	0.6	0.3	0.15	0.075
AL			100	90-100		30-50	5-7						3-6
		100	90-100	40-70		5-30	4-12						3-6
CA				78-89		28-37	7-18						
CO			100	90-100		35-57	12-33				3-15		2-8
			100	90-100		40-60	20-47				4-18		2-9
FL			100	85-100		10-40	4-12						2-5
GA		100	85-100	55-75		15-25	5-10						2-4
HI				100		30-50	5-15						2-5
ID		100	95-100	30-80		35-46				8-15			2-5
IL		100	90-100	30-50		10-18							2-5
KY			100	90-100		25-50	5-15						2-5
LA		100	90-100	30-50		10-30		5-20					2-6
				90-100		20-50	5-15						2-6
MD			100	90-100		20-40	5-15						0-5
MI			100	90-100		30-50	8-15						2-5
NV			100	90-100		35-55			5-18				0-3
				95-100		40-65			12-22				0-4
NJ			100	80-100		30-50	5-15						2-5
NM			100	90-100		25-55	0-12						0-4
NC			100	75-100		25-50	5-15						1-3
		100	85-100	55-75		15-25	5-10						2-4
OH			100	85-96		28-45	9-17						2-5
OR		99-100	90-98		25-40		2-12						1-5
	99-100	85-96	55-71		15-30		5-15						1-6
PA				100		30-50	5-15						0-5
RI		90-100				20-50	5-15						2-5
SC			100	98-100		40-70	2-20						0-2
TX		100	95-100	50-80		0-8	0-4						
UT			100	92-100		36-44	14-20						2-4
VT			100	95-100		30-50	5-15						2-5
WY			100	97-100		25-45	10-25						2-7
			100	97-100		20-40	10-20						2-7

Table 37: Asphalt Binders used for OGFCs

State	Asphalt Binder
AL	PG 76-22
CA	AR 2000, 4000, 8000
CO	AC 20R
FL	AC 30
GA	PG 76-22
HI	AR 80
ID	--
IL	AC 10
KY	PG 64-22
LA	PG 70-22
MD	AC 20
MI	---
NV	AC 20P, AC 30
NJ	AC 20
NM	---
NC	AC 20P
OH	AC 20
OR	PBA 5, PBA 6
PA	AC 20
RI	AC 20
SC	PG 64-22
TX	AC 20, AC 10
UT	PG 64-34
VT	AC 20
WY	AC 20, AC 10

To draw meaningful conclusions on materials and mix designs from the survey, the states were classified by the authors according to Strategic Highway Research Program (SHRP) climatic zone criteria into four groups: Wet-Freeze, Wet-No Freeze, Dry-Freeze, and Dry-No Freeze. Table 38 shows specific problems reported by some state agencies in these four climatic zones. In the Wet-Freeze zone the main problem seemed to be raveling of the OGFC and stripping of underlying layers. Problems not related to mix performance included difficulty in removal of snow and clogging up of voids by ice control materials such as sand and reduced permeability. In the Dry-Freeze zone, the main problem seemed to be removal of snow and closing up of voids by sand, although one state reported stripping in underlying layers. In the Wet-No Freeze zone, the problems included raveling of OGFC, stripping of underlying layers, and closing up of voids. In the Dry-No Freeze zone, the only reported problem was raveling of OGFC due to absorptive aggregate.

To study the differences in mix design of OGFC between states having good experience and states having bad experience with OGFC, three mix design items were listed by the authors for each state, as shown in Table 39. In the Wet-Freeze zone, most of the states which had good experience, and did use OGFC at the time of the survey used polymer modified binders, whereas those which had bad experience, and had stopped using OGFC did not use polymers. The percent passing the 2.36 mm sieve ranged from 5 and 15 percent for most of the states. Also, there was little difference in the use of other additives between states having good and bad experience.

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In the Wet-No Freeze zone, most of the states with good experience used polymers, and half of them used some other type additive such as rubber or fiber. However, most of the states with bad experience did not use polymer or any other additive type. The percentage passing the 2.36 mm sieve of the one state with bad experience which the gradation requirements were available was finer than for most of the states with good experience.

In the Dry-Freeze zone, all of the states which had good experience used hydrated lime, whereas three out of four states which had bad experience did not. The percentage passing 2.36 mm sieve seemed to be higher for states in this zone (about 10-30). Again, the most prominent difference seemed to be in the use of polymer modified binders: all of the states with good experience used polymer, whereas three out of four states which had bad experience did not use polymer.

For the Dry-No Freeze zone, all of the states with good experience used polymers and most of them use other additives. The only state with bad experience did not use polymer, but used silicone as an additive. There was no distinct difference between the percentage passing the 2.36 mm sieve used by the states with good and the state with bad experience with OGFC.

The survey on the use of OGFC revealed that the primary mix performance problems were raveling of OGFC and stripping of underlying layers. The authors indicated that raveling of OGFC is likely a problem with the loss of bond (cohesion) between the aggregate particles. The stripping of the underlying layers could be attributed to inadequate drainage of water through the OGFC. Therefore, the authors surmised that two of the most important features of OGFC mix are air voids and bonding of aggregates and asphalt binder. The drainage capacity of an OGFC is a direct function of the air voids.

Experience of states using polymer modified binders indicated that proper use of polymer and/or other additives can allow the use of higher air voids (for drainage, and hence prevent stripping in the underlying layer), higher asphalt binder content (for durability, and hence prevent raveling) by controlling draindown, as well as to provide improved adhesion and greater resistance to aging of binder.

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Table 38: Problems with OGFC

Zone: Wet-Freeze

State	Problem
IA	Removal of ice very difficult.[1]
MD	Raveling in OGFC
ME	Removal of ice very difficult[1].
MN	Deicing sand clogged voids[1]; stripping of OGFC
RI	Durability problem; widespread debonding; OGFC scraped by snow plows.
VA	Stripping in underlying layers; needed heavy fog coat after several years to prevent raveling.

Zone: Wet-No Freeze

State	Problem
AK	Filling up of voids, leading to moisture retention; prolonged freezing, and snow and ice removal problems.
LA	Extensive raveling.
TN	Stripping in underlying layers; aggregate loss in OGFC by raveling; snow and ice removal problem due to re-freezing of melted snow and ice[1].

Zone: Dry-Freeze

State	Problem
CO	Moisture damage to underlying layers.
ID	Sanding caused filling up of voids[1].
KS	During winter snow and ice storm, voids became filled with water and froze; developed icy surface; took substantially higher amount of salt to melt ice[1].
SD	Sand and salt plugged up the voids[1].

Zone: Dry-No Freeze

State	Problem
HI	Raveling because of absorptive aggregate.

Note: [1]: Problems not related to mix performance

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Table 39: Mix Design Practices of States with Good and Bad Experiences

Zone: Wet-Freeze

Good Experience				Bad Experience			
State	Polymer	Other Additive	% Passing 2.36 mm Sieve	State	Polymer	Other Additive	% Passing 2.36 mm Sieve
IL	Yes	No	---	IA	No	No	---
KY	Yes	No	5-15	MD	Yes	Antistrip	5-15
NJ	Yes	No	5-15	ME	No	No	---
OH	Yes	No	9-17	MN	No	No	---
PA	No	Antistrip	5-15	RI	No	Silicone, Antistrip	5-15
VT	No	Antistrip	5-15	WV	No	No	---

Zone: Wet-No Freeze

Good Experience				Bad Experience			
State	Polymer	Other Additive	% Passing 2.36 mm Sieve	State	Polymer	Other Additive	% Passing 2.36 mm Sieve
AL	Yes	No	5-7	AK	No	No	--
FL	No	Crumb Rubber	4-12	LA	No	No	>5-20
GA	Yes	Hydrated Lime	5-10	TN	No	No	--
NC	Yes	Fiber	5-15				
OK	Yes	No	---				
SC	No	Hydrated lime	2-20				

Zone: Dry-Freeze

Good Experience				Bad Experience			
State	Polymer	Other Additive	% Passing 2.36 mm Sieve	State	Polymer	Other Additive	% Passing 2.36 mm Sieve
NV	Yes	Hydrated lime	---	CO	Yes	No	12-33
OR	Yes	Hydrated lime	---	ID	No	Antistrip	---
UT	Yes	Hydrated lime	14-20	KS	No	No	---
WY	Yes	Hydrated lime	10-25	SD	No	No	---

Zone: Dry-No Freeze

Good Experience				Bad Experience			
State	Polymer	Other Additive	% Passing 2.36 mm Sieve	State	Polymer	Other Additive	Percent Passing 2.36 mm Sieve
CA	Yes	No	7-18	HI	No	Silicone	5-15
NM	Yes	Hydrated Lime	0-12				
TX	Yes	Fiber, Crumb Rubber	0-4				

1.14.4 Construction Practices

Most of the states specified the use of some kind of tack coat before construction of OGFC layers. As shown in Figure 8, 88 percent of the states surveyed used emulsion, where as only 8 percent used asphalt binder as tack coat material. Eight percent of the states surveyed did not use a tack coat. The percentages total more than 100 because some states allow the use of both emulsion and asphalt binder as tack coat material. Figure 9 shows that equal percentages of states specified 0.1-0.2, 0.2-0.3, 0.3-0.4, 0.4-0.5 liter sq. m application rates, respectively, whereas eight percent of the states specified an application rate of less than 0.1 liter per sq. m. Nine of the states specified a minimum air temperature to construct OGFC of 10ºC, 45 percent specified 15ºC, 32 percent specified 21ºC, and 14 percent do not have any specification. Twelve percent of the states specified minimum surface temperature of 9ºC, 35 percent specified 15ºC, 6 percent specified 21ºC, and 47 percent do not specified any minimum surface temperature. Five percent of the states specified in-place void content criteria for compaction of OGFC, 80 percent of the states specified roller weight and/or roller passes, whereas 15 percent do not have any specific compaction criteria. Eighty six percent of the states placed OGFC on new asphalt overlay in the same year, 5 percent placed after one year of traffic, whereas 9 percent of the states did not have any specific time period.

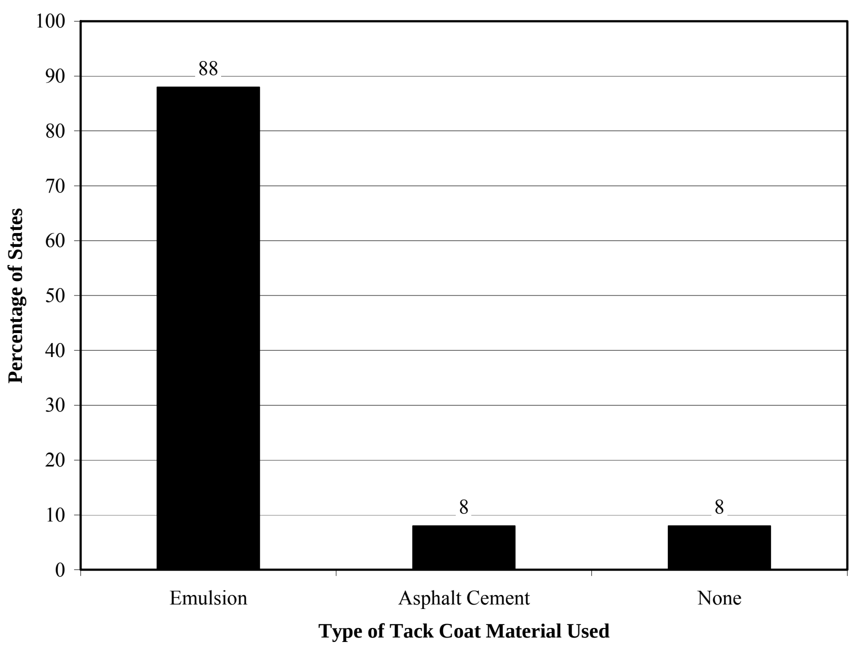

Figure 8: Tack Coat Materials Used for OGFC

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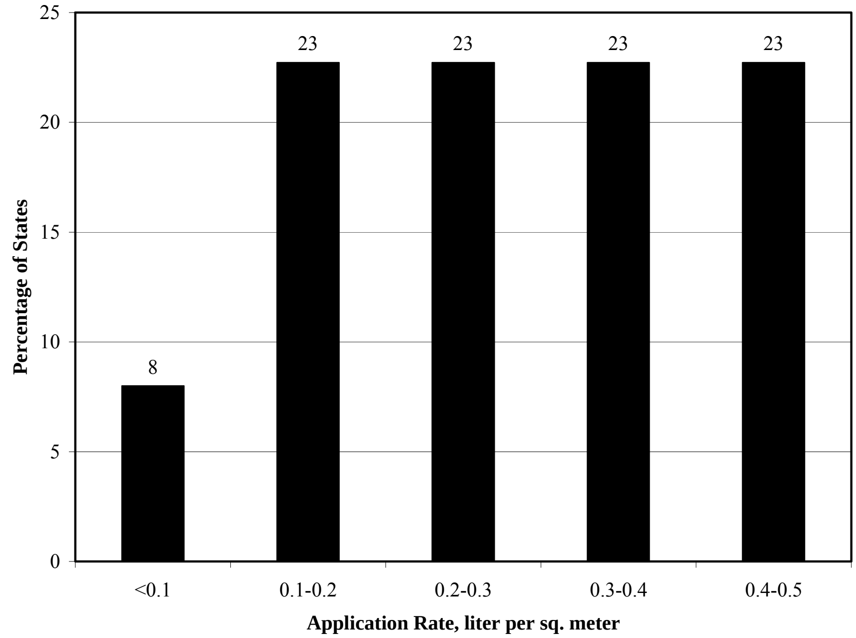

Figure 9: Tack Coat Application Rates

1.14.5 Maintenance Practices

No information on maintenance practices of porous asphalt mixtures was given.

1.14.6 Rehabilitation Practices

No information on rehabilitation of porous asphalt mixtures was given.

1.14.7 Performance

Reported average service life of OGFC by the different states is presented in Figure 10. Seventeen percent of the states reported an average service life of less than 6 years, 10 percent reported 6-8 years, 30 percent reported 8-10 years, 33 percent reported 10-12 years, whereas ten percent reported more than 12 years. Since 43 percent of states reported an average service life of more than 10 years, it indicates that OGFCs can be designed and constructed successfully.

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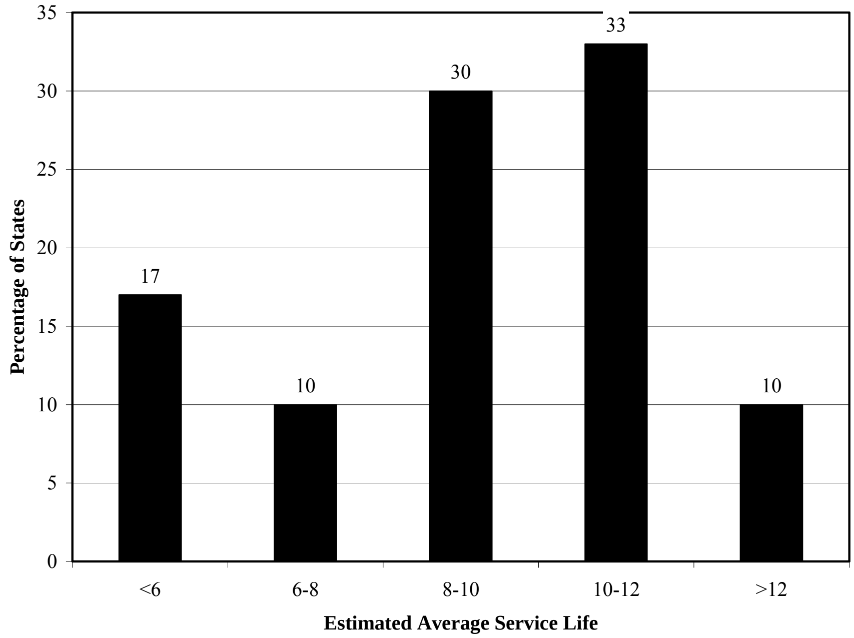

Figure 10: Reported Service Lives for OGFC

Performance of OGFC in terms of durability and surface friction were reported by highway agencies in scales of poor to excellent. In terms of durability, 11 percent of the states (surveyed) reported poor performance, 11 percent reported fair performance, 37 percent reported good performance, and 37 percent reported very good performance, whereas 4 percent indicate that they have observed excellent performance of OGFC. In terms of surface friction, none of the states reported poor performance, 4 percent reported fair performance, 11 percent reported good performance, and 55 percent reported very good performance, whereas 30 percent stated that they have observed excellent performance of OGFC. This indicates that OGFCs have generally provided good surface frictional properties as intended.

1.14.8 Structural Design

No information on structural design of porous asphalt mixtures was given.

1.14.9 Limitations

No information on limitations of porous asphalt mixtures was given.

1.15 Watson, D., A. Johnson and D. Jared. "Georgia Department of Transportation's Progress in Open-Graded Friction Course Development." Transportation Research Record No: 1616. Transportation Research Board. National Research Council. Washington, D.C. 1998.

1.15.1 General

In this paper Watson et al provide a comprehensive review of mix design and construction practices of porous asphalt mixes in Georgia. Watson et al describe the

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conventional Georgia DOT (GDOT) open-graded friction course (OGFC) mix, the porous European mix (PEM) and an overview of changes made to incorporate the good qualities of the PEM into OGFC.

With respect to materials and mix design, Watson et al mention that the use of a coarser gradation, polymer modified asphalt binder and fiber as stabilizers have enabled GDOT to produce mixes without draindown problems, and at the same time to achieve mixes with thicker asphalt films and greater durability. The elimination of the draindown problem has also enabled GDOT to produce these mixes at higher temperatures and thus improved workability.

Watson et al mentions several modifications that have been made in plants and construction practices to produce better porous mixes. These modifications include use of automatic feeders to introducing fibers into the mix, blending of asphalt binder and polymer at the asphalt terminal as opposed to a HMA plant, and the increase in production temperature. For construction, the rate of spreading has been increased to obtain thicker mats, and hence avoid irregularities on the surface. The use of proper truck insulation procedures, continuity of operation and material transfer devices has also been discussed. Drainage is ensured with an in-place permeability testing procedure.

Watson et al provides indications of better results, in terms of smoothness and permeability, with modified mixes. They discuss the higher cost associated with the modified mixes and show results of life cycle cost analyses to prove that the advantages of the modified mix far outweighs its higher initial cost. Watson et al mentions that the improved performance of the modified porous mixes have made it the surface mix of choice for GDOT for all interstates and state routes with high traffic volumes.

1.15.2 Benefits of Permeable Asphalt Mixtures

Watson et al mention that the primary benefits of porous asphalt mixes include reduction of hydroplaning and increasing driver safety through rapid removal of surface water from roadways during light to moderate rainstorms. They mention that draining of water through the pores (and not over the surface) results in increased coefficient of friction between tires and the pavement surface. They also mention that porous mixes reduce splash and spray, improve nighttime visibility, and improve the visibility of traffic striping.

1.15.3 Materials and Design

Watson et al provide descriptions of the older D type (original OGFC) mixes, the PEM as well as the hybrid OGFC mix that was developed by GDOT since 1992, to accommodate some of the good qualities of the PEM into the conventional OGFC. They mention that as a result of these changes, the permeability of the porous mixes has improved considerably and that the currently used OGFC (referred to as modified OGFC) is much coarser compared to the D mix, but slightly finer than the PEM.

Watson et al has presented the different changes that have been made and their basis, which has these have been summarized in Table 40 in this review.

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Table 40: Materials and Mix Design

<table>
<tr><th>Material/Design</th><th colspan="5">Changes</th></tr>
<tr><td>Aggregate properties</td><td colspan="5">No changes have been made.
Los Angeles abrasion (loss) < 50%, Soundness (loss) < 15%
Flat and elongated particles allowed (5:1 ratio) < 10%,
Mica schist allowed < 10% (Silica-rich aggregates only shall be used (e.g. granites). Carbonate-rich aggregates (e.g. limestones) are excluded.
Soundness loss is measured using magnesium sulfate (MgSO4).</td></tr>
<tr><td rowspan="10">Mix Characteristics</td><td rowspan="2">Sieve Size, mm</td><td colspan="3">Percent Passing</td><td>Tolerance</td></tr>
<tr><td>D mix (9.5 mm)</td><td>PEM</td><td>Modified OGFC (12.5 mm)</td><td></td></tr>
<tr><td>19 mm</td><td></td><td>100</td><td>100</td><td>±0.0 %</td></tr>
<tr><td>12.5 mm</td><td>100</td><td>90-100</td><td>85-100</td><td>±6.1 %</td></tr>
<tr><td>9.5 mm</td><td>85-100</td><td>35-60</td><td>55-75</td><td>±5.6 %</td></tr>
<tr><td>4.75 mm</td><td>20-40</td><td>10-25</td><td>15-25</td><td>±5.7 %</td></tr>
<tr><td>2.36 mm</td><td>5-10</td><td>5-10</td><td>5-10</td><td>±4.6 %</td></tr>
<tr><td>75 µm</td><td>2-4</td><td>1-4</td><td>2-4</td><td>±2.0 %</td></tr>
<tr><td colspan="5">Mix Design</td></tr>
<tr><td>Asphalt Content</td><td>6.0-7.25</td><td>5.5-7.0</td><td>5.75-7.25</td><td>±0.4</td></tr>
<tr><td>Asphalt Binder</td><td colspan="5">Hydrated lime is added as an antistripping agent.
GDOT has primarily used styrene butadiene (SB) and styrene butadiene styrene (SBS), to modify asphalt binders to increase binder stiffness to 8-10 times that of neat asphalt binder.

Empirical tests have been replaced with Superpave tests - a phase angle requirement of less than 75° has been added to help ensure that polymer modification is used to meet the binder grade requirements, and Superpave PG76-22 binder is now used.</td></tr>
<tr><td>Mineral Fiber</td><td colspan="5">Mineral fibers have been added to the materials to stabilize the asphalt film on aggregates, and hence reduce draindown.</td></tr>
<tr><td>Draindown Potential</td><td colspan="5">The draindown susceptibility of GDOT modified OGFC mixes can be determined using a test developed by the National Center for Asphalt Technology (NCAT). The NCAT procedure specifies that 0.3% is the maximum permissible draindown. Modified OGFC mixes which contain fibers and polymers have met this requirement.</td></tr>
</table>

In general, Watson et al mentions that the coarse gradation has improved permeability, and the use of a combination of polymer modification of asphalt binder and fibers as stabilizers has enabled GDOT to produce porous mixes with thicker asphalt films, and to use production temperatures higher than that used for conventional mixes.

1.15.4 Construction Practices

Watson et al mentions different modifications that have been made for production and paving of porous mixes as well as permeability tests carried out in the field for evaluation of the completed porous mix pavement. These modifications are summarized in Table 41.

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Table 41: Construction Practices

Step	Changes
Production	1. Fibers are introduced both in batch and drum mix plants via a separate fiber hopper and feeding system (instead of manual feeding). Fibers are fluffed and blown at a specified rate into the drum at drum mix plants and into the mixing chamber at batch plants. 2. Polymer modification of asphalt occurs at centralized terminal. This has resulted in greater quality control, and the capability of producing modified mixes with certified test results available prior to use. 3. The mixing temperature has been increased to 163°C to provide greater workability. This capability also helps in more effective removal of aggregate moisture, and hence improves adhesion of asphalt and aggregates, and improves durability. 4. Dry and wet mixing times are increased to prevent clumping of fibers.
Construction	The recommended spread rate has been increased to 49 kg/m ^{2} (90 lb./yd.^{2}) from 41 kg/m ^{2} (75 lbs./yd.^{2}) to increase the thickness of the courses and hence avoid streaking and resulting loss of smoothness. In addition to ensuring proper truck insulation and continuity of operation, material transfer devices are to be used to avoid the formation of cold lumps and resulting blemishes on the surface.
In-place Testing	GDOT performs permeability testing using a falling head permeameter. This device allows the user to determine a permeability coefficient, represented in meters (feet) per day, for the mix being tested. The apparatus consists of a circular base plate, a grease gun, and a cylindrical plastic water container. The base plate is fitted with two rubber O-rings mounted near the outer edge and placed 16 mm (¾”) apart. The base plate is placed in contact with the pavement surface, and grease is pumped between the O-rings, resulting in an impermeable seal. The plastic cylinder is then filled with water, which is allowed to flow through the base plate into the pavement below. By timing the flow of 1600 ml of water, a permeability coefficient can be calculated which is based on the known thickness of the pavement.

Watson et al indicates the benefits achieved through the modifications in mix design, production and construction process, by citing examples of improved smoothness and increased permeability. They mention that the smoothness levels on one project, consisting of over 136 lane-km (85 lane-miles) of modified OGFC overlay, averaged 143 mm/km (9 in./mi.). During fiscal year 1996, the average smoothness level statewide for modified OGFC was 158 mm/km (10 in./mi.), whereas dense-graded mixes averaged 397 mm/km (25 in./mi.). With respect to drainage capabilities, they indicate that modified OGFC typically drains 73 m/day (240 ft./day), significantly better than conventional OGFC (39 m/day, 130 ft./day).

Watson et al provides some cost comparison between modified and conventional OGFC. They state that the modified OGFC costs 34 percent more, because of additional materials needed in the mix and the equipment needed to introduce these materials in the plant, as well as increased production temperature and slower production rate. Using annualized cost in life cycle cost analysis, Watson et al shows that the modified OGFC would outweigh the conventional OGFC as a more desirable mix if it would last just 19 months longer. The authors have considered a life of 8 years for the conventional mix and 10-12 years for the modified mix.

1.15.5 Maintenance Practices

No information on maintenance practices of porous asphalt mixtures has been provided.

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1.15.6 Rehabilitation Practices
No information on rehabilitation of porous asphalt mixtures has been provided.

1.15.7 Performance
No information on performance of porous asphalt mixtures has been provided

1.15.8 Structural Design
No information on structural design of porous asphalt mixtures has been provided.

1.15.9 Limitations
No information on limitations of porous asphalt mixtures has been provided.

1.16 Choubane, B., J. A. Musselman, G. C. Page. "Forensic Investigation of Bleeding in Open-Graded Asphalt-Rubber Surface Mixes." TRB 1999 Annual Meeting CD-ROM, Transportation Research Board. National Research Council. Washington, D.C. 1999.

1.16.1 General
In this paper, Choubane et al provides a description of asphalt binder drainage or "bleeding" problems noted in open-graded friction courses (OGFCs) in Florida, possible reasons for the problems and a description of a detailed investigation of such a problem. The authors mention that the use of ground tire rubber has enabled Florida DOT (FDOT) to use more asphalt binder resulting in thicker asphalt binder films, and hence, improve their durability. However, they mention that "fat spots' resulting from asphalt binder bleeding has still remained a persistent problem.

Through a detailed investigation of production and laydown of an OGFC mix, the authors contend that part of the bleeding problem can be traced back to inconsistencies in binder contents in the mix during production, especially during start up in the plants, flushing of surface being overlaid with OGFC and build-up of tracked tacked material. The authors mention that the flushing and build-up of tacked materials go unnoticed during night paving.

Choubane et al concludes that fluctuations in binder content during start up could have caused bleeding problems and that lowering the binder content slightly below the optimum design content could help in avoiding significant problems with the mix. They contend that flushing of surface and build-up of tacked material could also lead to the bleeding problem. They recommend that standard start up procedures, as developed in this study, should be followed by the contractor and paving sites should be reviewed in day-light prior to night paving, in order to detect and remove any areas of excessive or tracked tack material. They also recommend that, to avoid design problems associated with OGFCs, dense-graded friction courses be used (in lieu of open-graded friction courses) wherever possible.

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1.16.2 Benefits of Permeable Asphalt Mixtures

In their discussion on the rationale for using OGFC mixes in Florida, Choubane et al mentions that the benefits of using OGFC include reduced hydroplaning potential through greater macrotexture and enhanced drainage of water from the tire/pavement interface resulting in improved tire contact with the pavement, and increased visibility through reduced splash and spray from tires.

1.16.3 Materials and Design

Choubane et al provides a description of the FC2 type OGFC mixes used in Florida. Table 42 summarizes the materials and mix design criteria.

Table 42: Materials and Mix Design Criteria for OGFC Mixes Used in Florida

Material/Mix	Type/Property
Aggregate	Crushed granite, slag, or oolitic limestone as the coarse aggregate component. A typical blend of materials for an FC-2 would include approximately 92% of a No. 89 Stone, and 8% sand or screenings.
Gradation	<u>Sieve Size</u> <u>Percent Passing</u> 12.5 mm 100 9.5 mm 85-100 4.75 mm 10-40 2.0 mm 4-12 75 µm 2-5
Asphalt Binder	AC-30 blended (wet process) with 12% GTR (by weight of asphalt cement
Binder Content	7.1% for mixes containing oolitic limestone, and 6.3% for granite and slag mixes

1.16.4 Construction Practices

In describing typical OGFC mixes used in Florida, Choubane et al mentions that these mixes are spread at a rate of 27 - 34 kg/m^2 , and then “seal rolled” with a tandem steel-wheel roller with a weight not exceeding 2.4 kg/mm of drum width. They also mention that there is no density requirement for OGFC mixes and the acceptance of open-graded friction courses at the plant is based on binder content and extracted gradation. The binder content acceptance is based on a meter/printer system, since the results of binder extraction could be affected by the presence of ground tire rubber (GTR) present in the mix.

1.16.5 Maintenance Practices

No information on maintenance practices of porous asphalt mixtures has been provided.

1.16.6 Rehabilitation Practices

No information on rehabilitation of porous asphalt mixtures has been provided.

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1.16.7 Performance

Choubane et al mentions several cases where bleeding has been observed, as well as a case where a thorough investigation was made to determine possible causes of bleeding. These projects and their performances/distresses, in terms of durability/bleeding are summarized in Table 43.

Table 43: Projects with Bleeding Problems

Project	Details	Performance/Distress
SR-64 Manatee County	Placed during the summer of 1995; The aggregate blend consisted of 88% oolitic limestone and 12% limestone screenings. The total binder content was 7.1%.	Following construction, flushing was observed at three distinct pavement locations, in less than 5% of the total length of the project. During production, the meter/printer system indicated the binder content was close to the target and gradations of the extracted aggregate also matched the mix design targets. A subsequent investigation indicated that the bleeding in the distressed areas resulted from a low binder viscosity caused by settlement of the GTR in the asphalt storage tank, which resulted in draindown during construction.
I-75, Pasco County	Placed during the summer of 1995. The aggregate blend consisted of 92% Nova Scotia granite and 8% sand, with a total binder content of 6.5%.	The project experienced flushing in several locations throughout its length. The cause of the bleeding was never clearly established. During production, the meter/printer system indicated the binder content was close to the target and gradations of the extracted aggregate also matched the mix design targets. The mix design was re-verified and the results indicated that the original binder content was appropriate. Furthermore, a similar mix design was placed on an I-75 project located in nearby Sumter County which did not experience any bleeding.
SR-54, Pasco County and SR-600, Hillsborough County	Placed during the spring of 1996. Aggregate blend consisted of 94% oolitic limestone and 6% sand, with a total binder content of 7.1%.	Both projects developed numerous areas of bleeding. A re-verification of the mix design showed that the optimum binder content of the mix had dropped from 7.1% when it was originally designed in 1994, down to 6.4%. The reduction in the binder content was related to a change in aggregate surface texture, apparently caused by a change in the crushing operation at the quarry. A review showed that out of 24 open-graded mixes utilizing aggregate from this source, the only two which experienced bleeding problems were the SR-54 and SR-600 projects.

In view of these projects (all in the Tampa area), Choubane et al mentions that FDOT decided to re-verify all OGFC mix designs to be used on projects in the Tampa area prior to their use, and issue these mix designs at a binder content 0.2% lower than optimum. These were done to prevent the plant produced OGFC from changing significantly from its mix design and hence to prevent resultant durability problem. However, the authors mention that in the summer of 1997, another project in the Tampa area developed intermittent bleeding project, which prompted an investigation. The details of this project, the investigation and follow-up work are summarized in Table 44.

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Table 44: Investigation and Follow-Up Work

Mix Design/Construction	Distress/Cause	Follow-up work
Aggregate blend: 92% Nova Scotia granite and 8% sand. The optimum binder content of the mix was 6.0%. The mix was placed on this project during the early summer of 1997.	Within four to six weeks after construction, several areas of the project appeared to have excessive asphalt that had flushed to the surface. There were approximately 5 locations within the project that had bleeding ranging in length from 3 to 30 m (10 to 100 feet). In most instances, the bleeding occurred within the wheel-paths; the bleeding seemed to be more severe at a major intersection within the project. The intermittent occurrence of the bleeding throughout the project seemed to indicate that the problem was not exclusively related to the mix design (confirmed with reverification of mix design).	1. FDOT redesigned the open-graded friction course mix, substituting limestone screenings for the sand. It was believed that the screenings would increase the overall absorption of the mix, thus minimizing the potential for draindown. The total binder content was 5.9% (which was approximately 0.5% below the optimum determined by the open-graded friction course design procedure.) 2. The production of the open-graded friction course mix was monitored at the contractor's plant. The asphalt content for each sample was determined using the ignition method 3. Core samples were tested for binder content, gradation, and thickness.

Choubane et al indicates the following observations from the follow-up work:
1. Binder content from first night's production showed excessive fluctuations in binder contents (7.09, 5.94, & 6.82 with a target of 5.90). The binder content data obtained for the second night was more consistent, with only one relatively high test result.
2. Test results on cores indicated constant binder contents at most areas but significantly higher binder contents at areas showing bleeding.
3. The course was placed very thin, 6-10 mm, and hence would have the potential of exhibiting more bleeding than usual, because of draindown or build-up of tacked materials.

Subsequently, Choubane et al mentions, FDOT monitored another OGFC paving project, and determined that materials and mix design parameters closely met target values. However, they did note that there was flushing on the surface which was being overlaid and also build-up of tacked material. Choubane et al indicates that concurrent to FDOT's monitoring operations, the contractor implemented an enhanced method of production of OGFC mixes. This method consists of: allowing the drum mixer to run empty (clean-out) for 5 minutes; tighter documentation on material usage; closer visual observation of the mix during production; constant/maximum production rates for each plant; and testing the initial mix as it is produced.

In addition, Choubane et al indicates that sampling and monitoring (and testing of GTR and AC 30) were also conducted during the GTR blending operation in the asphalt rubber terminal in Tampa. Testing of materials was conducted at different stages – start from the

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terminal, arrival at the contractor's plant and prior to use. The results indicated that the materials met all specifications.

1.16.8 Structural Design
No information on structural design of porous asphalt mixtures has been provided.

1.16.9 Limitations
No information on limitations of porous asphalt mixtures has been provided.

1.17 Rogge, D. and E.A. Hunt. "Development of Maintenance Practices for Oregon F-Mix – Interim Report SPR371." Oregon Department of Transportation. Salem, OR. August 1999.

1.17.1 General
This paper outlines Oregon's experience with a 19mm open mix known as the F-mix which they began using in the 1970's. Even at the time of this report in 1997, little experience and knowledge was available on the correct maintenance and rehabilitation procedures for the F-mix. Thus, the research presented in this work was aimed at determining the current state of practice and future direction for the maintenance and rehabilitation of the F-mix.

The research approach by Rogge et al involved surveying ODOT maintenance personnel on their thoughts and perceptions regarding F-mix maintenance, conducting a literature review regarding others experience with porous (open) asphalt, and research into new maintenance materials and techniques.

The overall response to the survey conducted in this research was low, with only 25 surveys returned. From these results the most common distresses in the F-mix were: clogging, icing problems, fat spots/bleeding, and cracking (alligator). Other distresses noted were deformation rutting, tire stud rutting, and gouging/scarring (snow plow etc.). The respondents were also asked to identify the maintenance techniques they had employed on the F-mix. Blade patching with a dense-graded hot mix was the most widely used technique followed closely by mill, inlay and screed patch with dense-graded hot mix. Only 8 of the 25 respondents said that they had used a fog seal for the F-mix. Most expressed concerns over fog sealing clogging the porous pavement, reducing skid resistance, and disrupting traffic operations. Finally the survey addressed the perceptions that the maintenance personnel had on the F-mix as compared with a conventional dense mix. Rogge et al state "Clearly the perception of maintenance supervisors is that F-mix requires more maintenance than dense-graded asphalt pavements, and that the repairs for F-mix are more expensive than for dense-graded pavements."

Through the literature review conducted by Rogge et al identified the maintenance challenges of the F-mix. Most significant are clogging, permeability, winter maintenance, and physical/mechanical distress.

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Rogge et al concluded by discussing new techniques in maintenance for the F-mix. Furthermore they state: a clogged porous pavement still drains better than a conventional mix, conventional repair techniques may be used on limited areas of F-mix provided drainage is not impeded, the greatest hindrance to F-mix repair is lack of F-mix material in small quantities, and optimum maintenance procedures for porous pavements have not yet been identified.

1.17.2 Benefits of Permeable Asphalt Mixtures
Rogge et al did not discuss benefits of permeable asphalt mixtures.

1.17.3 Materials and Design
Rogge et al did not discuss materials and design.

1.17.4 Construction Practices
Rogge et al state that the F-mix is placed 50 mm thick.

1.17.5 Maintenance Practices
Rogge et al state that use of traditional dense-graded mixes to repair the F-mix "…destroys the free draining characteristics of the F-mix, changes noise and ride characteristics, and increases the possibility of rutting problems." Many issues have been raised over the correct emergency maintenance procedures as there are only procedures written for conventional dense-graded mixes.

To mitigate clogging, Rogge et al discuss efforts undertaken in Europe to use a suction sweeper to clean the porous pavement, although they conclude that these efforts have not been promising.

In regards to winter maintenance, Rogge et al state that a de-icing agent is needed that can stay on the surface of the porous pavement rather than slipping into the voids. Future possibilities include mixing calcium magnesium acetate with another material that will stick to the surface.

Rogge et al were unable to find any evidence to endorse or condemn the use of fog seals as a preventive maintenance practice for the F-mix.

1.17.6 Rehabilitation Practices
Rogge et al state that there are three methods for porous asphalt rehabilitation: mill and inlay, in-place recycling or repaving, and overlays.

1.17.7 Performance
Rogge et al state that field performance of the F-mix has been excellent; however they have many concerns over the proper maintenance and rehabilitation of these mixes as they become older. Furthermore Rogge et al state that, "Dissatisfaction with maintenance procedures could jeopardize ODOT's continued use of a successful paving procedure (F-mix)."

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1.17.8 Structural Design

Rogge et al did not discuss structural design.

1.17.9 Limitations

Clogging is of concern mainly in the shoulder areas of the roadway because debris collects there. Other authors have suggested placing an impermeable surface dressing to mitigate these problems. Also Rogge et al note that clogging occurs, "…but clogged pavement still allows drainage through the pavement, whereas dense-graded pavements do not."

Winter maintenance issues with porous asphalt are also of great concern, especially winter icing. Winter maintenance is different for porous pavements because of the "…different temperature behavior for porous asphalt, and because of difficulty maintaining a sufficient salt level at the level of contact between tire and pavement." Other authors have concluded that ice forms quicker on porous asphalt than a dense mix.

The Rogge et al research also concluded that the main physical/mechanical distress in porous asphalt is raveling or particle loss. The problem results from cold mix, low compaction, or segregation from the binder.

1.18 "Noise-Reducing Pavements for Urban Roads." Danish Road Directorate (DRD). Nordic Road & Transport Research. Volume No. 3. 1999.

1.18.1 General

This article describes a research project to investigate the use of two-layer porous asphalt for reducing noise. The Danish government has an initiative to reduce the number of dwellings exposed to a noise level of 65 dB(A) by two-thirds (approximately 100,000 dwellings) by 2010. Based upon research in the Netherlands, the Danish have identified the two-layer porous asphalt as potentially the most effective means of achieving this goal.

A two-layer porous asphalt pavement system is composed of a bottom layer of porous asphalt with a large aggregate gradation size and a top layer with a small aggregate gradations size (Figure 11). Both layers contain between 20 and 25 percent air voids. The top layer of porous asphalt has a small aggregate size gradation which keeps dirt and debris from clogging the lower layer. The bottom layer of porous asphalt utilizes a larger aggregate size gradation that produces large size air voids and is used to remove any dirt and debris that penetrates through the upper layer.

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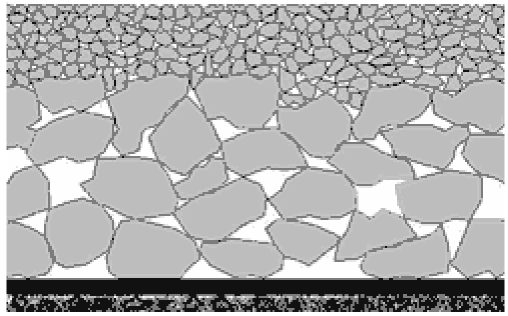

Figure 11: Principle of Two Layer Porous Asphalt Pavements

According to Dutch experience, two-layer porous asphalt has good noise-reducing characteristics compared to dense-graded pavements. The reason for this is the structure of the porous asphalt layer, which contains a large number of interconnected voids. Tires rolling on the road result in air pumping since in front of the tire air is forced away and behind the tire the air is sucked out of the pavement. This pumping action generates a high-frequency noise. On porous asphalt, the pumping, and therefore the noise generated, is reduced because the air is instead pumped down into the interconnected voids of the porous layer.

Porous asphalt also reduces noise by absorbing some of the noise emitted by vehicles. On roads with dense pavements, the noise emitted towards the pavement is reflected to the surroundings; however, on porous asphalt some of this noise is absorbed by the pavement through the interconnected void structure. The article states that Dutch experience shows noise absorption depends on the thickness of the porous asphalt layer. The thicker the layer of porous asphalt, the lower is the frequency at which the maximum absorption occurs.

1.18.2 Benefits of Permeable Asphalt Mixtures

Benefits listed in the article included reduced potential for hydroplaning, reduced splash and spray, improved visibility of pavement markings and reduced tire/pavement noise levels.

1.18.3 Materials and Mix Design

The only information related to materials and mix design included in the article was the aggregate sizes of the upper and lower layers of porous asphalt. Aggregate gradation sizes of 2 to 5 mm and 5 to 8 mm have been used for the top layer. For the bottom layers, sizes of 11 to 16 mm and 16 to 22 mm have been used.

1.18.4 Construction Practices

Porous asphalt should be placed over an impermeable layer so that water cannot penetrate into the underlying pavement layer. On roadways with curb and gutter, it is necessary to construct drainage along the side of the road, so that water from the pavement can be led away into the stormwater collection system.

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1.18.5 Maintenance Practices
The article states that more salt is needed for winter maintenance because of the porous nature of the layer. The open nature of the layer also causes the layer to cool quicker than typical dense-graded layers and, therefore, it is necessary to initiate winter maintenance earlier.

1.18.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.18.7 Performance
No specific performance measures were given.

1.18.8 Structural Design
No specifics on inclusion within structural design were given.

1.18.9 Limitations
No specific limitations were given.

1.19 Backstrom, M. "Ground Temperature in Porous Pavement During Freezing and Thawing." *Journal of Transportation Engineering*. American Society of Civil Engineers. Reston, VA. Volume 126, Issue 5, September 2000, pp.375-381.

1.19.1 General
This paper addresses a full scale study of porous pavements during periods of freezing and snowmelt. The objective behind the study was to determine if porous pavements were suitable for storm water management in cold regions.

A field site was setup in northern Sweden. The specific existing pavement area had frost damage and was noted to accumulate water (puddle) after snowmelts. The exiting pavement was removed and 45mm thick porous asphalt (5% asphalt content) was placed on one section. On another section a 45mm thick impermeable base was placed (6% asphalt content). These two sections were used for comparative purposes.

During the study the following items were monitored for each pavement section: climatic conditions (ambient temperature and precipitation); depth of ground water table below pavement surface; ground temperature; temperature response in each pavement; frost penetration depth; and frost heave.

Some highlights of Backstrom's findings were: two major factors that influence freezing in porous pavements (variations in air temperature and ground heat flux from the soil below), frost penetration in the soil below the subgrade was related to the number of negative degree days, tests showed that even a clogged porous pavement retained sufficient drainage capacity to handle snowmelt, and thawing of porous pavement is a function of air temperature, ground heat flux, increased net radiation and energy content of the infiltrating meltwater.

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Overall Backstrom concluded that, "…porous pavements are more resistant to freezing than an impermeable pavement due to higher water content in the soil, which increased the latent heat in the ground. Cooling of the porous pavement is governed by variations in ambient air temperature, and freezing of the soil below the subgrade is related to the freezing index. Thawing of the porous pavement is a quick process, which was explained by the meltwater infiltration. The thawing process in a comparable impermeable pavement is slower. The frost penetration depth is decreased, and the frost period is shorter in a porous pavement compared with an impermeable pavement. Consequently, there is a lower risk for frost heave damage on porous pavement rods than on conventional roads."

1.19.2 Benefits of Permeable Asphalt Mixtures

Backstrom states that porous asphalt provides benefits such as, "…storm-water flow attenuation, aquifer recharge, and storm-water pollution control."

1.19.3 Materials and Design

Backstrom did not discuss materials and design.

1.19.4 Construction Practices

Backstrom did not discuss construction practices.

1.19.5 Maintenance Practices

Backstrom did not discuss maintenance practices.

1.19.6 Rehabilitation Practices

Backstrom did not discuss rehabilitation practices.

1.19.7 Performance

Backstrom did not discuss performance of permeable asphalt mixes.

1.19.8 Structural Design

Backstrom did not discuss structural design.

1.19.9 Limitations

Backstrom did not discuss limitations.

1.20 Cooley, L. Allen, Jr., E. R. Brown, and D. E. Watson. "Evaluation of OGFC Mixtures Containing Cellulose Fibers." Transportation Research Record No: 1723. Transportation Research Board. National Research Council. Washington, D.C. 2000.

1.20.1 General

In this paper Cooley et al provide a description of a field and a laboratory study carried out to compare the performance of OGFC mixes with different types of fibers. The primary objective was to evaluate the use of cellulose fibers as stabilizers in OGFC mixes. To achieve this objective, the researchers looked at field sections containing OGFC mixes with cellulose and mineral fibers and tested materials and mixes prepared in the laboratory.

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Cooley et al mention that fibers are used for reducing draindown in OGFC mixes, which results from the open gradation. This study was prompted by some concern that cellulose fibers would absorb water and lead to moisture damage in mixes. Cooley et al selected several field sites containing OGFC with a variety of asphalt binders and fibers, on an interstate in Atlanta, Georgia, and evaluated distresses in these sections. They also prepared OGFC mixes in the laboratory with different types of fibers and conducted tests to determine absorption characteristics of OGFC mixes with fibers and mechanical properties of the mixes.

Based on both field and laboratory studies, Cooley et al concludes that OGFC mixes with cellulose fibers would perform as well as OGFC mixes with mineral fiber.

1.20.2 Benefits of Permeable Asphalt Mixtures

In discussing the need for fibers in OGFC mixes in their introduction, Cooley et al mention the following benefits of OGFC: 1) Improved surface frictional resistance; 2) Minimized hydroplaning potential; 3) Reduced splash and spray; 4) Improved night visibility; and 5) Lower pavement noise levels.

1.20.3 Materials and Design

Cooley et al provides result of both a field survey as well as laboratory testing of several OGFC mixes. The materials and design of the laboratory mixes are summarized in Table 45. Table 46 provides a description of test results from the laboratory mixes. (The materials of the in-place mixes and the results of survey conducted on the in-place mixes are provided in the "Performance" section of this review).

Four different mixes were designed, corresponding to four different fibers. The mix design samples were compacted with 25 blows per face with a Marshall hammer at varying asphalt contents. Optimum asphalt content was selected as the one producing the lowest voids in mineral aggregate (VMA).

Table 45: Materials and Mix Design for Laboratory Mixes

Material	Number - Type
Aggregate	One – Granite; One percent lime by total aggregate mass was used.
Gradation	One - Georgia DOT, 12.5 mm OGFC mix
Asphalt Binder	One - PG 76-22 modified with styrene butadiene styrene (SBS) polymer.
Fiber	Four – three cellulose (loose fiber, a 66/34 pelletized fiber (66 percent cellulose fiber and 34 percent asphalt), and an 80/20 pelletized fiber) and one mineral (slag wool)

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Table 46: Tests and Results

Property	Test	Results
Absorption	This test was conducted by allowing compacted OGFC mixtures to soak in a 60°C water bath for 72 hours. After soaking, the specimens were allowed to dry at room temperature. Mass measurements were obtained at 1, 2, 4, 21, 24, 48, and 72 hours to determine mass loss. Three replicates of each mixture were tested. The amount of water in a sample was determined by subtracting the mass of a sample prior to any conditioning from the mass of the same sample after conditioning and drying at room temperature for the various times. The percent water at any time was then calculated as the amount of water in the sample at that time divided by the original mass of that sample and expressing as a percentage.	All four mixtures had approximately the same rate of water loss.
Moisture sensitivity	GDT-66, "Method of Test for Evaluating the Moisture Susceptibility of Bituminous Mixtures by Diametral Tensile Splitting." (similar to modified Lottman procedure). The four mixtures were evaluated after 1, 3, and 6 freeze-thaw cycles.	Four mixes performed similarly.
Moisture sensitivity	GDOT-56, "Test Method for Heat Stable Anti-Strip Additive." Loose OGFC mixture was placed into boiling water for ten minutes. A visual inspection was then performed to determine the approximate percentage of aggregate particles in which the asphalt binder was totally or partially removed.	No visual stripping in any of the four mixtures.
Rutting/ Moisture sensitivity	GDT-115, "Method of Test for Determining Rutting Susceptibility Using the Loaded Wheel Tester," while submerged in water at 60°C. This testing was conducted for only the loose cellulose and mineral fiber mixtures at optimum asphalt content. Prior to testing, samples were conditioned in a 60°C water bath overnight.	Loose cellulose mixture had a lower rut depth than did the mineral fiber mix.

1.20.4 Construction Practices
No information on construction practices of porous asphalt mixtures has been provided

1.20.5 Maintenance Practices
No information on maintenance practices of porous asphalt mixtures has been provided.

1.20.6 Rehabilitation Practices
No information on rehabilitation of porous asphalt mixtures has been provided.

1.20.7 Performance
Cooley et al indicates that performances of six sections with different OGFC mixes were evaluated. These sections, types of mixes and the distress surveyed are shown in Table 47. The results of visual survey are shown in Table 48.

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Table 47: In-Place Mixes

Property	Types/Method
Section/Route	Six experimental OGFC pavement sections located on Interstate 75 south of Atlanta, Georgia; constructed in 1992
Types of mixes	Coarse OGFC, Coarse OGFC with 16% crumb rubber, Coarse OGFC with mineral fibers, Coarse OGFC with cellulose fibers Coarse OGFC with styrene-butadiene (SB) polymer, Coarse OGFC with SB and cellulose fibers
Survey	Surface texture, rutting, cracking, and raveling.
Permeability Testing Conducted on Cores	Three 150 mm cores from each section were tested in the laboratory to determine permeability with a falling head permeameter.

Cooley et al concluded that the field as well as the laboratory data shows that OGFC mixes containing cellulose fiber performed as well as mixes with mineral fiber.

Table 48: Results of Visual Survey

Property	Results
Surface texture	All six showed some coarse aggregate pop-out. The Coarse OGFC with 16% crumb rubber section had the most coarse aggregate pop-out while the Coarse OGFC with cellulose fibers, Coarse OGFC with mineral fibers, and Coarse OGFC with styrene-butadiene (SB) polymer sections appeared to have the lowest amount. All sections showed fat spots, ranging in diameter from approximately 8 cm (3 in) to 20 cm (8 in), but insignificant in extent. The Coarse OGFC and Coarse OGFC with SB and cellulose fibers section had the most fat spots.
Rutting	Rut depth measurements were made in each experimental section with a stringline. The amount of rutting in all sections was insignificant.
Cracking	All six sections exhibited reflective cracking from underlying Portland cement concrete pavement. Percentage of reflective cracks was determined by counting the number of transverse reflective cracks visible at the pavement surface. Reflective longitudinal cracks were observed on all sections, except the one with Coarse OGFC with mineral fibers. Longitudinal reflective cracking was very low severity in the Coarse OGFC with cellulose fibers and Coarse OGFC with styrene-butadiene (SB) polymer sections. For the Coarse OGFC and Coarse OGFC with 16% crumb rubber sections some of the longitudinal cracks had opened. Only the Coarse OGFC with 16% crumb rubber section showed any other type of cracking, besides reflective, with secondary cracks near the reflective cracking.
Raveling	Raveling was minimal except for the Coarse OGFC with 16% crumb rubber section, which showed some medium severity raveling along cracks.
Permeabil-ity	Statistically, no significant differences were found between permeability values of the cores from the six sections. The Coarse OGFC with cellulose fibers and Coarse OGFC with SB and cellulose fibers sections showed the highest mean permeability values and the Coarse OGFC with styrene-butadiene (SB) polymer showed the least.

1.20.8 Structural Design
No information on structural design of porous asphalt mixtures has been provided.

1.20.9 Limitations
No information on limitations of porous asphalt mixtures has been provided.

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1.21 Huber, G., "Performance Survey on Open-Graded Friction Course Mixes." Synthesis of Highway Practice 284. National Cooperative Highway Research Program. Transportation Research Board. National Research Council. Washington, D.C. 2000.

1.21.1 General

This synthesis documents the recent [as of 2000] performance of OGFC in both Europe and the US. Specific chapters within the synthesis deal with the use of OGFC; design and construction practices; and performance, maintenance and rehabilitation practices from around the world.

Huber indicated that OGFC was developed in the U.S. during the 1940's, when the California Department of Highways experimented with plant mix seal coats due to problems encountered with chip seals. Plant mix seal coat pavement sections tended to have aggregates tightly bonded to the road surface eliminating windshield damage; fewer climatic problems during construction (especially after a rain); and more construction related advantages due to being able to place plant mix seal coats with a paver and being able to place the mix at relatively thin lifts (15 to 20 mm). These plant mix seal coats became known as OGFCs.

1.21.2 Benefits of Permeable Asphalt Mixtures

Huber states that the primary benefits realized with OGFCs are increased permeability and noise reduction. Permeability leads to improved wet weather frictional properties, improved wet weather visibility, and less splash and spray. Huber describes that water sits on the road surface during a rain event where it can be splashed or thrown into the air in the form of a mist when vehicle tires pass over the pooled water. Traffic mist can reduce visibility more severely than fog because the airborne particles within the mist are larger than the particles within fog. Porous pavements reduce (and almost alleviate) the droplets of water caused by vehicles passing over the roadway because the water infiltrates into the interconnected voids of the pavement layer.

Noise reduction is also provided due to the porosity of OGFC pavements. Huber reports that noise reductions at highway speeds (comparing OGFC to typical dense-graded mixes) are on the order to 3.0 dB(A), which is a 50 percent reduction in noise pressure. A roadway test in Denmark showed that even after 5 years porous asphalt mixtures are 2 to 4 dB(A) quieter than typical dense-graded mixes.

Huber states that in most instances, OGFC pavements are more desirable on high speed roadways. High speeds are needed in order to generate enough hydraulic action under vehicle tires to allow the pavement to be self-cleaning and, therefore, maintain porosity.

The synthesis included a survey of all 50 US state highway departments and 10 provinces from Canada. As part of the survey, Huber requested each agency to identify the number one benefit sought with the use of OGFC. Figure 12 illustrates the results of this survey. This figure indicates that the number one benefit perceived by the survey respondents was improved frictional resistance. The second most perceived benefit was improved driver visibility during wet weather.

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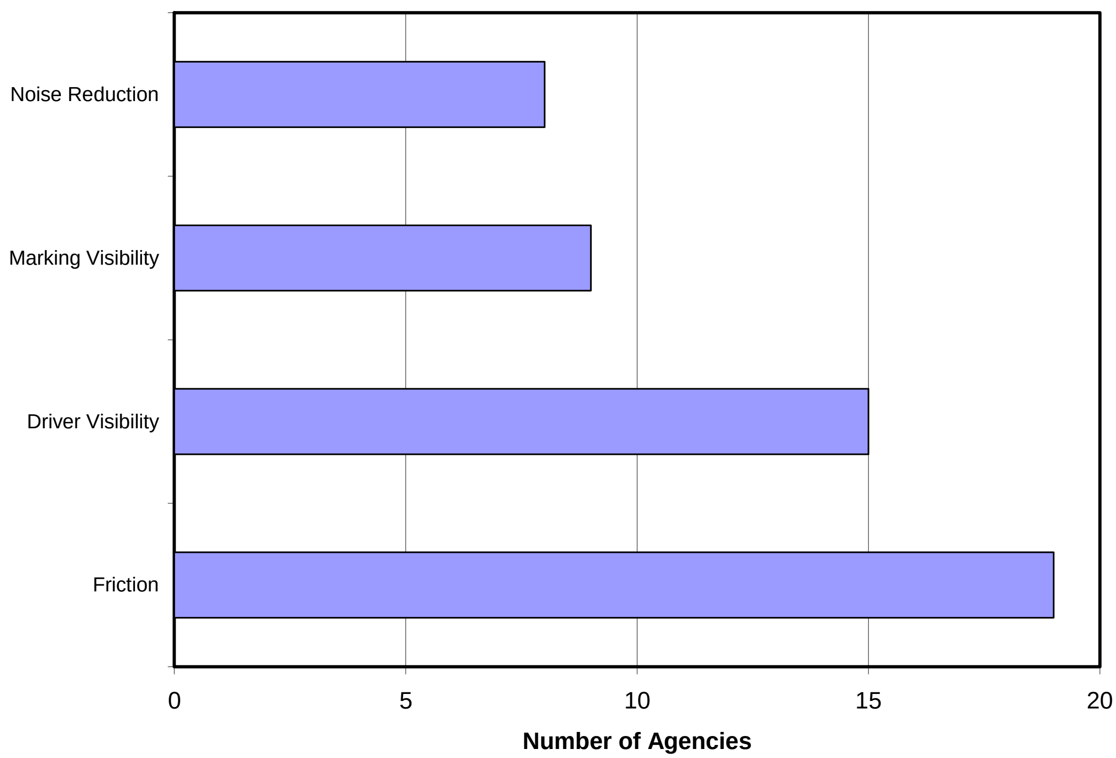

Figure 12: Benefits of Open-Graded Mixes Cited by Agencies

1.21.3 Materials and Mix Design

Huber described the 2000 standard mix design method of OGFCs. The standard, at that time, was based upon the FHWA Technical Advisory T5040.31. Material requirements contained within this Technical Advisory are summarized as follows: 1) polish resistant aggregate with 75 percent crushed two faces and 90 percent one face; 2) mineral filler meeting AASHTO M17; 3) aggregate gradation meeting Table 49; 4) AC-20 graded asphalt binder; and 5) antistrip additives.

Table 49: FHWA Design Gradation Band

	Percent Passing	
Sieve, mm	Minimum	Maximum
12.5	100	---
9.5	95	100
4.75	30	50
2.36	5	15
0.075	2	5

Huber summarized the design steps contained within the Technical Advisory as follows: 1) determine percentage of S.A.E. No 10 oil retained on the aggregates after soaking and draining; 2) estimate asphalt binder content based on the percentage of oil retained; 3) determine air void content in coarse aggregate by compacting dry coarse aggregate using

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a vibratory hammer; 4) calculate amount of fine aggregate; 5) test for draindown; and 6) evaluate moisture susceptibility of the designed mix.

Huber also described some recent advancements in OGFC mix design technology. Specifically, the inclusion of polymer modified binders and fibers to reduce the potential of draindown was incorporated into the materials selection process. The combination of fibers and modified binders provided the extra benefit of allowing higher production temperatures. Higher production temperatures aided in more effectively removing moisture from aggregates, thereby, reducing the potential for moisture damage. Another advancement mentioned by Huber was the increasing of air voids in the designed mixtures. Increased air void contents provided self-cleaning due to the hydraulic action of traffic. Huber also noted that some agencies had increased the nominal maximum aggregate size from 9.5 mm to 12.5 mm and 19mm. Tables 50 and 51 present a summary of mix design criteria used within the U.S. at the time this synthesis was published.

Huber also describes mix design methods from several European countries and South Africa. Table 52 summaries various porous asphalt mix design requirements from other parts of the world. Of particular interest in Table 52 is that all of these mix design criteria contain a minimum air void requirement. All six sets of criteria include a minimum air void content of 18 percent of more. In contrast, none of the US specifications had minimum air void contents.

An interesting mix design concept is utilized by Spain. Minimum asphalt binder content is selected based upon a durability test called the Cantabro Abrasion test. The Cantabro Abrasion test measures disintegration of compacted samples using the Los Angeles Abrasion drum. Samples are placed in the drum without steel spheres and subjected to 300 revolutions at room temperature. After the 300 revolutions, the percentage of mass loss is then determined.

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Table 50: Summary of 9.5mm OGFC Mixture Designs used in United States

	Arizona Unmodified		Arizona Rubber		California		Florida		Nevada		Nevada		Wyoming		New Mexico		Georgia	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Gradation (mm)																		
12.5	100		100		100		100		100		100		100		100		100	
9.5	100		100		90	100	85	100	95	100	90	100	97	100	90	100	85	100
4.75	35	55	30	45	29	36	10	40	40	65	35	55	25	45	25	55	20	40
2.36	9	14	4	8	7	18	--	--	--	--	--	--	10	25	--	--	5	10
2.00	--	--	--	--	--	--	4	12	--	--	--	--	--	--	0	12	--	--
1.18	--	--	--	--	--	--	--	--	12	22	5	18	--	--	--	--	--	--
0.075	0	2.5	0	2.5	0	3	2	5	0	4	0	3	2	7	0	4	2	4
Binder Grade	PG 64-16		PG 64-16 + 20% Rubber		AR4000, AR8,000 or PDA-6		AC30 + 12% Rubber		AC20P or AC30		AC20P or AC30		PG 64-22 or PG 70-28		PAC20		PG 67-22	
Binder Content	--		--		--		5.5 – 7.0		6.5 typical		6.5 typical		6.3 – 6.8		--		6.0 – 7.3	
Aggregate Properties																		
Specific Gravity	--		2.35 - 2.85		--		--		--		--		--		--		--	
Water Abs.	--		2.5 max.		--		--		4 max.		4 max.		--		--		--	
Sand Equiv.	45 min.		55 min.		--		--		--		--		--		--		--	
Crushed Faces	70 min.		95 min.		90 min.		100 min.		90 min.		90 min.		95 min.		75% min.		--	
Flakiness Index	25 max.		--		--		--		--		--		--		--		--	
L.A. Abrasion	--		40 max.		40 max.		--		37 max.		37 max.		35 max.		40 max.		--	
Carbonates	--		30 max.		--		88 max		--		--		--		--		--	
Mg Soundness	--		--		--		--		12 max.		12 max.		--		12 max.		--	
Compaction Method	Dbl. Plunger		Dbl. Plunger		--		--		Static Compression		Static Compression		--		--		Marshall	
Air Voids	--		--		--		--		--		--		--		--		--	

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Table 51: Summary of 12.5mm and 19.0mm OGFC Mixture Designs used in the United States

	California		Georgia OGFC		Georgia PEM		Oregon		Oregon	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Gradation (mm)										
12.5	100		100		100		100		100	
9.5	100		100		90	100	85	100	95	100
4.75	35	55	30	45	29	36	10	40	40	65
2.36	9	14	4	8	7	18	--	--	--	--
2.00	--	--	--	--	--	--	4	12	--	--
1.18	--	--	--	--	--	--	--	--	12	22
0.075	0	2.5	0	2.5	0	3	2	5	0	4
Binder Grade	PG 64-16		PG 64-16 + 20% Rubber		AR4000, AR8,000 or PDA-6		AC30 + 12% Rubber		AC20P or AC30	
Binder Content	--		--		--		5.5 – 7.0		6.5 typical	
Aggregate Properties										
Specific Gravity	--		2.35 - 2.85		--		--		--	
Water Abs.	--		2.5 max.		--		--		4 max.	
Sand Equiv.	45 min.		55 min.		--		--		--	
Crushed Faces	70 min.		95 min.		90 min.		100 min.		90 min.	
Flakiness Index	25 max.		--		--		--		--	
L.A. Abrasion	--		40 max.		40 max.		--		37 max.	
Carbonates	--		30 max.		--		88 max		--	
Mg Soundness	--		--		--		--		12 max.	
Comp. Method	Dbl. Plunger		Dbl. Plunger		--		--		Static Compression	
Air Voids	--		--		--		--		--	

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Table 52: Summary of Non-North American Porous Asphalt Mixtures

	British (20 mm)		British (10mm)		Spanish (P-12)		Spanish (PA-12)		Italian		South Africa	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Gradation (mm)												
28	100	---	---	---	---	---	---	---	---	---	---	---
20	95	100	---	---	100	---	100	---	100	---	---	---
19	---	---	---	---	---	---	---	---	---	---	100	---
14	55	80	100	---	---	---	---	---	75-	100	---	---
13.2	---	---	---	---	---	---	---	---	---	---	90	100
12.5	---	---	---	---	75	100	70	100	---	---	---	---
10.0	---	---	90	100	60	90	50	80	15	40	---	---
9.5	---	---	---	---	---	---	---	---	---	---	25	65
6.3	20	30	40	55	---	---	---	---	---	---	---	---
5.00	---	---	---	---	32	50	15	30	5	20	---	---
4.75	---	---	---	---	---	---	---	---	---	---	10	15
3.35	8	14	22	30	---	---	---	---	---	---	---	---
2.50	---	---	---	---	10	18	10	22	---	---	---	---
2.36	---	---	---	---	---	---	---	---	---	---	8	15
2.00	---	---	---	---	---	---	---	---	0	12	---	---
0.63	---	---	---	---	6	12	6	13	---	---	---	---
0.008	---	---	---	---	3	6	3	6	---	---	---	---
0.075	2	7	2	7	---	---	---	---	0	7	2	8
Asphalt Binder Grade	100 pen. + SBS 100 pen. + EVA 100 pen.		100 pen. +SBS 100 pen. + EVA 100 pen.		60/70 + SBS 60/70 + EVA 80/100 + SBS 80/100 + EVA 4.5% typical		60/70 + SBS 60/70 + EVA 80/100 + SBS 80/100 + EVA 4.5% typical		80/100 + SBS		Asphalt rubber Polymer modified	
Content	4.5% min.		4.5% min.		4.5% typical		4.5% typical		4-6%		4.5% min.	
Aggregate Properties									16% max.		21% max.	
L.A. abrasion									---		25 max.	
Flakiness index	12% max.		12% max.		20% max.		20% max.		---		45 min.	
Sand equivalent	25 max.		25 max.		25 max.		25 max.		---		100% (high traffic)	
Crushed faces	---		---		---		---				90% (low traffic)	
(2 faces)	100%		100%		100%		100%					
Mixtures Properties	50 blow Marshall		50 blow Marshall		50 blow Marshall		50 blow Marshall		Marshall		50 blow Marshall	
Compaction	20% min.		20% min.		20% min.		20% min.		18-23%		>22% high volume	
Air voids											18-22% low volume	
Cantabro dry	---		---		25˚C/25% max.		25˚C/25% max.		25˚C/25% max.		25˚C/25% max.	
Cantabro aged	---		---		---		---		---		25˚C/30% max	
Cantabro wet	---		---		---		---		20˚C/30% max.		25˚C/30% max	

Note: L.A. = Los Angeles; SBS= styrene-butadiene-styrene; EVA= ethylene-vinyl acetate; pen.=penetration.

1.21.4 Construction Practices

Huber described common practices for the production and laydown/compaction of OGFCs. Huber indicates that no specific plant modifications are required to produce OGFCs. However, when modified asphalt modifiers are used, mechanical agitators may be required in the asphalt storage tank. When fibers are included, the plant must be equipped with a metering system to feed the fibers into the production process. Also, the system must be able to evenly distribute the fibers. Nonuniform fiber distribution can result in portions of the mixture being dry and unworkable while other portions of the mix can be soft and susceptible to draindown problems.

Once the mixture's produced, storage time should be minimized as long storage times in a heated silo can result in an increased potential for draindown. Huber reports that agencies require maximum storage times from 1 to 12 hours.

Transportation of the produced mix to the construction site can be conducted in the same trucks in which dense-graded mixes are transported. Release agents can be used to prevent the mixture from sticking to the truck bed. Tarps should also be used to maintain mix temperature during transportation and to prevent crushing of the mix.

Once at the construction site, the OGFC can be delivered to the paving train in much the same manner as dense-graded mixtures. As with any HMA, good construction practices are required. When windrows are used, the length of the windrows should be carefully controlled so that the mixture will not lose temperature. Huber indicates that within favorable climatic conditions, windrows should not be over 50m (150 ft).

Huber indicates that roll down may be less for OGFC mixes than for dense-graded HMA. Also, when an extendable screed is used, auger extensions should be included. Huber also states that typical rolling patterns should be used for compaction.

Transverse joints are more difficult to construct with OGFC mixes than for dense-graded HMA. Handwork tends to be difficult. Construction of longitudinal joints according to Huber, are similar to dense-graded pavements. Huber states that tacking the vertical face of longitudinal joints can be done but care should be taken not to apply excess tack coat. Excessive tack coats on the longitudinal joint can limit drainage across the joint.

Huber recommends the use of static steel wheel rollers for compacting OGFC layers. Rollers should be 10 Mg (11 tons) or smaller to prevent excessive aggregate breakdown.

1.21.5 Maintenance Practices

Huber defines two categories of maintenance for OGFCs: surface maintenance and winter maintenance. Winter maintenance, as the title indicates, deals with the activities required to maintain the driving surface during cold and inclement weather. Huber indicates that OGFC mixes behave differently in freezing environments. The high air void contents associated with OGFC mixes act as insulation, resulting in the OGFC layer more resistant to the flow of heat through the pavement structure. Huber indicates that the heat conductivity of a porous asphalt mixture is 40 to 70 percent of a typical dense-graded mix. The surface temperature of OGFC layers is generally 2˚C (3.6˚F) cooler

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than a dense-graded layer. This means that an OGFC surface can drop below freezing resulting in the formation of ice/frost when a nearby dense-grade surface does not freeze. Because of the thermal properties of OGFCs, they will also stay frozen longer then dense-graded mixes.

Salts used on OGFCs during winter maintenance have less contact time with the icy surface. On OGFC, salt beings to melt the ice and form brine, which then disappears into the void structure of the OGFC.

Surface maintenance entails activities required to restore or preserve the surface condition of the pavement. Typical surface maintenance activities include crack sealing, pot hole repair, fog sealing and striping.

Of the 17 states that responded to Huber's survey, all 17 report that potholes and delaminated areas within OGFC are repaired with dense-graded HMA. Only one state agency mentioned crack filling as a routine surface maintenance activity. Crack sealing may cut off the flow of water within an OGFC, leading to other problems. Huber described a technique used in Britain in which pavement texture depth is monitored in the spring of each year. An increase in surface texture resulting from loss of surface fines may be an indication of impending failure due to raveling.

Huber states that lane markings are difficult to maintain on porous asphalt. Because of the high macrotexture and high air voids associated with OGFC, low viscosity striping materials will flow down into the voids and texture. A report from Ohio referenced in the synthesis states that 30 percent more material is required if epoxy is used and 50 percent more material is required if paint is used. Some agencies reported that high viscosity thermoplastic works well.

The use of fog seals in maintenance is mixed in the US. Some agencies use them while others do not. Fog seals are reported to reduce in-place air voids and, therefore, drainage capacity.

Vacuum-machines are reportedly used in some European countries to remove debris from an in-place OGFC pavement. High pressure water is sprayed on the pavement surface to dislodge debris and a high power vacuum is then used to remove the debris and water. This cleaning action is supposed to declog the pavement and extend the performance life.

1.21.6 Rehabilitation Practices

Huber states that the literature recommends the removal of OGFC layers prior to replacement. Hot in-place recycling has been used to rehab OGFC layers.

1.21.7 Performance

Huber defines the performance of OGFC into two categories: performance life and service life. Performance life is used to describe how long the OGFC maintains its permeability and ability to reduce noise levels. Service life deals with the ability to maintain friction and smoothness. A number of referenced papers were discussed that indicates that OGFCs maintain their sound attenuation for five years or more as long as

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the design air voids are above about 18 percent. Another important factor in maintaining performance life is traffic speed. Huber indicates that the combination of high air voids and high traffic speeds helps maintain an open void structure. The pressures induced by fast moving tires over a wet pavement surface causes hydraulic scouring of debris from the void structure. This was illustrated through a reference that indicated an OGFC placed in a slow speed urban environment lost permeability within 2 years.

In order to combat clogging tendencies in slow speed environments, a two-layer porous asphalt has been developed in Europe. The concept is to place a large aggregate layer of porous asphalt with a smaller aggregate OGFC on top (two-layer OGFC). The smaller aggregate size wearing surface will trap larger debris in order to maintain the permeability of the lower layer. The air void space in the bottom layer allows a water/jet vacuum machine to restore permeability.

Huber conducted a survey of 17 agencies that were major and minor users of OGFC. Fourteen of the respondents indicated that raveling was the most common failure mechanism for OGFCs. Cracking and potholes were both reported as the cause of failure by two agencies of the respondents. Delamination was identified by three agencies as cause of failure.

1.21.8 Structural Design

No specifics about inclusion of OGFC layers in structural design were given; however, Huber states that typical layer thicknesses in the U.S. are 20 to 25 mm (3/4 in. to 1 in.). Europe and South Africa have used thicker layers, 40 to 50 mm (1.5 in. to 2 in.).

1.21.9 Limitations

Huber cites a number of issues that can be considered detrimental to the performance of the pavement rather than true limitations. The synthesis states that OGFC pavements tend to form ice on the pavement surface at a warmer temperature than dense-graded surface. This leads to more frequent deicing applications. Huber also indicates that raveling is the most typical distress. Raveling can occur very quickly causing the pavement to almost disintegrate within a few months time.

Clogging can be the reason to discontinue the use of OGFC. Significant clogging will reduce all of the benefits related to permeability. Huber also states that clogged OGFC pavements may also accelerate moisture damage within underlying pavement layers. An unclogged OGFC layer may also result in moisture damage in underlying pavement layers. Huber states that the placement of OGFC may create a moist microenvironment at the surface of the underlying layer (bottom of the OGFC). The increased humidity caused by the moisture may retard the evaporation of moisture from the underlying layer resulting in moisture damage within the existing pavement.

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1.22 **Khalid, H. A. and C. M. Walsh. "Relating Mix and Binder Fundamental Properties of Aged Porous Asphalt Materials." 2nd Eurasphalt & Eurobitume Congress. Barcelona, Spain. Book 1. pp 398-405. 2000.**

1.22.1 General
This paper describes a research project that evaluated methods to characterize the aging characteristics of porous asphalt mixes. Three asphalt binders were included in the research in order to evaluate the effect of modifiers.

1.22.2 Benefits of Permeable Asphalt Mixtures
No specific benefits were given.

1.22.3 Materials and Mix Design
No specific mix design data was provided by the authors; however, the paper indicated that a 20mm nominal aggregate size porous asphalt mix was utilized. Aggregates included a hard basalt [no properties were given] with the inclusion of 2 percent hydrated lime by mass of aggregate. The hydrated lime was included in the research along with an unmodified pen 100 binder, 200 pen plus 5 percent SBS and a 100 pen plus 5 percent EVA.

Aging of the porous mixes was conducted with a device that forced air through the samples. Each sample was encased within a rubber membrane within a sealed platen system. Air from a valved manifold was forced through the sample. The air was heated to a temperature of 60˚C.

The effect of aging was evaluated using an indirect resilient modulus test to define a Stiffness Aging Ratio (SAR) (ratio of aged to unaged modulus). Stiffness Aging Ratio values were obtained over a 21 day period. For each of the binder types included in the experiment, the SAR increased with time. However, samples with the unmodified pen 100 asphalt binder showed more aging than the two modified binders.

Similarly, the asphalt binders were extracted, recovered, and tested in the dynamic shear rheometer to further evaluate the degree of aging. This data was used to define a Binder Aging Ratio that also indicated that the unmodified pen 100 binder aged more than the two modified binders.

The research showed an increased resistance to aging when polymer modified binders were used in porous asphalt.

1.22.4 Construction Practices
No specific construction practices were given.

1.22.5 Maintenance Practices
No specific maintenance practices were given.

1.22.6 Rehabilitation Practices
No specific rehabilitation practices were given.

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1.22.7 Performance

No specific performance measures were given.

1.22.8 Structural Design

No specifics on inclusion with structural design were given.

1.22.9 Limitations

No specific limitations were given.

1.23 Mallick, R. B., P.S. Kandhal, L. A. Cooley, Jr., and D. E. Watson. "Design, Construction, and Performance of New Generation Open-Graded Friction Courses." NCAT report No. 2000-01. National Center for Asphalt Technology. Auburn University. 2000.

1.23.1 General

This report documents an extensive laboratory study that was conducted to develop a laboratory mix design system for new generation OGFCs (or PFCs). This project was likely the first "national" effort in the US to develop mix design criteria for PFCs.

Also included within this report is a field evaluation of OGFC pavements that were designed and constructed in a manner that closely resembled a PFC pavement. The pavements had been in-service for six years at the time of evaluation.

1.23.2 Benefits of Permeable Asphalt Mixtures

The authors state that OGFC pavements improve wet weather driving conditions by allowing water to drain from the pavement surface. This drainage of water reduces hydroplaning, reduces splash and spray, improves wet weather frictional properties, improves surface reflectivity and reduces traffic noise.

1.23.3 Materials and Mix Design

The majority of this report was dedicated to describing a large laboratory study designed to develop a mix design system for PFC mixes. Two phases were included within the laboratory study. The first phase of laboratory work was designed to optimize the gradation requirements for PFC mixes. Additionally, the authors used the first phase of research to evaluate potential criteria for various laboratory tests for inclusion within the mix design system to be developed. The second phase of the laboratory study was designed to optimize the type of asphalt binder and additives to enhance performance. The following paragraphs describe the work conducted in the two phases of research

In the first phase of the study, blends were prepared with gradations similar to and coarser than the FHWA recommended gradation for OGFC mixes. Table 53 provides the FHWA recommended gradation and the other three new gradations evaluated in this study.

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Table 53: Gradations Used

	Percent Passing				
Sieve Size	Original FHWA Gradation	Gradation Similar to FHWA Used	New Gradation #1	New Gradation #2	New Gradation #3
19 mm	---	100	100	100	100
12.5 mm	100	95	95	95	95
9.5 mm	95-100	65	65	65	65
4.75 mm	30-50	40	30	25	15
2.36 mm	5-15	12	7	7	7
0.075 mm	2-5	4	3	3	3

The FHWA gradation has 40 percent material passing the 4.75 mm sieve, and the coarsest of the other three gradations has 15 percent material passing the 4.75 mm sieve. The coarsest gradation is very similar to the gradation that is being used by many states reporting good experience at the time with OGFC mixes. Mixes were prepared for these blends with an unmodified PG 64-22 asphalt binder. Mix designs were conducted according to the FHWA recommended procedures. These four blends were evaluated for stone-on-stone contact with voids in the mineral aggregate (VMA) and voids in the coarse aggregate (VCA) plots, and VCA data from dry-rodded tests with the coarse aggregate fraction only.

Samples prepared with the FHWA gradation and coarser gradations were tested for draindown potential, permeability, abrasion resistance, aging potential, and rutting. All samples were initially compacted with 100 gyrations of the Superpave gyratory compactor, which was considered to be equivalent to 50 blows of Marshall hammer in SMA mix design. The primary objective of Phase 1 was to evaluate the relative improvements in mix characteristics when the FHWA gradation was made coarser.

The average air voids (VTM), voids in mineral aggregate (VMA), voids in coarse aggregate (VCA), and voids filled with asphalt (VFA) data for the four different mixes are shown in Table 54. Although there is a difference of only 0.13% in asphalt binder content between the mixes with the four gradations, there is a significant range in voids (VTM, VMA and VCA). The VTM and VMA generally decrease with an increase in percent passing 4.75 mm sieve. Hence, the coarser the mix, the higher is the VTM and VMA. The dry rodded coarse aggregate VCA (VCA_{DRC}) falls between the compacted mix VCA values for gradations with 15 percent and 25 percent passing the 4.75 mm sieve. This indicates that stone-on-stone contact begins at some point between 25 percent and 15 percent (approximately at 22 percent) passing the 4.75 mm sieve for the materials used in the study. High VTM associated with the coarser gradation will also facilitate better drainage of water.

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Table 54: Summary of Mix Volumetric Properties

Gradation (% passing 4.75 mm sieve)	Asphalt Content	TMD*	Compacted OGFC Mix			
			VTM, %	VMA, %	VCA, %	VFA, %
15	5.55	2.475	15.1	26.3	37.3	42.6
25	5.51	2.512	14.3	24.5	43.3	41.7
30	5.48	2.511	13.6	24.0	46.6	43.3
40	5.42	2.487	12.5	23.9	54.1	47.3

* TMD = Theoretical maximum density
Dry rodded VCA = 41.7%.

Draindown tests were conducted on uncompacted OGFC mixes (with PG 64-22 binder) at 160C and 175C according to the NCAT draindown test method. The results of NCAT draindown test showed that the mix with 15 percent passing 4.75 mm sieve had the highest draindown potential. As the gradation became finer, draindown potential lessened.

The Cantabro abrasion test was conducted on the mixes with different percentages of material passing the 4.75 mm sieve. The data showed that under both aged and unaged conditions the abrasion loss increased as the mix became coarser. This indicates a higher potential for durability problems for the coarser gradations.

The permeability of mixes with different percentages of material passing the 4.75 mm sieve was tested with a falling head permeameter. As expected, the mixes with lower percentages of material passing the 4.75 mm sieve showed higher permeability. There was a significant increase in permeability between the mix with 30 percent passing the 4.75 mm sieve and the mix with 15 percent passing the 4.75 mm sieve.

Rut tests were conducted on the four mixes at design asphalt binder contents. The Asphalt Pavement Analyzer (APA) was used to evaluate rut potential. The rut depths at 8,000 cycles were very small, less than 5 mm for all mixes, and were considered acceptable.

In the second phase of the laboratory study, mixes were prepared at only 15 percent passing the 4.75 mm sieve and 6.5 percent asphalt binder content using six different binder/additive combinations.

A study was carried out to determine the required number of gyrations to provide a density closer to that seen at the time of construction (about 18 percent air voids). Three samples of each mix were compacted with 100 gyrations of the SGC and 50 blows of Marshall hammer. The air voids at different gyrations were compared to air voids generally found in the field and the air voids of the sample compacted with 50 blows Marshall. It was determined that about 50 gyrations with the SGC and 50 blows with the Marshall hammer produce about 18 percent air voids generally found in the field. The

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mixes were then prepared with six different types of binder (Table 55). The samples were tested for volumetric properties, draindown, aging, rutting, and moisture susceptibility.

The average draindown values at 157C (315F) are shown in Table 56. Results from a multiple comparison test are also shown in Table 56. These results indicate whether there is any significant difference between the different means, and if there is, provides the ranking of the different mixes based on the means. Table 56 indicates that the use of fibers greatly reduces the potential for draindown; more so than does polymer modification.

Table 55: Volumetric Properties of mixes with Difference Binders (Average Values)

Binder	Bulk Sp. Gr.	TMD	VTM	VMA	VCA
PG 64-22	2.044	2.441	16.3	29.0	37.3
PG 64-22 with cellulose	2.043	2.441	16.3	29.0	37.3
PG 64-22 with slagwool	2.071	2.441	15.2	28.1	37.3
PG 64-22 with SBS	2.026	2.441	17.0	29.6	37.3
PG 76-22-SB	2.002	2.441	18.0	30.5	37.3
PG 76-22 with slagwool	2.046	2.441	16.2	28.9	37.3

Table 56: Results of Draindown Tests from Mixes with Different Binders

Draindown at 157C (315F)		
Duncan Grouping	Mean (%)	Asphalt Binder
A	1.3585	PG 64-22
A	1.1845	PG 76-22-SB
B	0.5405	PG 64-22 with SBS
B	0.1245	PG 76-22-SB with slagwool
B	0.0510	PG 64-22 with slagwool
B	0.0040	PG 64-22 with cellulose

Samples of mixes prepared with the different binders were tested with the Cantabro Abrasion test to determine the effect of aging. All of the samples were aged at 50C for 168 hours (7 days). Table 57 shows the test values and the results of multiple comparison tests. Results show that the mixes with unmodified PG 64-22 binder had the highest abrasion loss, and the mixes with PG 76-22-SW had the lowest abrasion loss. The data clearly showed that the combined use of polymer modified binder and fiber will minimize the laboratory abrasion loss and, thus, increase durability of the mixture.

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Table 57: Abrasion Loss (Aged Samples) for Mixes with Different Types of Binder

Duncan Grouping		Mean (%)	Asphalt Binder
	A	26.2	PG 64-22
B	A	19.3	PG 64-22 with slagwool
B	A	18.8	PG 64-22 with cellulose
B	C	15.7	PG 76-22-SB
B	C	13.0	PG 64-22 with SBS
	C	9.0	PG 76-22 with slagwool

Moisture susceptibility of mixes was evaluated by conducting tensile strength test on conditioned (5 freeze/thaw cycles) and unconditioned compacted samples of mixes with different binders. Test samples were compacted to the standard laboratory compaction effort instead of a target air void content for this testing. This test was included in Phase 2 to evaluate the effect of binder type and fibers on the moisture susceptibility of OGFC mixes. The test results indicated that both polymer-modified binder and fiber should be used especially in the northern tier states of the U.S., which experience cold climates and freeze/thaw cycles.

Based on the research conducted by the authors, the following were concluded:

1. A gradation with no more than about 20 percent passing the 4.75 mm sieve is required to achieve stone-on-stone contact condition and provide adequate permeability in OGFC mixes.

2. Mixes with 15 percent of the aggregate fraction passing the 4.75 mm sieve are susceptible to significant draindown of the binder. Therefore, it is necessary to provide a suitable stabilizer such as fiber in the mix to prevent excessive draindown.

3. Abrasion loss of OGFC mixes resulting from aging can be reduced significantly with the addition of modifiers. In this study, all of the modified binders had significantly lower abrasion loss than mixes using an unmodified binder. The use of both polymer-modified binder and fiber can minimize the abrasion loss and thus increase the durability of OGFC.

4. For the binders used in this study, rut depths as measured with the APA did not vary over a wide range. However, within the range of rut values obtained, the mixes with modified binders had less rutting than mixes with unmodified binders. A higher PG binder grade seems to have a greater effect in reducing rutting than a lower PG binder grade. A polymer-modified asphalt with fiber gave the least amount of rutting.

5. Moisture susceptibility, as measured by TSR values, is lower for mixes with modified binders than mixes with unmodified binders. All of the modifiers except slagwool (with PG 64-22) produced mixes which had TSR values in excess of 80 percent.

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Again, both polymer-modified binder and fiber should be most effective especially in cold climates with freeze/thaw cycles.

To provide a clear understanding of the methods used by the authors, the following paragraphs describe the various tests utilized in the research. Similar to stone matrix asphalt (SMA), OGFC must have a coarse aggregate (retained on No. 4.75 mm) skeleton with stone-on-stone contact to minimize rutting. The condition of stone-on-stone contact within an OGFC mix is defined as the point at which the voids in coarse aggregate (VCA) of the compacted OGFC mixture is less than the VCA of the coarse aggregate alone in the dry-rodded test (AASHTO T19).

The NCAT draindown test method was used to evaluate draindown potential. A sample of loose asphalt mixture was prepared and placed in a wire basket, which is positioned on a plate or other suitable container of known mass. The sample, basket, and plate or container were then placed in a forced draft oven for one hour at a pre-selected temperature. At the end of one hour, the basket containing the sample is removed from the oven along with the plate or container and the mass of the plate or container is determined. The amount of draindown was then calculated as the mass of asphalt binder that drained onto the plate divided by the total mass of mix.

The Florida DOT falling-head laboratory permeability test was used. This test uses a falling head concept to determine permeability.

The resistance of compacted OGFC specimens to abrasion loss was analyzed by means of the Cantabro Abrasion test. This is an abrasion and impact test carried out in the Los Angeles Abrasion machine (ASTM Method C131). In this test, a single OGFC specimen compacted with 50 blows on each side was used. The mass of the specimen was determined prior to testing. The test specimen was then placed in the Los Angeles Rattler without the charge of steel spheres. The operating temperature was room temperature. The machine was operated for 300 revolutions at a speed of 30 to 33 rpm. The test specimen was then removed and its mass determined. The percentage abrasion loss was then calculated.

The recommended maximum permitted abrasion loss value for freshly compacted specimens was 20 percent. However, the authors state that some European countries specify a maximum value of 25 percent.

Both unaged and aged compacted OGFC were subjected to Cantabro abrasion testing to evaluate the effect of accelerated laboratory aging on resistance to abrasion. Because of the very high air void contents, the asphalt binder in OGFC is prone to hardening at a faster rate than dense-graded hot mix asphalt, which may result in reduction of cohesive and adhesive strength leading to raveling. Aging was accomplished by placing five Marshall specimens compacted with 50 blows in a forced draft oven set at 60C for 168 hours (7 days). The specimens were then cooled to 25C and stored for 4 hours prior to conducting the Cantabro Abrasion test. The average of the abrasion losses obtained on 5

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aged specimens should not exceed 30 percent, while no individual result should exceed 50 percent.

Raveling of the OGFC may take place due to stripping in the mix, especially from freeze and thaw cycles in northern tier states with cold climates. The modified Lottman test (AASHTO T283) was used in this study; however, instead of using one freeze/thaw cycle, five cycles were used. Samples were compacted using 50 blows per face instead of targeting 7 percent air voids as stated in AASHTO T283. Since the air void content is higher in the OGFC compared to dense-graded HMA, more severe conditioning was deemed necessary to evaluate the stripping potential.

The potential for rutting of OGFC was evaluated with the Asphalt Pavement Analyzer (APA). Cylindrical OGFC specimens were loaded at 64C (both dry and under water) for 8,000 cycles and rut depth measured.

The following tentative mix design system was recommended for the new-generation OGFC mixes on the basis of the laboratory study, observation of in-place performance of OGFC mixes in Georgia, and experience in Europe.

Step 1. Materials Selection

The first step in the mix design process is to select materials suitable for OGFC. Materials needed for OGFC include aggregates, asphalt binders, and additives. Additives include asphalt binder modifiers, such as polymers and fibers.

Guidance for suitable aggregates can be taken from recommendations for SMA. The binder selection should be based on factors such as environment, traffic, and expected functional performance of OGFC. High stiffness binders, such as PG 76-xx, made with polymers are recommended for hot climates or cold climates with freeze-thaw cycles, medium to high volume traffic conditions, and mixes with high air void contents (in excess of 22 percent). The addition of fiber is also desirable under such conditions and also has been shown to significantly reduce draindown.

Step 2. Selection of Design Gradation

Based upon this laboratory study and experiences in Georgia with Porous European Mix, the following master gradation band was recommended.

Sieve	Percent Passing
19 mm	100
12.5 mm	85-100
9.5 mm	55-75
4.75 mm	10-25
2.36 mm	5-10
0.075 mm	2-4

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Selection of the design gradation entails blending selected aggregate stockpiles to produce three trial blends. It was suggested that the three trial gradations fall along the coarse and fine limits of the gradation range along with one falling in the middle. For each trial gradation, determine the dry-rodded voids in coarse aggregate of the coarse aggregate fraction (VCA_{DRC}). Coarse aggregate is defined as the aggregate fraction retained on the 4.75 mm sieve.

For each trial gradation, compact specimens at between 6.0 and 6.5 percent asphalt binder using 50 gyrations of a Superpave gyratory compactor. If fibers are a selected material, they are included in these trial mixes. Determine the voids in coarse aggregate (VCA) for each compacted mix. If the VCA of the compacted mix is equal to or less than the VCA_{DRC}, stone-on-stone contact exists. To select the design gradation, choose a trial gradation that has stone-on-stone contact combined with high voids in total mix.

Step 3. Determine Optimum Asphalt Content

Using the selected design gradation, prepare OGFC mixes at three binder contents in increments of 0.5 percent. Conduct draindown testing on loose mix at a temperature 15C higher than anticipated production temperature. Compact mix using 50 gyrations of a Superpave gyratory compactor and determine air void contents. Conduct the Cantabro Abrasion test on unaged and aged (7 days @ 60C) samples. Rutting tests with the Asphalt Pavement Analyzer and laboratory permeability testing are optional. Insufficient data was accumulated in this study to recommend a critical rut depth. However, laboratory permeability values greater than 100 m/day are recommended. The asphalt content that meets the following criteria is selected as optimum asphalt content.

1. **Air Voids.** A minimum of 18 percent is acceptable, although higher values are more desirable. The higher the air voids are the more permeable the OGFC.

2. **Abrasion Loss on Unaged Specimens.** The abrasion loss from the Cantabro test should not exceed 20 percent.

3. **Abrasion Loss on Aged Specimens.** The abrasion loss from the Cantabro test should not exceed 30 percent.

4. **Draindown.** The maximum permissible draindown should not exceed 0.3 percent by total mixture mass.

If none of the binder contents tested meet all four criteria, remedial action will be necessary. Air voids within OGFC are controlled by the binder content and gradation. If air voids are too low, the asphalt binder content should be reduced or the gradation made coarser. If the abrasion loss on unaged specimens is greater than 20 percent, more asphalt binder or a stiffer asphalt binder is needed. Abrasion loss values of aged specimens in excess of 30 percent can be remedied by either increasing the binder content or changing the type of binder additive. If draindown values are in excess of 0.3 percent, the amount of binder and/or type of binder additive can be adjusted. Fiber stabilizers are typically incorporated into the mix at a rate of 0.2 to 0.5 percent of the total mix.

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Step 4. Evaluate Mix for Moisture Susceptibility

The mix designed with Step 1 through 3 should be evaluated for moisture susceptibility using the modified Lottman method (AASHTO T283) with five freeze/thaw cycles in lieu of one cycle. The retained tensile strength (TSR) should be at least 80 percent.

1.23.4 Construction Practices

The authors reported on the construction of six OGFC test sections placed on Interstate 75 near Atlanta, Georgia. The test sections were part of a field research study designed to evaluate OGFC mixes. The Georgia Department of Transportation wanted to compare their conventional OGFC mixes to coarser OGFCs. More specifics on the field project are provided in the section on performance within this review. Production of the mixes in the field was accomplished utilizing a double-barrel drum plant. The plant was slightly modified in order to incorporate fibers into the mixture as well as the addition of dry crumb rubber for one test section.

1.23.5 Maintenance Practices

No specific maintenance practices were given.

1.23.6 Rehabilitation Practices

No specific rehabilitation practices were given.

1.23.7 Performance

This paper presents information and data on six field test sections that were constructed near Atlanta, Georgia. The six test sections were characterized as a coarse OGFC (D), coarse OGFC with 16 percent crumb rubber (D16R), coarse OGFC with cellulose fibers (DC), coarse OGFC with mineral fibers (DM), coarse OGFC with SB polymer (DP), and coarse OGFC with SB polymer and cellulose fibers (DCP). Mix designs for each of these mixes were conducted using the "Method of Determining Optimum Asphalt Content for Open-Graded Bituminous Paving Mixtures," which is a standard procedure for GDOT (GDT-114).

Job-mix-formula (JMF) data for each of the six mixes are presented in Table 58. This table shows that all six OGFC mixes had identical gradations and only differed by respective asphalt contents. Of interest, the JMF gradation falls within the gradation band recommended by the authors of this paper in the new mix design system.

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Table 58: Laboratory Test Results for the Six OGFC Mixes (6)

Test	D	D16R	DM	DC	DP	DCP
Percent passing 19.0 mm	100	100	100	100	100	100
Percent passing 12.5 mm	99	99	99	99	99	99
Percent passing 9.5 mm	75	75	75	75	75	75
Percent passing 4.75 mm	18	18	18	18	18	18
Percent passing 2.36 mm	8	8	8	8	8	8
Percent passing 0.075 mm	2	2	2	2	2	2
Percent Asphalt Binder of Total Mix						
% AC	6.0	6.6	6.3	6.4	6.2	6.4
Other Test Data						
Cantabro (% Wear)	13.5	8.6	5.7	5.8	8.6	8.2
Drainage (% Loss)	0.37	0.05	0.06	0.06	0.34	0.04

During production, truck samples were obtained to determine asphalt content, gradation, air voids, and Cantabro abrasion loss values. Table 59 presents the results of this testing.

Table 59: Laboratory Test Results for Field Produced OGFC Mixes (6)

Sample Type	JMF	D	DM	DC	DCP	DP	D16R
Sieve Size, mm	Total Percent Aggregate Passing by Weight						
19.0	100	100	100	100	100	100	100
12.5	99	98.3	98.9	96.7	97.0	99.1	96.3
9.5	75	70.0	76.2	64.0	68.6	69.9	60.3
4.75	18	21.0	23.9	19.0	19.1	23.1	15.7
2.36	8	8.7	9.0	7.7	7.8	8.4	7.4
0.075	2	3.6	3.1	2.8	2.4	3.1	2.6
Miscellaneous Test Data							
Asphalt Content	Extracted	5.85	6.22	6.16	6.14	6.25	6.41
TMD	---	2.484	2.445	2.429	2.424	2.476	2.451
VTM	---	12.2	11.4	11.5	10.9	14.1	12.0
Cantabro (% Wear)	---	10.3	8.1	14.7	7.0	15.9	7.6

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Based on Table 59, five of the produced mixes were finer than the JMF gradation on the 4.75 mm sieve. Only one gradation (DM) did not meet the recommended gradation band presented previously. However, this mix only varied from the band by 1.2 percent on the 9.5 mm sieve. Asphalt contents ranged from 5.9 to 6.4 percent. Air void contents of lab compacted samples using 25 blows per face of a Marshall hammer ranged from 10.9 to 14.1 percent and are lower than would be anticipated on the roadway. Cantabro abrasion loss values ranged from 7.0 to 15.7 percent and are all lower than the suggested 20 percent maximum criteria.

In addition to testing truck samples, cores were obtained from each of the six test sections. Testing of these samples included asphalt contents and gradations by extraction and air void calculations. An additional test conducted was the in-place permeability of each section. Results of this testing are presented in Table 60.

Table 60: Laboratory Test Results for Roadway Core Samples from OGFC Test

Sample No.	JMF	D	DM	DC	DCP	DP	D16R
Sieve Size, mm	Total Percent Aggregate Passing by Weight						
19.0	100	100	100	100	100	100	100
12.5	99	99.3	98.6	99.2	97.6	99.3	99.2
9.5	75	77.3	77.2	75.5	73.1	76.5	76.7
4.75	18	28.1	28.3	28.0	26.9	27.8	28.0
2.36	8	13.1	13.6	13.7	13.0	13.1	13.1
0.075	2	3.8	4.1	3.5	3.9	3.8	3.4
Miscellaneous Test Data							
Asphalt Content	Extracted	5.51	5.87	6.18	5.27	5.85	5.69
VTM	---	17.8	17.2	16.4	16.0	17.6	18.1
Permeability (m/day)	---	46	82	71	71	84	67

Of note in Table 60 are the in-place air void contents of the compacted mixes. Air void contents ranged from 16.0 to 18.1 percent which relate well to the data accumulated in the laboratory part of this study. These values also seem to validate the selection of 18 percent air voids minimum for the new mix design system as mixes meeting the gradation requirements can be constructed to have 18 percent air voids. Permeability values obtained from the six test sections ranged from 46 to 84 m/day and appear to correspond reasonably well with permeability data from the laboratory work in this study.

During 1998 (six years after construction), a visual distress survey was performed on the six OGFC test sections. The survey consisted of evaluating each section for surface texture, rutting, cracking, and raveling. During the course of the survey, cores were obtained from each section and used to determine the laboratory permeability.

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All six test sections had experienced some coarse aggregate popout. The D16R section appeared to have the most while the DC, DM, and DP sections all had a very low amount. Another surface texture item was the existence of small fat spots on the pavement surface. Each of the six sections had these fat spots. However, none were larger than approximately 15 cm diameter. The D and DCP appeared to have the most but were not deemed significant.

Rutting

Rut depth measurements were made for each section using a stringline. Rut depths ranged from 0.0 mm for the DP section to 4.1 mm for the DC section. None of the sections were characterized as having significant amounts of rutting.

Cracking

The primary form of cracking on all six sections was reflective from a Portland cement concrete pavement underlying each section. Table 61 presents descriptions and percentages of reflective cracks encountered. Percentages were determined by counting the number of transverse cracks visible at the pavement surface.

Table 61: Severity and Percentage of Transverse Reflective Cracks

Section	Description	% Cracks Showing
D	Low to medium severity	75
D16R	Low to high severity	87
DM	Low severity	55
DC	Low severity	45
DP	Low to medium severity	61
DCP	Low to medium severity	65

Table 61 shows that five of the six sections had low to medium severity reflective cracking. The two OGFC mixes containing only fibers (DM and DC) had the least amount (and severity) of cracking while the D16R section had the highest amount and severity cracking. Reflective longitudinal cracks were also observed on five sections. Only the D and D16R sections had what could be characterized as medium severity longitudinal cracking.

Besides reflective cracking, only the D16R section showed any other type of cracking. Secondary cracking around some reflective cracks had occurred.

Raveling

All six sections showed some signs of raveling. However, all raveling was minimal except the D16R section which showed some medium severity raveling next to some cracks.

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Three 150 mm cores were obtained from each of the six sections. Table 62 presents the average laboratory permeability values from each section as well as average in-place air void contents. Statistically, no significant differences exist between the permeability values; however, the DC and DCP sections did have the highest average permeability at 74 and 70 m/day, respectively. In-place air void contents ranged from 15 to 19 percent. Bulk specific gravity measurements were determined by volumetric measurements. It is interesting that this range of air void contents correspond well with the roadway core air void contents (at the time of construction) presented in Table 60. Again, this appears to provide validity to the selection of 18 percent air voids minimum in the new mix design system. This criterion appears to be related to air void contents at both construction and during the life of an OGFC pavement. Additionally, it appears that the mix design procedure used by GDOT in this experiment resulted in OGFC mixes with stone-on-stone contact even though it was not specifically tested.

Table 62: Average Permeability and In-Place Air Void Contents for the Six Test Sections

Section	Avg. Permeability m/day	Avg. In-Place Air Voids, %
D	25	16.7
D16R	38	15.8
DM	28	19.9
DC	74	16.2
DP	16	13.9
DCP	70	19.2

1.23.8 Structural Design

No specifics on inclusion within structural design were given.

1.23.9 Limitations

No limitations on use were given.

1.24 Molenaar, J.M.M. and A.A.A. Molenaar. "An Investigation into the Contribution of the Bituminous Binder to the Resistance to Raveling of Porous Asphalt." 2nd Eurasphalt & Eurobitume Congress. Barcelona, Spain. pp 500-508. 2000.

1.24.1 General

This paper provides the results of a research project designed to evaluate the effect of polymer modification on the properties of porous asphalt as related to short-term raveling and to develop a method for selection of an appropriate asphalt binder to improve the resistance to short-term raveling. For the purposes of this paper, short-term raveling was defined as raveling caused by intense shearing forces at the tire/pavement interface that occurs within newly placed porous asphalt. Long-term raveling was assumed by the authors to be caused by gravity segregation of the mastic during the service life of the

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porous asphalt. Upon the mastic segregating from the coarse aggregate, the action of traffic will break loose the aggregate particles.

1.24.2 Benefits of Permeable Asphalt Mixtures
No specific benefits were given.

1.24.3 Materials and Mix Design
The researchers utilized two laboratory tests to evaluate the short-term potential for raveling: the Wheel Fretting Test and the Californian Abrasion Test. For the Wheel Fretting Test (WFT), a treaded tire inflated to 600 kPa (87 psi) and loaded with 3 kN (675 lb) was run in a circular path on top of porous asphalt test specimens under an inclination angle of 2 to 5 degrees. A total of 3 million revolutions (applications) of the treaded tire were applied to the porous asphalt test specimens at a test temperature of approximately 20ºC. The fretting performance was characterized as a mass loss after the wheel passes.

The Californian Abrasion test (CAT) utilizes a mechanical shaker that is operated at 20 cycles per second with a given vibration amplitude. A sample of porous asphalt is placed within a container along with water and steel spheres and subjected to the vibration action for 15 minutes at a test temperature of 4ºC. Test results are expressed as mass loss after the 15 minutes of abrasive action.

During the research, porous asphalt was prepared with a total of eight asphalt binders. Two of the binders were neat (non-modified) binders while six were polymer-modified. Of the six modifiers used, four were elastomeric and two were plastomers.

Based upon the results of testing, the authors concluded that polymer modification is beneficial to resist the effect of short-term raveling, whether this raveling occurs at low or at high pavement temperatures. The WFT and CAT were effective at providing an indication of short-term raveling at their respective test temperature.

1.24.4 Construction Practices
No specifics on construction practices were given.

1.24.5 Maintenance Practices
No specific maintenance practices were given.

1.24.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.24.7 Performance
No performance measures were given.

1.24.8 Structural Design
No specifics on inclusion within structural design were given.

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1.24.9 Limitations
No limitations on the use were given.

1.25 Pasetto, M. "Porous Asphalt Concretes with Added Microfibres." 2nd Eurasphalt & Eurobitumen Congress. Barcelona, Spain. pp. 438-447. 2000.

1.25.1 General
This paper describes a research effort designed to evaluate different fibers within porous asphalt. Three different fiber types were investigated: polypropylene, polyacrylonitrile and cellulose. Additionally, the use of a Superpave gyratory compactor to provide compactive effort during mix design was evaluated.

1.25.2 Benefits of Permeable Asphalt Mixtures
No specific benefits were given.

1.25.3 Materials and Mix Design
Materials utilized for the porous asphalt during the research included aggregates, asphalt binder and the three types of fiber. Three aggregate stockpiles were included within the mixture: porphyric aggregate, basaltic aggregate and sand. Pasetto indicates that the two coarse aggregate sources had Los Angeles Abrasion loss values of 14 percent. Also, the Polishing Stone Value was higher than 0.59 for both aggregate sources. Table 63 provides the specified gradation band and the design gradation utilizing the selected aggregates. A polymer-modified penetration-graded 50/60 asphalt binder was used.

Table 63: Gradation Requirements and Design Gradation

Sieve, mm	Gradation Band	Design Gradation
16.0	100	100
12.5	22 – 100	63.2
9.50	20 – 35	26.4
6.30	17 – 30	17.2
4.75	15 – 27	17.1
2.00	10 – 18	14.6
0.425	7 – 12	9.6
0.180	6 – 10	7.4
0.075	5 – 8	5.1

As stated above, three different types of fibers were included within the research. Table 64 provides properties of the three fibers. The polypropylene fiber was added at 0.35 percent, the polyacrylonitrile fiber was added at 0.126 percent and the cellulose was added at 0.3 percent.

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Table 64: Properties of Fibers

Property	Units	Polyacrylonitrile	Polypropylene	Cellulose
Length	mm	12	6	1.1
Thickness	mm	0.016	0.020	0.045
Specific Weight	g/cm^3	1.180	1.32-1.40	---
Elastic Modulus	GPa	23	---	---
Tensile Strength	MPa	800	550	---
Ultimate Elongation	%	9.8	33	---

Samples of the porous asphalt were compacted using both the Marshall hammer and the Superpave gyratory compactor. [No specifics on the number of blows per face used to compact samples with the Marshall hammer were provided; however, it is assumed that 50 blows per face were used.] Samples having a diameter of 150 mm were compacted to a maximum of 130 gyrations in the Superpave gyratory compactor.

Results of testing indicated an increase in the strength properties of porous mixes containing fibers when compared to a control mix with no fibers. This was true for mixes containing all three fiber types. Strength tests within the study included Marshall stability and indirect tensile strength. Also included within the study was the Cantabro Abrasion test. Again, inclusion of fibers reduced the amount of abrasion loss when fibers were added to the porous asphalt.

Figure 13 illustrates the results for the comparison between the Marshall hammer and Superpave gyratory compactor. This figure shows that 50 gyrations of the Superpave gyratory compactor is comparable to the standard Marshall hammer compactive effort (assumed as 50 blows per face).

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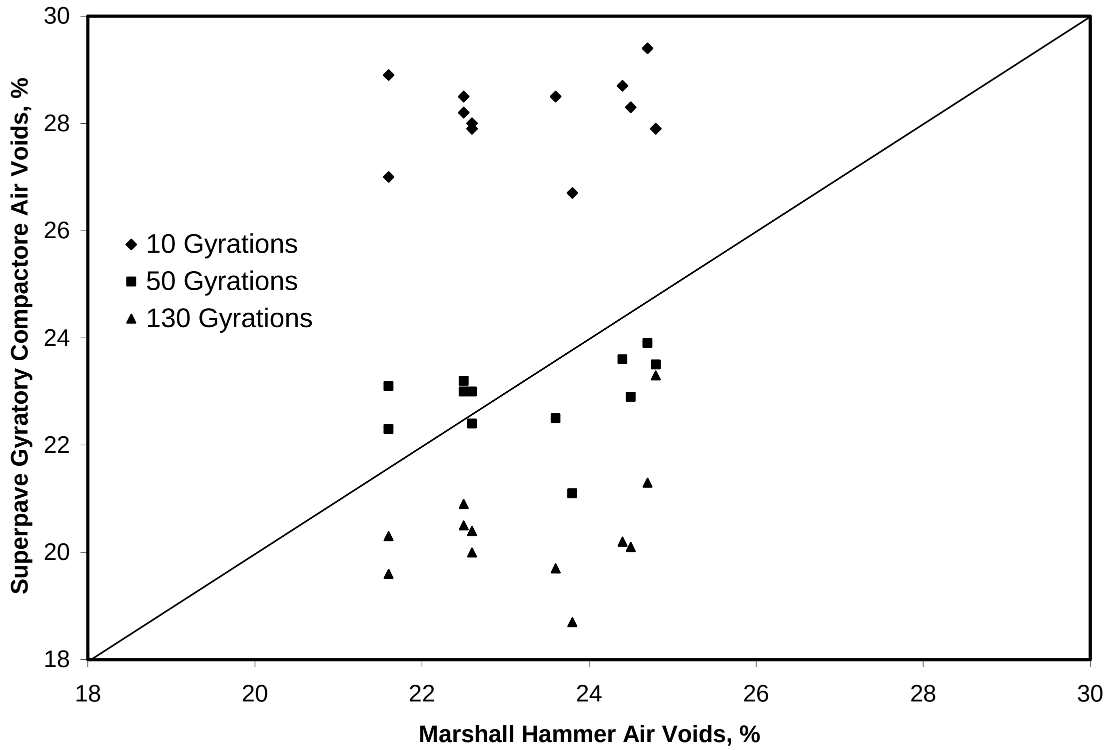

Figure 13: Comparison of Air Void Contents

1.25.4 Construction Practices
No specific construction practices were given.

1.25.5 Maintenance Practices
No specific maintenance practices were given.

1.25.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.25.7 Performance
No specific performance measures were provided; however, Pasetto indicated that raveling was a problem.

1.25.8 Structural Design
No specifics on inclusion within structural design were given.

1.25.9 Limitations
No specific limitations were given.

1.26 Spillemaeker, P.E., and P. Bauer. "Development of 0/6 Porous Asphalt." 2nd Eurasphalt & Eurobitume Congress. Barcelona, Spain. Pp. 553-557. 2000

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1.26.1 General

This paper describes an effort to develop a 2/4 gap-graded 0/6 porous asphalt mixture. This designation means that the aggregate gradation has a maximum aggregate size of 6mm with a gap in the grading between 2 and 4mm.

1.26.2 Benefits of Permeable Asphalt Mixes

The authors mention frictional properties and reduced tire/pavement noise as benefits.

1.26.3 Materials and Mix Design

Specific recommendations for individual material requirements were not provided. Also, gradation requirements for individual sieves were not provided; however, between 10 and 12 percent of aggregate pass the 2mm size for the mix type. A polymer-modified binder was included. The Cantabro Abrasion test was used during mix design using a test temperature of 20˚C. Other tests used to evaluate the mix included the rotary shearing press test (rutting test) and the LCPC Duriez test (moisture susceptibility).

1.26.4 Construction Practices

The authors state that the porous asphalt was produced in a drum mix facility. The tack coat used on the existing layer was a 65 percent emulsion placed at a rate ranging from 350 to 450 g/m^2 of residual binder.

The mix was placed with pavers over the full width of the roadway (11.5m). Both vibratory and static steel wheel rollers were used to compact the mix, generally using four to six passes.

1.26.5 Maintenance Practices

No specifics on maintenance practices were given.

1.26.6 Rehabilitation Practices

No specifics on rehabilitation were given.

1.26.7 Performance

The authors used permeability tests conducted in the field as a performance measure.

1.26.8 Structural Design

No specifics on inclusion within structural design were given.

1.26.9 Limitations

No specific limitations were given.

1.27 Bishop, M. C. and M. F. Oliver. "Open Graded Friction Course Pavements In British Columbia." Proceedings of the 46th Annual Conference of the Canadian Technological Asphalt Association. Toronto, Canada. 2001.

1.27.1 General

In this paper Bishop and Oliver describe several OGFC projects constructed in British Columbia (BC), Canada. The authors present their experiences regarding benefits of

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OGFC, project selection, mix design, construction, structural strength, performance, maintenance, rehabilitation and cost of OGFC.

Based on the experience of the OGFC projects and literature review, the authors make several recommendations for designing and constructing good performing OGFC mixes. These recommendations include proper repair or rehabilitation of existing roads to remove any distressed areas before overlaying with OGFC, providing adequate thickness of the OGFC mixes, reducing draindown with the use of fibers and polymer modification, using the right balance of air voids and film thickness and following certain construction practices. These recommended construction practices include use of tack coat, not sealing longitudinal joint faces, producing mixes at a lower temperature compared to conventional mixes, providing unimpeded edge drains and using light rolling for compaction.

Regarding maintenance Bishop and Oliver mention that close attention should be paid to OGFC surfaces, and that the greater cost of using more salts in such surfaces is well justified, considering its benefits. They indicate that the best option for repair of slight distresses is to apply a light seal or an open graded asphalt mix as early as possible. Regarding rehabilitation Bishop and Oliver mention that a second OGFC layer could be applied on an existing OGFC mix, as long the latter is sealed properly, and that the entire width of the pavement needs to be rehabilitated at once to ensure adequate drainage.

1.27.2 Benefits of Permeable Asphalt Mixtures

Bishop and Oliver mention the following benefits of OGFC mixes: 1) Higher friction and shorter stopping distances in wet conditions; 2) Improved visibility by reduction of glare, and hence reduction in accidents and congestion; 3) Increased rutting resistance due to "rock-to-rock contact" in OGFC mixes; and 4) Reduction of traffic noise.

1.27.3 Materials and Design

Bishop and Oliver provide a description of materials and mix designs used in several projects, and also provide recommendations, based on their experience and literature surveys. Table 65 summarizes the materials and mix designs for the projects.

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Table 65: Materials and Mix Design Used in Projects in BC

Project	Materials/ Construction	Observations/Special Provisions
Highway 19 – Nanaimo Projects – Project 1, traffic volume of 54,000 AADT.	100% fractured 12.5 mm maximum size aggregate; spreading rate of 40 kg/m², equivalent to a thickness of 18 mm.	1. Lift thickness insufficient for covering existing irregularities and providing internal drainage. 2. Drainage to edge drain was affected by shoulder gravel and concrete curb. 3. Provision of drainage through butt joints must be made, to allow flow of water particularly when a downward slope towards an abutting dense-graded pavement mix is present.
Highway 19 – Nanaimo Projects – Project 2 traffic of 54,000 AADT.	application rate to 75 kg./m² (≈43 mm)	1. The edge of the mat was left 100 mm short of the existing concrete curbs; gutters were made for proper drainage. 2. Angled transverse butt joints were specified for tying to existing pavement. 3. The rate of application of tack coat was increased to 0.75 l/m² to ensure sealing of the underlying pavements.
Vancouver Island Highway Project - Duke Point Access - Traffic volumes were projected to be 5000 AADT with a high percentage of loaded trucks.	A 50 blow Marshall mix design method was used to provide 15% air voids with a 15 micron film thickness. The fines returned from baghouse dust collection system were not put back in the mix, to increase air voids (to approximately 18%) and film thickness (to approximately 20 microns).	1. Design procedures were based on South African process as well as experiences from the Nanaimo projects. 2. All tangents were constructed with a 2.5 % crown to improve the internal drainage of the pavement. This was done to mitigate the internal drainage problems expected because of the width of grade being constructed (4 lanes + shoulders + median).
Highway 16 near Terrace in 1993; carries a significant number of heavy logging trucks.	Class D seal coat aggregate; aggregates and mix design similar to those used in Nanaimo Projects	Pavement is still performing well (reported in 2001)
Likely Road - 150 Mile House; placed in 1990.	This was a relatively thin lift utilizing a gradation similar to that used in Nanaimo. The lift thickness would put it in the "Carpet Coat" category.	This was initially surfaced with a graded aggregate seal. Over time the seal had deteriorated exposing the concrete. In 1995 this section was resurfaced with a thin lift OGFC pavement. Reports to date indicate that it is performing well and providing the desired qualities of enhanced friction and surface drainage.

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Table 65: Materials and mix design used in projects in BC (continued)

Project	Materials/Mixes/ Structure/ Construction	Observations/Special Provisions
Coquihalla Highway - Phase 2 placed in 1997 on Phase 2 of the Coquihalla Highway.	Using the same parameters as the Nanaimo projects	---
Nakusp This project was completed in the summer of 2000, located in an area with significant freeze thaw cycles and intensive winter maintenance.	The mix design used on this project had 13.8% air voids and a film thickness of 17.2 microns. The mix was produced with 85% 12.5 mm maximum size material from a single stockpile and 15% 16.0 mm maximum size rock that was produced for the conventional hot mix used on this project. An anti stripping agent was added to the asphalt cement.	Problems encountered in this project include those with gradation, erosion of shoulder material, intersection with side roads and achieving adequate film thickness. The details are as follows. 1) 15 % crushed 16.0 mm aggregate was added to make the gradation coarser. A minimum limit on percentage retained on 9.5 mm (instead of the 100 percent passing), and splitting of stockpile on 4.75 m sieve were suggested; 2) Difficulty in creating a drainage path between the OGFC and the curb; 3) It was determined that putting roadside barriers with asphalts curbs behind them is a good idea for ensuring proper flow of water along the curb; 4) To reduce problems associated with intersecting roads, the limits of the OGFC were extended and also by milling and creating the intersections at an angle to facilitate drainage through OGFC; 5) Lack of finish rolling can cause problem with smoothness; 6) Tenderness of mix can be reduced by lowering compaction temperature and adding silicone to reduce the effect of free moisture; 7) The South African recommendation of considering edge drains in projects with curbs and gutters was considered; 7) Design film thickness values were not obtained, probably due to degradation of aggregates in the plant.

The authors point out the following observations: a) A 50 blow Marshall procedure can be used. b) Draindown can be minimized using a rubber or polymer-modified asphalt and by adding cellulose fiber. c) A clear relationship between draindown problems and the degree to which the mix could be described as single sized.

Bishop and Oliver point out that a number of agencies that experienced difficulties with draindown used gradations where all the material was essentially retained on one or two sieves. They postulate that agencies that allow gradations with more fines appeared to have less concerns with draindown. Based on the success of the first projects in British Columbia, the targets were set at 15% minimum air voids and 15 micron minimum film thickness after deduction of absorbed asphalt. They suggest the gradation given in Table 66.

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Table 66: Suggested Gradation

Sieve Size, mm	Percent passing
16	100
12.5	92-100
9.5	75-90
4.75	50-70
2.36	10-35
1.18	0-21
0.6	0-10
0.3	0-8
0.15	0-6
0.075	0-4

Regarding materials, the authors mention that aggregates should be clean, sound, polish resistant with a minimum of flat particles, and that asphalt should generally be of a grade equal or harder than that used for dense-graded mixes.

1.27.4 Construction Practices

Bishop and Oliver mention that asphalt surfaces to be overlaid are tack coated at an application rate of 0.75 l/m^2, and that it is essential that the underlying pavement be sealed as thoroughly as possible to prevent delamination of the surface course and degradation of underlying pavement layers. They mention that longitudinal joints are not painted with an asphaltic material to prevent sealing and reducing drainage, and that the film thickness of the OGFC mix should be sufficient to ensure a satisfactory bond.

Bishop and Oliver indicate that OGFC mix using conventional asphalt binder should be produced at a temperature approximately 10°C lower than for conventional HMA. This reduces draindown of the asphalt component. This reduction in mixing temperature might not be required with polymer/rubber-modified asphalts because of their inherently higher viscosity. They mention that maximum compaction is achieved very easily because of the high stone content. A single pass with a non-vibrating 10-12 tonne steel roller is usually sufficient to compact the material. They also point out that handwork is quite difficult because of the lack of fines and rapid cooling due to the open graded material.

1.27.5 Maintenance Practices

Bishop and Oliver mention that surface temperatures on OGFC mixes could be lower than in conventional mixes and proper attention must be paid to prevent formation of black ice. They indicate that the critical temperature is 0-3 degree Celsius and that the additional attention and amount of salt needed for these mixes are more than offset by the benefits from these mixes.

Regarding drainage maintenance, Bishop and Oliver mention that OGFC mixes tend to self-flush the materials in the voids, and clogging is not a problem. However, they indicate problems do occur in maintaining transverse drainage in pavements with adjacent curb. These drainage paths need to be cleaned properly. The authors also mention the problem of water draining longitudinally to dense graded mixes, when the

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longitudinal slope is greater than the transverse slope. They mention that a solution is to cut a transverse butt joint to make the transverse flow rate greater than the longitudinal flow rate.

To maintain the functionality of OGFC mixes, Bishop and Oliver point out that it is necessary to seal slight raveling with light application of asphalt sealant, and that periodic monitoring of surface should be made to detect any signs of major loss of aggregate or potholes. If such a problem is detected, they suggest that an open graded asphalt patching mix be used as soon as possible to avoid further deterioration.

1.27.6 Rehabilitation Practices

Bishop and Oliver cite references and experiences to contend that life expectancies for OGFC pavements are about 12 years, compared to 14 to 15 years for conventional pavements. They mention that while riding surfaces will not be obtained by using methods such as hot in-place recycling on existing OGFC mixes, it is possible to use another OGFC layer as a rehabilitation method, provided the existing OGFC is sealed properly. They also mention that it is important to rehabilitate the entire pavement, including the shoulders to make sure that adequate drainage is ensured.

1.27.7 Performance

Bishop and Oliver mention results of performance evaluations conducted primarily on the two Nanaimo projects. Their observations are summarized in Table 67.

Table 67: Results of Performance Evaluations of Different Projects

Property	Performance of different projects
Overall Pavement Quality	The mix, including the longitudinal joints is performing well. Some asphalt stripping in the wheelpath has been noted, along with indications of underlying base problems in some other small areas.
Noise Levels	A reduction of 5db (30 percent reduction in total noise levels) was achieved in the first Nanaimo project.
Skid Resistance	An initial and one year skid resistance test with the British Pendulum tester indicated reduction of 10 percent and 20 percent in stopping distance, respectively.
Internal Drainage	No clogging and reduction in drainage has been noted.
Road Splash and Glare	Significant reduction (as obtained from visual observation) in spray and glare has been made possible with the use OGFC mixes.
Service Life	Experience indicates that these pavements will provide a reasonable service life with acceptable levels of noise reduction, enhanced skid resistance and superior drainage.

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1.27.8 Structural Design

Bishop and Oliver mention that currently BC Ministry of Transportation and Highways currently considers a structural strength value of 1.25 (in terms of Crushed Granular Equivalency) for OGFC, as compared to 2.0 for conventional asphalt pavement.

1.27.9 Limitations

Although not mentioned specifically, the authors caution against the use of OGFC mixes in areas which would not get early or close attention for winter maintenance because of their location or importance.

1.28 Bolzan, P. E., J. C. Nicholls, G. A. Huber. "Searching for Superior Performing Porous Asphalt Wearing Courses." TRB 2001 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2001.

1.28.1 General

This paper provides descriptions of design of porous asphalt mixes in the United Kingdom (UK), the United States (US) and Argentina. Field trials were described for the UK and Argentina, whereas results of a survey conducted in the US have been summarized. Detailed descriptions of materials, mix designs, equations relating lives with material properties and specifications were provided. The authors have stressed the use of two different performance "lives" of porous asphalt mixes – spray reduction life and structural life.

Bolzan et al mention that in the UK porous asphalts have been improved with the use of larger size aggregates, use of modified binders, gap-graded aggregate structures and larger air void contents. With respect to use in the US, they mention that European type mixes are gaining much wider acceptance, and that several states have been successful in adopting European concepts and preparing effective specifications. On the Argentinean experience, Bolzan et al provides information on several trials and resultant specifications on the design of porous asphalt mixes.

Bolzan et al concludes that for good design of porous asphalt mixes it is necessary to have a high binder content, with modification to prevent draindown, gap-graded aggregate structure, good compaction, use of durable aggregates, high air voids, good construction practices and an effective quality control/assurance system. They recommend that further research should be carried out to make use of the Superpave binder selection method, and to develop maintenance and rehabilitation techniques for extending the spray reduction life of porous asphalt pavements.

1.28.2 Benefits of Permeable Asphalt Mixtures

Bolzan et al mentions that the main reason for using porous asphalt is to enhance the driving environment, specifically by reducing noise in all conditions and spray in the wet condition. When used as a wearing course, Bolzan et al indicates that other benefits of permeable asphalt mixtures include reduction of noise in both wet and dry conditions, tire splash and spray in wet conditions, reflected glare at night in wet conditions and

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reduction in rolling resistance leading to improved fuel economy. They also mention that safety is improved through better tire-road grip, because there is less water on the road surface, increased capacity of roads in wet conditions by enhanced driver visibility, and because of greater texture depth of the road surface under dry conditions.

1.28.3 Materials and Design

Bolzan et al mention that there are records of 28 roads trials (with 83 sections) conducted between 1967 and 1991 in the UK (by the Transportation Research Laboratories, TRL). These trials have included a range of materials with aggregates having nominal maximum sizes of 20 mm, 14 mm and 10 mm, various grades of unmodified asphalt binders, binder contents between 3.2 % and 5.7 % and different polymer, fiber and other modifiers.

Bolzan et al contend that the durability of porous asphalt can be expressed in terms of the time for which the material effectively reduces spray during wet weather (the spray-reducing life) (or noise during all weather conditions) or the period over which it retains structural integrity (the ultimate life). Based on this contention, the authors have used actual lives or estimated service lives of the different sections to develop performance life versus factors models. Although wide variations have been observed in both spray reduction and ultimate lives, and although site conditions and standard of workmanship have been suspected to be important factors in many cases, the authors have managed to develop models to predict ultimate lives on the basis of important factors such as the aggregate source and size; the bitumen grade; the use of, and type of, binder modifier; and the proportions of these component materials.

Before presenting their models, the authors note that the ultimate and spray reducing life of the pavements which have been replaced are directly available, whereas for those that are still functioning after 12 year, estimates of remaining and total ultimate and spray reducing life were made. The total number of trial sections reported in this paper was 68 (or 89 if different lanes, for which separate data were available, are treated as separate sections). The authors also mention that estimation of the spray reducing life is a subjective process, and that their data does not include those from more recent commercial trials or for more developed designs.

Bolzan et al then provides a description of the progressive reduction in spray reducing and ultimate life or porous pavements. They mention that the ultimate life of porous asphalt in the road trials generally resulted from progressive binder hardening until the binder was no longer able to accommodate the strains induced by traffic. Brittle fracture was found to begin during winter and, if the surfacing survived a cold winter, it was usually found to remain serviceable during the following warmer months. They mention that an indication of imminent failure was often provided by the onset of raveling, particularly during the winter, which was usually accompanied by an increase in texture depth. Bolzan et al indicates that at this stage, core samples were taken from the surfacing and the condition of the binder was used to establish a failure criterion.

Based on the observation from the field performance and results of tests conducted on binder extracted from in-place cores, equations were developed relating ultimate and

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spray reducing lives to mix design variables. These equations and inferences are shown in Table 68.

Table 68: Performance Life Equations from UK Trials

Life	Equation	Inference
Ultimate	$Lu = -3.3 + 0.5\ 10^{-4}T + 0.4A + 1.4\ 10^{-3}P + M + 0.6B$, ($Radj^2 = 0{,}34$)	1. Traffic has only a small effect (beneficial for the ultimate life and detrimental for the spray-reducing life); 2. The use of larger nominal sizes of aggregate and of softer grades of bitumen are both beneficial; 3. The inclusion of modifiers appears to be beneficial over and above the increased binder content they permit; and 4. An increase in binder content extends the ultimate life but reduces the spray-reducing life.
Spray reduction	$Ls = 11 - 0.6\ 10^{-4}T + 0.2A + 3\ 10^{-3}P + 1.5M - 2.3B$, ($Radj^2 = 0{,}38$)	

Note: L_U = ultimate life (years), LS = spray-reducing life (years), T = traffic intensity (cv/l/d), A = nominal size of aggregate (mm), P = penetration of the base bitumen (mm/10), M = modified (1) or unmodified (0) binder, B = proportion of binder in the mixture by mass (%), R_{adj} = correlation coefficient adjusted for the degrees of freedom.

Bolzan et al provides a description of different materials used in the UK road trials, and some basis for the selection of such materials. These descriptions are shown in Table 69. Based on the field observations, the authors make the following conclusions:

1. For aggregates, 20 mm porous asphalt grading, particularly when using modified binders at higher binder contents, is an effective compromise that maximizes durability at the expense of some loss of hydraulic conductivity, and hence spray-reducing life.
2. The use of high penetration graded asphalt binders enhances durability, but at the expense of earlier closing-up of the surfacing. The extra durability is related to the time taken for the binder to harden to the critical condition, when it can no longer accommodate the traffic induced strains at low temperatures.

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Table 69: Use of Different Materials in the UK Road Trials

Material	Basis of use	Specification/Type used	Observation from field trials
Aggregate	1. Tire-induced stresses are applied to relatively few point-to-point contact areas between the essentially single-sized coarse aggregate skeleton; and, 2. relatively cubicle aggregates will provide good drainage and enhance the potential spray-reducing life.	UK *Specification for Highway Works* requires the coarse aggregate in porous asphalt to have: 1. A minimum 10 per cent fines value of 180 kN, 2. A maximum aggregate abrasion value of 12; 3. A maximum flakiness index of 25, 4. A minimum polished stone value dependent on the design traffic intensity, 5. A maximum aggregate impact value of 30 %; and 6. A minimum magnesium sulphate soundness value of 75. Aggregates used were strong aggregates with low flakiness index. The aggregates in porous asphalt are gap-graded such that they consist of coarse aggregate bound with a fine mortar. The proportion of aggregate in the smaller fractions needs to be restricted in order to avoid choking the porous asphalt and, hence, reducing the hydraulic conductivity. For 20 mm porous asphalt, the coarse aggregate is in the 20 mm to 14 mm fraction and the gap should be in the 10 mm to 6.3 mm fraction.	The average ultimate life of porous asphalt was: 5 years (with a range of 1 to 12 years) for 10 mm aggregate mixtures and 8 years (with a range 0 to 15 years) for 20 mm aggregate mixtures, The spray-reducing life of porous asphalt, which is not unrelated to the effectiveness of the surface in reducing noise, was 4 years (with a range 2½ to 6½ years) for 10 mm aggregate mixtures and 6½ years (with a range 2 to 8 years) for 20 mm aggregate mixtures.

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Table 69 Continued: Use of Different Materials in the UK Road Trials.

Material	Basis of use	Specification/Type used	Observation from field trials
Asphalt Binder	High binder contents improved the durability of porous asphalt by providing a thicker binder film, but they also reduced permeability by filling the pores. The maximum binder content is limited by the tendency of excess binder to drain from the mixture, which can result in areas of the finished mat which are either binder-rich or lean and lacking in fines; the binder-rich areas will have inadequate permeability while the binder-lean areas may be prone to premature raveling.	The test was first used on a series of trials in 1987 that showed that the most satisfactory target binder content, around 4.5 % for 20 mm nominal size aggregates, was difficult to achieve with unmodified binder without draindown. The penetration of the binder from the sections of porous asphalt in UK trials was monitored using samples taken from the mixing plant tanks and after recovery from the surfacings when laid together with recovered binder from cores taken at various intervals during the life of the surfacings. The results of binder recoveries are subject to several sources of error such as variation within the surfacings (sampling errors) and errors introduced by dissolution, recovery and testing. The errors are particularly evident for polymer-modified binders when the polymer is less than 100 % soluble in the solvent used to extract it. Also, there was evidence of polymer instability in sections from both trials.	1) There was general hardening with time; between mixing and laying, a typical reduction in penetration of 30 % was observed, although there were wide variations; 2) The hardening proceeded at about 20 % reduction in penetration per year; 3) Irrespective of the presence or type of modifier, the critical binder penetration was judged to be about 15 mm with the softening point generally close to 70 °C, after which failure generally occurred when sub-zero temperatures were next encountered; 4) The higher binder content materials showed a slightly lower hardening rate; 5) The presence of hydrated lime tended to lower the hardening rate; and 6) Binders with EVA tended to behave as harder binders with a consequential reduction in longevity. Predictions based on those findings apply to unmodified binders with binder contents of about 4 %(less than the more ideal target binder content of 4,5 % discussed above). The predicted ultimate life of porous asphalt with 100 pen bitumen is 7 to 8 years and that with 200 pen bitumen is 10 to 11 years. However, 100 pen bitumen is still preferable to 200 pen unmodified bitumen for heavily trafficked roads in order to minimize early closing up of the porous asphalt and hence loss of the desired spray- and noise reducing properties.
Modifier	Modifiers may be used to increase the binder content that can be incorporated into the mixture without the occurrence of binder draindown.	These modifiers include both polymer-modifiers and fibers, which increase the surface area over which the bitumen will be spread. Polymer-modifiers can be regarded as part of the binder whilst fibers modify the mixture rather than the bitumen. Polymer modifiers that have been tried include natural rubber, styrene-butadiene-styrene (SBS) block copolymer, ethylene vinyl acetate (EVA), epoxy resin and hydrated lime.	---

In describing current practice of using permeable asphalt mixes in the UK, Bolzan et al mention that the UK has finally accepted that permeable asphalt mixes can be durable, but has not pursued many projects with this mix. This is because the thin surfacings have gained a wider acceptance at this time, on the basis that they provide similar benefits as porous mixes at reduced cost and with better durability. This decision has also been affected by problems and need for replacement of porous mixes in two prominent projects.

Bolzan et al then provide a summary of the use of porous asphalt mixes in the US, with some insight into recent developments. Their descriptions are based on the results of a survey (published in a circular) conducted by the TRB Committee on Characteristics of Bituminous-Aggregate Combinations to Meet Surface Requirements, A2D03, in 1998. For the convenience of this review, the results, as shown by Bolzan, are summarized in Table 70.

Table 70: Summary of results from a Survey on the Use of Open-Graded Friction Course (OGFC), as Presented by Bolzan et al (based on response from forty-two states)

Topic	Results of survey
States using OGFC	Nineteen indicate that they use OGFC mixtures. Some agencies construct more than a thousand lane-km per year, others only a few. Seven agencies construct more than 300 lane-km per year. Another ten agencies routinely construct some open-graded mixtures each year. The remaining agencies have either discontinued use or have never used open-graded mixtures.
Materials and mixes used in states using large quantities of OGFC mixes	Florida uses only one gradation for OGFC. A modified binder is used that is composed of an AC30 bitumen with 12 % (by weight of binder) ground tire rubber. Several aggregates are allowed including crushed granite, blast furnace slag, crushed oolitic limestone (high friction limestone) and lightweight aggregate. Arizona DOT specifies two different OGFC mixtures, one for unmodified binder and the other for rubber modified binder. The bitumen for the unmodified mixture is PG 64-16. For the modified mixture, the base bitumen is PG 64-16 except in colder locations (at high altitude) where PG 58-22 is used. The binder is bitumen modified by the addition of 20 % (by weight of binder) of ground tire rubber. Arizona uses several criteria to specify acceptable aggregate including proportion of carbonate, crushed faces, flakiness index, Los Angeles abrasion, sand equivalent, water absorption and combined bulk specific gravity. Binder content is determined by a formula that depends upon aggregate water absorption, aggregate specific gravity and proportion passing the 2.36 mm sieve. A binder drainage test is not required for the rubber-modified porous asphalt mixture because the binder is very resistant to binder drainage.

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In showing examples of recent changes occurring to open-graded mixes in the US, Bolzan et al mention the following:
1. Modified binders are being used to reduce the potential of asphalt binder draindown during construction. They mention that modified binders also produce more durable mixes with less aging and potential for raveling.
2. Larger aggregates, with nominal maximum size of 12.5 mm and 16 mm are being used instead of previously used 9.5 mm, to provide larger voids, and hence reduce the chance of clogging.

In citing examples of use of this "new generation" type OGFC mixes, Bolzan et al provide a description of practices of the Georgia Department of Transportation (DOT) which was one of the first to adopt these new mixes. Georgia DOT distinguishes between the older and the new mix designs by labeling them as Open-Graded Friction Course (OGFC) and Porous European mix (PEM), respectively.

In describing the Georgia DOT practice, Bolzan et al dwell primarily on the use of modified binders and their benefits. They point out the following benefits of using modified binders.

1. Less susceptibility of draindown during construction and service. Draindown which is the separation of the binder/fine aggregate mastic from the coarse skeleton, can occur in a mixture storage silo or in the truck during transport. They mention that mixtures that have suffered from binder drainage produce binder rich areas on the road that have a flushed surface and no voids as well as other areas with little binder and high voids that quickly ravel.
2. Retention of thicker binder films. They mention that thick films of unmodified binder tend to drain downward with time during hot summer weather. The remaining thinned films on the surface particles age and become brittle more rapidly. When the binder becomes sufficiently brittle, aggregate particles are dislodged by traffic and the layer ravels. Modified binders retain film thickness, thereby, reducing aging and stone loss.

Bolzan et al also mention that PEMs have higher air voids than OGFC mixtures; this is important since continued benefit from an open-graded mixture is dependent on the void structure remaining open. If the voids are clogged with road debris and winter sands, the effectiveness of the OGFC mixes in draining water is reduced. Bolzan et al mentions that increasing the voids to 20 percent or more provides more resistance to clogging, larger voids tend to be cleaned by hydraulic action of traffic during rainfall, particularly on high-speed pavements.

The authors mention that Georgia DOT requires PEM mixtures to have polymer-modified binder and fiber stabilizing additives. PG 76-22 bitumen is typically modified with an SBS or linked SB polymer. Polish resistant, crushed aggregate is required. The mix design criteria include binder content, retained coating after boiling and resistance to binder draindown.

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Bolzan et al provide a comparison of porous mixes used in the US and in Europe, on the basis of gradation, air voids, aggregates and binders. This comparison has been summarized in Table 71.

Table 71: Comparison of US Mixtures to European Mixtures

Mix design/material	European practice	US practice
Gradation	European gradations allow for a more gap-graded mixture than North American mixtures, although not always	The Georgia specification for Porous European Mix is similar to the gradation specified in South Africa.
Air voids	All European agencies specify minimum air void contents	Only one US agency specifies minimum air void content; some US agencies do not compact specimens at all; Air void contents of the US mixtures tend to be considerably lower than European mixtures; Georgia DOT, who developed a specification patterned after the European approach, found permeability of their new mixture to be more than double that of conventional OGFC.
Aggregate	European agencies generally demand higher standards for aggregates than do US agencies. Los Angeles abrasion values are specified from 12 to 21 %.	For open-graded mixtures, US agencies specify 35 to 40 %
Binders	European agencies use modified binders almost exclusively	US agencies are shifting toward the use of modified binders.

In the next sections, Bolzan et al provided detailed descriptions of the use of porous asphalt mixes in Argentina. They present results of field trials, general experiences and conclusions and recommendations based on these experiences.

Bolzan et al mention that the use of porous asphalt in Argentina started as a result of a search for improvement of wet friction properties and reduction of the potential for hydroplaning. The first use was on a toll road in east central Argentina, connecting

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Buenos Aires with Mar del Plata. The design was based on European and US experience, with consideration of Spanish, Dutch and French specifications. Principal adoptions were the use of relatively large maximum aggregate size, larger voids and the use of modified binders. Details of the field trials and the experiences, as reported by Bolzan et al, are summarized in Table 72.

Table 72: Description of Full Scale Field Trials of Porous Asphalt Mixes in Argentina

Topic	Description
Quality control	Brookfield Rotational Viscometer (ASTM D 4402) and torsional elastic recovery test (Spanish Standard NLT 329) were adopted as quality control (QC) tools for the modified binder on site. The Spanish Cantabro Test (NLT 352) was employed as a mixture QC parameter. In-place hydraulic conductivity was measured during construction; a special specification was written based on the European experience and the full-scale trials. In the field, a detailed QC/QA plan was implemented.
Description	All trials were conducted on Road No. 2, which is a 400 km-length road that has two carriageways of two lanes each. The rainfall in the region is around 800 to 1000 mm per year; road has hard shoulders although it is not a highway because it has numerous crossings along it from the surrounding rural areas. These intersections are an inconvenience for the porous asphalt because many of the vehicles from the feeder roads carry soil that can clog the voids in the material.
Time of construction	The first full-scale trial was laid down in 1997 on a 300 m-length section with two layers of porous asphalt, one placed as the base layer and the second as the wearing course. The success of the trial was followed by the construction of an entire 22 km-length section (2 lanes on both carriageways). A year later, a 100 km section was constructed divided into four sections. A total of more than 100,000 tonnes were placed in this project in about two years. Following this project, other roads within Buenos Aires Province followed the trend giving a total of more than 170,000 tonnes in a little over two years.
Binders	The modified binders used were of two types, EVA-modified and SBS-modified, with different levels of modification that, in terms of torsional elastic recovery at 25 °C, ranged between 35 and 75 % of its original shape; Two levels of modification, based on the torsional elastic recovery test, were set at not less than 40 % and 70 %.
Aggregate	Aggregates comprised of granite from three locations in the south-west of Buenos Aires Province: Olavarria, Tandiland, Balcarce
Structure	The layer thickness in all cases was 40 mm and the maximum particle size was mostly 19 mm. Other trials have been undertaken in order to compare binder types, different aggregate gradations and two thicknesses (40 and 50 mm). A twin-layer porous asphalt was constructed along one kilometre length using two different formulas. In this twin-layer system, a 20 mm thick layer with 12 mm maximum aggregate size was placed on top of a 40 mm thick layer with 19 mm maximum aggregate. Both layers were design and controlled separately.

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Table 72 Continued: Description of Full Scale Field Trials of Porous Asphalt Mixes in Argentina.

Topic	Description
Mix Design	The Cantabro test (NLT 352/86) was adopted in the two modes: dry (6 hours at 25 °C, 300 revolutions) and wet (24 h at 60 °C, test at 25 °C, 300 revolutions); the binder drainage test was adopted from the UK; first trial was conducted according to the Spanish specifications, with two aggregate gradation bands, a modified binder with up to 15 % elastic recovery, Portland cement as a filler, a 12 mm aggregate maximum size and an EVA modified binder. Later trials were conducted with SBS modified binders, hydrated lime as filler (2.5 to 3 %) and a 19 mm aggregate maximum size. The layer thickness was to be 40 mm. The mixture was composed of 86 % coarse aggregates (6 to 20 mm), 11 % fine aggregates (0 to 6 mm) and 3 % hydrated lime. The binder content was established at 4.5 % of the total weight of the mixture by performing both Wet and Dry Cantabro tests and the binder drainage test. The total voids content in the mixture was set at 23 %. The in-situ total air voids content ranged from 18 to 25 %. In the case of the 12 mm maximum aggregate size mixture, the composition was 80 % coarse aggregate (6 to 12 mm), 17 % fine aggregate (0 to 6 mm) and 3 % hydrated lime. The binder content was fixed at 5.0 % by weight of the total mixture. The Cantabro (25 °C, 300 revolutions) and Indirect Tensile (25 °C, 50 mm / min) tests were performed on all the mixtures both on plant-prepared Marshall briquettes and on cores taken from the pavement. The Cantabro test values at 25 °C were in the range of 10 to 17 % for plant produced mixture and between 15 and 20 % for the cores. The acceptance criteria were maximum values of 25 % in the dry condition and 35 % in the wet condition test. The wet condition test was conducted every 3,000 tonnes of plant produced material with the results showing an average 22 % abrasion loss. The Indirect Tensile test was performed mainly as a mean to verify the uniformity of the cores extracted from the road. Thus, every core was subjected to the Indirect Tensile test and the results showed a resistance in the range of 0.5 to 0.6 MPa. Prior to testing, the specimens were analyzed for voids content, thickness, density and exterior appearance.
Aggregate	Los Angeles Abrasion test values were between 20 and 23 % for gradation B (ASTM C 131). No material was allowed with Los Angeles values higher than 25%. All the particles were 100 % crushed, with a Flakiness Index not greater than 25 %, and were binder compatible (there was no need of an anti-stripping additive). The finer gradation had a Sand Equivalent Test value of 70 %. Accelerated Polishing Stone Value higher than 45 were adopted from the Spanish recommendations, although the British require values nearing 60. However, local granites can only reach 48 to 50 PSV.
Gradation	The first trial was performed using a gradation envelope following the Spanish PA12 gradation band. Later it was adjusted to a different gradation. -- PA12 Sieve mm 19.0 12.5 9.5 4.75 2.36 0.63 0.075 Percent Passing 100 70-100 50-80 15-30 10-22 6-13 3-6 Revised Percent Passing 100 70-90 50-80 15-20 10-18 6-13 3-5 -- For the mixture to be efficient at allowing water to drain through it the proportion passing the No.4 sieve was kept in the range of 15 to 19 % and the difference between sieves No.4 and No. 8 was kept at less than 8 %. The material passing 74 microns was kept at 4 %, and a 19 mm maximum size was adopted in two out of the three projects.

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Table 72 Continued: Description of Full Scale Field Trials of Porous Asphalt Mixes in Argentina

Permeability	Of the different types of available falling head permeameters, There are several types and Argentina adopted the LCS type, standardized by the Spanish in NLT-327/88, for measurement of hydraulic conductivity. This piece of equipment works by percolating water through the material while taking two readings as the water goes down. Once the reading in seconds is obtained, it can be converted into the air voids content in the mixture through the following equations: Ln (K) = 7.624 – 1.348 Ln (T), Ln (H) = 4.071 – 0.305 Ln (T) where: *K* = the permeability coefficient (cm/s 10^2), *H* = the proportion of air voids in the mixture (%), *T* = the evacuation time (s); correlation was developed on the basis of mixes containing between 3.5 and 5.5 % modified binder, maximum aggregate size of 10 to 12 mm, between 10 and 15 % of aggregate passing the No.8 sieve, having between 2 and 6 % of aggregate passing the No.80 sieve. The total air voids content measured in the projects ranged from 18 % to 25 % with the mean value between 21 and 23 %.

Bolzan et al mention that based on the first project and European experience, a special specification was developed around mid 1998. Early in 1999, before the start of the HighwayEzeiza-Cañuelas (near the international airport) project, a modified version of the original specification was made that incorporated recent experience, more user-friendly modified-binder specifications and tighter tolerances. The authors mention that the tighter tolerances were on the binder content, the aggregate gradations, the volumetric properties and the filler content variations during construction. Also, additional requirements were placed on the construction of the asphalt base course because this layer has to have enhanced properties of in-place voids content, fatigue resistance properties and rut resistance characteristics.

Bolzan et al discuss some structural considerations of porous asphalt mix layers in their paper. On the basis of the facts that porous asphalt is a mixture in which fractions of the aggregate grading are absent, they contend that porous asphalt mixtures have lower strength than dense-graded mixtures. They mention that some researchers accept that these mixtures have up to 70 % of the strength of a conventional mixture; others indicate the ratio is only 50 % whilst the Spanish believe that they are structurally equivalent with conventional dense-graded asphalt mixtures. They are also considered to be less shear stress resistant. Bolzan et al indicates that Argentina adopted a 50 % structural capacity for porous asphalt mixtures in the initial projects. The resilient modulus (ASTM D 4123) at 25 °C and 10 Hz of porous asphalt mixtures, prepared in the laboratory of Argentina, was found to be about 2200 MPa, approximately 60 % of the conventional mixtures. However, Bolzan et al point out that at both higher and lower temperatures, polymer-modified porous asphalt mixes perform better than unmodified conventional mixes and that further research needs to be conducted to reach at definitive conclusions.

In discussing the quality control and quality assurance aspects in construction of porous asphalt mix courses, Bolzan et al mention that a QC/QA system was developed and implemented. The system comprised of control testing by the contractor, quality assurance testing by the owner; and independent assurance sampling and testing to measure resilient modulus of the mixture and initial friction properties. Plant controls and

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checks consisted of all the operations related to the control of the quality of the incoming materials (binder, aggregates, filler, etc), periodic controls during the construction stage and testing of the end product. The key components of the program were:
1. Volumetric Quality Control, 2. Materials Quality Certification, 3. Technical Evaluation through Data Analysis; and 4. Acceptance based on end product specification.

Bolzan et al indicates that the specifications are based on Marshall density, relative hydraulic conductivity, total in-situ air voids content, binder content, surface tolerance and texture depth. The main goal is to obtain sufficient interconnected air voids in order to provide enough relative hydraulic conductivity and to maintain a high pavement surface texture depth. Non-destructive testing employing two Falling Weight Deflectometers (FWD) were carried out to evaluate the pavement structural capacity achieved as part of the QC/QA system. In the Ezeiza-Cañuelas Highway project, a Kuab FWD was used for the first 24 km and a Dynatest FWD for the second 12 km length. A special testing protocol was written specifying measurements on the outside wheel path of the four lanes at 50 and 100 m intervals applying a 40 kN load. Based on empirical measurements and the estimated future traffic, a maximum initial deflection of 500 microns was set. All the individual maximum deflection values measured were lower than the specified value.

In discussing durability expectations, Bolzan et al points out that the spray or the functional life ends with the clogging of interconnected voids, whereas the ultimate or structural life ends with raveling. The use of polymer modified binder and hydrated lime were made to optimize the aggregate-binder bond. They mention that the functional life has been estimated as being between 3 and 4 years, and the structural life has been assumed to be 8 years.

1.28.4 Construction Practices
No information on construction practices has been presented.

1.28.5 Maintenance Practices
No information is provided on maintenance practices of friction course.

1.28.6 Rehabilitation Practices
No information is provided on rehabilitation practices of friction course.

1.28.7 Performance
Relevant information on performance has been presented in the mix design section.

1.28.8 Structural Design
Relevant information on structural design has been presented in the mix design section.

1.28.9 Limitations
Although not labeled as limitations, Bolzan et al provide several disadvantages of using porous asphalts in their opening paragraph. They mention that these disadvantages include increased costs, relatively low structural strength, due to its high void content,

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possibly shorter service life, complications to winter maintenance procedures; and, maintenance patching difficulties, susceptibility to high stress sites, and requirement of minimizing the drainage path length to allow water passing through the layer to enter the drainage system.

1.29 Corrigan, S., K. W. Lee and S. A. Cardi. "Implementation and Evaluation of Traffic Marking Recesses for Application of Thermoplastic Pavement Markings on Modified Open Graded Friction Course." TRB 2001 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2001.

1.29.1 General

This paper describes a study carried out to develop a specification for longer lasting and better performing thermoplastic pavement marking materials on "modified" open-graded friction courses (OGFC). The authors indicate the durability problems that result from the use of snow plows on these markings on OGFC mixes, and then describe three different techniques used for enhancing the durability.

Corrigan et al indicates that the new technique consisted of applying thermoplastic pavement marking on recessed and semi-recessed areas of OGFC, and comparing their performance with that of markings on non-recessed areas. The durability was measured at different times through several years, in terms of markings retained, and the performance was measured in terms of retro reflectivity, under dry and wet night conditions.

The authors mention that even though the non-recessed conventional inlaid markings were found to have a higher retro reflectivity compared to the recessed markings, the higher snow plow damage of the non-recessed markings, especially in curved sections, warrant the use of recessed markings. Based on durability, the life cycle cost of the recessed markings was found to be lower than that of the non-recessed markings. However, more research is needed on modifying the properties of the marking materials, such as size of glass beads, to increase the retro reflectivity of the recessed markings to acceptable levels.

1.29.2 Benefits of Permeable Asphalt Mixtures

No information on benefits of porous asphalt mixtures has been provided.

1.29.3 Materials and Design

No information on materials and design of porous asphalt mixtures has been provided.

1.29.4 Construction Practices

Corrigan et al mentions that snow plow damage of thermoplastic pavement marking strips on OGFCs has been a significant problem in Rhode Island. The heated theromoplastic materials (at 400-440^{o}F), when applied on the porous OGFC, penetrate about 1/8th inch into the surface and forms a strong bond with the OGFC materials. During snow removal in winter maintenance period, plows hit the pavement material and scrape off part of the OGFC as well as the pavement markings. The authors argue that if

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the markings are applied over a constructed recess, along the surface, then the snow plows would pass over the recess and not cause any damage to the marking and the pavement materials.

Corrigan et al mentions that although there have been studies with recessed markings, snow plow resistant markings and inlaid preformed tapes, these studies have produced mixed results and that no study has been conducted with modified OGFC.

In the study reported in this paper, based on a newly developed specification and application methods, recessed markings were used in OGFCs on a 1,000 ft tangent section, a 500 ft exit ramp section, and a 500 ft curved section. Each test section included three types of recesses and a non-recessed control. The authors mention that in addition to the durability and performance comparisons, a study of cost associated with equipment and labor to produce the recessed markings as well as a life cycle cost study were conducted. The details of construction of the test sections are summarized in Table 73.

Table 73: Details of Test Sections with Different Markings

Condition	Marking	Comments
Sunny, 48^{o}F	A gasoline powered pavement cutter equipped with a 6-inch carbide tipped blade was used to create the 6-inch wide traffic marking recesses. The debris was removed from the finished traffic marking recesses with a gasoline-powered blower. The white Alkyd thermoplastic skip stripes were applied to the traffic marking recesses with a small portable thermoplastic applicator. The molten thermoplastic was loaded from the Vulcan melting kettles into the portable thermoplastic applicator's storage reservoir (kept heated at 400 to 440^{o}F). The thermoplastic was then applied to the traffic marking recesses though an extrusion die. The glass beads were uniformly applied onto the extruded markings by a gravity drop-on glass bead dispenser, located just behind the extrusion die.	Drops of moisture were observed on the backing of the removed tabs even though it had been 4 days since the last precipitation had occurred. The region received the first significant snowfall of the season the day after the installation. This was the first time permanent inlaid marking tape has been used as traffic marking skip stripes on Modified OGFC in Rhode Island.

1.29.5 Maintenance Practices

No information on maintenance practices of porous asphalt mixtures has been provided.

1.29.6 Rehabilitation Practices

No information on rehabilitation of porous asphalt mixtures has been provided.

1.29.7 Performance

Corrigan et al mentions that the performance of the different sections was evaluated on the basis of percentage retained after a certain time and the retro reflectivity numbers

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under dry and wet night conditions. The details of testing procedures are summarized in Table 74.

Table 74: Testing of Durability and Retro Reflectivity for Recessed and Non-Recessed Pavement Markings

Property	Test Method
Durability	Percentage retained method, defined as the nominal area of the marking minus the area of loss, divided by the nominal area, and multiplied by 100. Durability evaluations for the stripes within the 1,000 ft tangent test section were conducted on December 5, 1996, December 30, 1996, April 7, 1997, July 30, 1997, November 19, 1997, July 2, 1998, December 7, 1998, and June 30, 1999. The average durability for each type of recessed traffic marking was determined by averaging the percent retained of the six skip stripes within each grouping category. The evaluations within the 500 ft exit ramp test section were performed on December 5, 1996, December 30, 1996, April 7, 1997, July 30, 1997, November 19, 1997, July 2, 1998, December 7, 1998, and June 30, 1999. The evaluations within the 500 ft curved test section were conducted on December 5, 1996, January 6, 1997, April 9, 1997, July 30, 1997, November 19, 1997, July 2, 1998, December 7, 1998, and June 30, 1999.
Retro reflectivity	The Retroflex 1500 pavement marking retroreflectometer with a 30 meter fixed geometry was chosen to evaluate the retroreflectivity of the traffic markings. A total of four retroreflectivity measurements were taken on each skip strip, two measurements 3 ft from the leading edge and two measurements 6 ft from the leading edge. These measurements were then averaged to obtain a single representative retro reflectivity value for each stripe. Three evaluations were conducted within the 1,000 ft tangent test section during dry daylight conditions. Two evaluations were conducted during dry night conditions. The retro reflectivity was determined by averaging the measurements of the six skip stripes; The July 30, 1997 evaluation was conducted at night with simulated wet conditions. The retro reflectivity of the traffic markings during wet conditions was simulated by uniformly pouring water over the traffic marking stripes. The retro reflectivity readings were then taken approximately 60 seconds after the initial wetting of the stripe. The December 7, 1998 evaluation was conducted during a night with light scattered showers that tapered off as the wet night evaluation progressed. The retro reflectivity evaluations for the stripes within the 500 ft exit ramp test section were performed on December 30, 1996, April 9, 1997, and November 19, 1997 during dry daylight conditions. Dry night evaluations were performed on July 2, 1998 and June 30, 1999. The reflectivity for each type of recessed traffic marking was determined by averaging the readings of the four skip stripes within each grouping category (three skip stripes for the non-recessed). The evaluations within the 500 ft curved test section were conducted on Jan 6, 1997, April 9, 1997, and November 19, 1997 during dry daylight conditions. Dry night evaluations were performed on July 2, 1998 and June 30, 1999.

Corrigan et al mentions that a significant amount of reduction in retro reflectivity was recorded for all different types of marking within the first three years of winter. The results are summarized in Table 75.

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Table 75: Results of Durability and Retroreflectivity Evaluations

Time Tested	Section	Fully Recessed	Semi Recessed	Tapered Recessed	Non-Recessed	Permanent Inlaid Marking Tape
		Durability (wear), average percent retained — Types of markings				
Over three winters	1,000 ft, high speed lane	> 97%	> 97%	> 97%	95%	50.8%*; 68.3%**
	1,000 ft middle lane	> 98%	> 98%	> 98%	---	---
	500 ft ramp	> 97.0%	> 97.0%	> 97.0%	88%	---
	500 ft curve	> 97.0%*	> 97.0%*	> 97.0%*	---	31.2%*, 42.5 % **; 81%***
		Retroreflectivity, mcd/m^2.lux				
Initial	1,000 ft, high speed lane	415	395	406	413	593
	1,000 ft middle lane	402	388	427	391	649
	500 ft ramp	358	303	285	199	---
	500 ft curve	329	316	324	333	205
After three years, reduction	1,000 ft, high speed lane	125, 70%	117, 70%	128, 69%	125, 70%	---, 80%
	1,000 ft middle lane	119, 70%	106, 73%	115, 73%	124, 68%	134, 79%
	500 ft ramp	90, 75%	96, 68%	107, 62%	148, 26%	---
	500 ft curve	112, 66%,	108, 66%	107, 67%	103, 69%	143, 30%

* at the end of the first winter maintenance season, ** at the end of one snowplowing, *** at the end of third winter maintenance season (after repair after the first winter maintenance season)

In general, Corrigan et al note that the durability of recessed markings is better than non recessed and permanent inlaid marks. In terms of retro reflectivity, the authors mention that significant reduction was noted within the first three winter maintenance periods, and that the retro reflectivity readings, both initial and reduced, are found to be higher for the non recessed marks.

In discussing the results of statistical analysis of retro reflectivity results from different markings, Corrigan et al makes comparison between non recessed and recessed sections

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in terms of wet night and simulated wet night retroreflectivity. Their comparisons are summarized in Table 76.

Table 76: Inferences from Statistical Analysis of Retro Reflectivity Results from Different Sections, for Different Conditions

Lane	Condition		
	Dry	Wet night	Simulated wet night
1000 ft, Fast	Wear in fully recessed same as the wear in non-recessed markings.	Retro reflectivity of all marks were substantially reduced	Retro reflectivity of all marks were substantially reduced
1000 ft, Middle	Wear in recessed was less than wear in non-recessed markings.	Retro reflectivity of all marks were substantially reduced	Retro reflectivity of all marks were substantially reduced
500 ft ramp	Wear in fully recessed same as the wear in non-recessed markings.	Retroreflectivity of all marks were substantially reduced	Retroreflectivity of all marks were substantially reduced
500 ft curve	----	---	Retroreflectivity of all marks were substantially reduced

Note: Wear is indicated by a reduction in retro reflectivity

Based on the increase in durability of recessed markings, the authors have presented information on additional cost incurred for equipment and labor needed for creating the recesses and an evaluation of life cycle cost. They report that the cost of making one linear foot of recess is about $0.20, and the cost of placing the thermoplastic material is $0.53 per linear foot (hence a total cost of $0.73 per linear foot). On the basis of consideration of a 3 year life span for a non recessed mark and a six year life for a recessed mark, the authors show that the life cycle cost of thermoplastics material with recessed markings is about $15,000, compared to about $19,000 for similar materials in non recessed markings.

1.29.8 Structural Design
No information on structural design of porous asphalt mixtures has been provided.

1.29.9 Limitations
No information on limitations of porous asphalt mixtures has been provided.

1.30 "Performance Characteristics of Open-Graded Friction Courses." Massachusetts Highway Department, Pavement Management Section. Boston, MA. February 15, 2001.

1.30.1 General
This paper outlines the Massachusetts Highways Department's (MHD) experience with Open Graded Friction Course (OGFC) up to 2001. This paper was written to address internal concerns within MHD in regards to the effectiveness and performance of OGFC in Massachusetts.

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The paper addresses construction issues, maintenance practices, and rehabilitation techniques that are used in Massachusetts. Data from a comprehensive skid resistance study was presented that confirmed that OGFC pavements provide excellent friction. Accident history data was presented for 1998 for both conventional and OGFC sections, to illustrate the added safety benefits of OGFC. The data showed that the wet/dry accident ratio is 13 percent lower on the OGFC sections as compared to typical dense-graded sections.

Finally MHD states that OGFC pavements are performing well in Massachusetts with some sections that are 19 years old that are still performing adequately.

1.30.2 Benefits of Permeable Asphalt Mixtures

MHD states the following benefits of OGFC as: reduced hydroplaning, increased wet weather friction, reduced splash and spray, reduced wet weather glare, reduced noise, and improved night visibility of pavement markings during wet conditions.

1.30.3 Materials and Design

MHD suggests using fibers or polymer-modified binders to allow the OGFC mix temperature to be increased without additional draindown concerns and would allow for more nighttime paving operations.

1.30.4 Construction Practices

MHD recommends the following:

1. OGFC should be placed on structurally sound pavement.
2. OGFC should not be relied upon to correct structural problems.
3. OGFC is difficult to feather or taper, thus transition areas should be cut in the old pavement and the OGFC started full depth.
4. Over rolling or rolling over cooled mix can crush the aggregates. Rolling should be done behind the paver, with one or two passes with a medium steel wheel roller.
5. The maximum temperature of the OGFC should not exceed 250°F when a non-modified binder is used. Temperatures in excess of 15°F above the maximum can lead to draindown and fat spots.
6. OGFC should not be placed on projects with heavy vehicle braking, turning, merging or traffic weaving. These areas are prone to debond and pavement delamination.
7. OGFC should be placed when the ambient temperature is 60°F and rising.
8. OGFC should be placed in 19 to 25mm lifts.

1.30.5 Maintenance Practices

MHD states that maintenance of OGFC presents problems due to its open surface texture and high voids. Winter maintenance is also of importance because the OGFC pavement temperatures tend to be lower than dense-graded mix.

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Localized repairs can be made to correct small surface defects by patching with a dense-graded mix.

1.30.6 Rehabilitation Practices

MHD offers the following rehabilitation techniques:

1. The most desired practice is to mill out the OGFC to a depth of 63.5mm and replace it with 45mm of dense binder and 19mm of OGFC.
2. The second option is to mill the OGFC to the top of the dense binder and replace it with 9.5mm or less of surface treatment.
3. The final option is to micro-mill the OGFC to the top of the dense binder.

1.30.7 Performance

MHD paving policy is to place OGFC on all high speed, high volume roadways.

1.30.8 Structural Design

MHD did not discuss structural design.

1.30.9 Limitations

MHD recognizes the following limitations of OGFC:

1. OGFC can be prone to premature raveling and weathering due to oxidation and hardening of the binder.
2. Application of thermoplastic paint markings can heat up the OGFC surface and cause localized draindown of the binder material from the aggregate. This can lead to delamination of the OGFC and /or raveling of the mix under the thermoplastic line markings.
3. Snow plows can hit off raised markers and bounce along the OGFC surface causing a "chatter" or plow marks in the surface of the OGFC.
4. Primary causes for OGFC failure were raveling and delamination.

1.31 Milne, R. "Open-Graded Comes Clean." Asphalt Review. Australian Asphalt Pavement Association. Volume 20, Number 3. pp. 11-12. December 2001.

1.31.1 General

This article provides a description of equipment used in New Zealand to clean open-graded porous asphalt.

1.31.2 Benefits of Permeable Asphalt Mixtures

No specific benefits were given.

1.31.3 Materials and Mix Design

No specifics on materials and mix design were given.

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1.31.4 Construction Practices
No specifics on construction practices were given.

1.31.5 Maintenance Practices
This article briefly describes a truck mounted equipment that utilizes high pressure water and a suction system used to clean open-graded porous asphalt pavements. The system uses multiple high speed rotating water nozzles to apply a high water volume at a high pressure to dislodge debris within the pavement. A double suction bar is then used to remove the water and debris from the pavement surface. All processes are computer controlled which allows the operator to control water pressure and volume. The article states that New Zealand has been using this process for approximately 6 months with "substantial success."

1.31.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.31.7 Performance
No specific performance measures were given; however, the article did state that cleaning was needed on open-graded porous asphalt pavements at an age ranging between 6 and 8 years.

1.31.8 Structural Design
No specifics on inclusion within structural design were given.

1.31.9 Limitations
No limitations on use were given.

1.32 Momm, L. and E. M. Filho. "Study of the Aggregate for the Pervious Asphalt Concrete." 2nd International Symposium on Maintenance and Rehabilitations of Pavements and Technological Control. Auburn, Alabama. July 29-August 1, 2001.

1.32.1 General
This paper presents the results of a laboratory study conducted to evaluate the effect of aggregate gradation on durability, permeability and rutting. The effect of gradation was evaluated as both maximum aggregate size and various aggregate sizes that were gapped to create the desired gradation (e.g., the breakpoint sieve was altered).

1.32.2 Benefits of Permeable Asphalt Mixtures
No specific benefits were given.

1.32.3 Materials and Mix Design
Contents of this paper do not specifically deal with materials or mix design; rather the paper provides research results that show the influence of aggregate gradation characteristics on laboratory performance. Therefore, the results presented in the paper are discussed within this section.

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Aggregates used within this project were granite. No physical properties of the granite were provided other than it was crushed. A viscosity graded AC-20 that had been modified with 4 percent of a SBS polymer was used as the asphalt binder.

Three maximum aggregate size (MAS) gradations were evaluated during the study: 9.5, 12.5 and 19mm. Evaluation of the effect of gradation on durability, permeability and rutting entailed three separate experiments. In the first experiment, a total of six gradations were fabricated: 9.5mm MAS with gap beginning with the 2mm sieve, 12.5mm MAS with gaps beginning at the 2 and 4mm sieves and 19mm MAS having gaps beginning at 2, 4 and 6mm. Each of the gradations had 3 to 5 percent passing the 0.075mm sieve. The six gradations were combined with asphalt binder at contents ranging from 3.6 to 5.2 percent. The paper does not provide information on how samples were compacted in the laboratory; however, the paper does mention Marshall samples. Two tests were conducted on the samples, a Cantabro Abrasion test and a test to determine the volume of interconnected voids. The test to determine the interconnected voids content was described as a test that measures the volume of water that penetrates into a test specimen.

Results from this first experiment indicated that the percentage of interconnected voids within the samples was considered by the authors as low. Typical ranges were approximately 5 percent interconnected voids and lower. There was a trend toward higher percentages of interconnected voids for larger maximum aggregate size gradations and lower asphalt contents. Cantabro Abrasion results were also deemed low by the authors. All results were below 14 percent. There was a trend toward higher loss values as the maximum aggregate size increased and asphalt binder contents decreased.

Using the information obtained from the first experiment, the authors fabricated three additional gradations for a second experiment. For these three gradations, the aggregates were essentially single size from the maximum aggregate size to 7.7 percent passing to create an open-grading. Each of the three gradations was combined with asphalt binder to yield binder contents of 3.6, 4.0 and 4.4 percent. Each mix was again evaluated for the percentage of interconnecting voids and Cantabro Abrasion; however, these mixes were also subjected to rut testing in the LCPC wheel tracking device and permeability testing.

Cantabro loss values for these mixes were higher than from the first experiment. Results ranged from 10 percent to 29 percent loss. Similar to the first experiment, Cantabro loss values tend to increase for larger maximum aggregate size gradations and lower asphalt binder contents. The percentage of interconnected voids was higher for these open gradations of the second experiment. Results ranged from about 20 percent interconnected voids to 12 percent. Again, larger maximum aggregate size gradations and lower asphalt binder contents generally resulted in more interconnected voids. Accordingly, permeability values were much higher for larger maximum aggregate size gradations [the influence of asphalt binder content was not presented].

Results of the rutting tests from the LCPC wheel tracking device were similar for all three maximum aggregate size gradations. All three were shown to be resistant to rutting.

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The third experiment again included three gradations representing the three maximum aggregate sizes. All three were again open-graded the only difference being that the three gradations became similar at 10 percent passing rather than 7.7 percent. [In essence, there was very little difference between the second and third experiment gradations.] All three gradations were used to prepare mix at 4 percent asphalt binder. Again, Cantabro Abrasion, rutting, permeability and interconnecting voids were evaluated. Similar relationships between maximum aggregate seize and the responses were observed during the third experiment.

1.32.4 Construction Practices
No specifics on construction practices were given.

1.32.5 Maintenance Practices
No specifics on maintenance practices were given.

1.32.6 Rehabilitation Practices
No specifics on rehabilitation practices were given.

1.32.7 Performance
No specific performance measures were given.

1.32.8 Structural Design
No specifics on inclusion within structural design were given.

1.32.9 Limitations
No specific limitations were given.

1.33 Moore, L. M. and R. G Hicks. "Design, Construction, and Maintenance Guidelines for Porous Asphalt Pavements." Transportation Research Record No: 1778. Transportation Research Board. National Research Council. Washington, D.C. 2001.

1.33.1 General
This paper presents an overview of the use of porous pavements – historical perspective, design, construction, quality control and maintenance practices in Oregon. Moore et al mentions that since the 1970s porous asphalt mixes have shown good performance and, hence, this type of mix continues to be a preferred mix in Oregon. They gave a detailed description of performance evaluations, mix design and construction criteria that have been developed for open graded asphalt mixes in Oregon.

The authors indicate significant benefits of open-graded asphalt mixes, and special considerations that need to be made for design, construction, maintenance and rehabilitation of pavements with these mixes. Major mix design considerations include specifications on retained strength, compaction air voids and draindown, whereas principal construction considerations include the use of appropriate mixing temperatures and compaction equipment. Moore et al provide a description of quality control

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considerations, which include putting more emphasis on asphalt content and percent passing the 0.075 mm sieve and use of these considerations for development of pay factors.

In terms of structural design and limitations of use Moore et al mention the specification of minimum depths, repair of existing surfaces prior to the placement of open-graded asphalt mixes, and the recommendations on not using the open graded mixes on low volume rods, roads which need much hand work and roads with heavy snowfalls where steel plows are used.

The paper indicates that the Oregon Department of Transportation (ODOT) has developed specifications on mix design and construction of open-graded mixes, on the basis of test sections, research projects and surveys. Although adequate recommendations have been developed for addressing maintenance problems, it seems that the issue of adopting a proper rehabilitation methodology is still being researched (at the time of publication of this paper).

1.33.2 Benefits of Permeable Asphalt Mixtures
In discussing the reasons for using porous asphalt mixes in Oregon, Moore et al mention the following benefits: safety through improved high speed frictional properties and reduced splash and spray, user comfort during winter driving, quieter roads and similar performance as dense-graded mixes historically used for wearing surfaces.

1.33.3 Materials and Design
Moore et al mention that in Oregon, porous asphalt mixes, called open-graded asphalt concrete, is characterized by the use of a large percentage of coarse aggregate in the mix without a significant portion of fines, as found in dense graded mixes. They mention that of the different classes of asphalt concrete (class B through F), E and F are open-graded, whereas, the rest are dense graded. Relevant information provided on these two classes of mix is summarized in Table 77.

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Table 77: Use and Characteristic of Open-Graded Asphalt Concrete

Class of Mix	Primary Use	Nominal Maximum Aggregate Size	Gradation
E	Nonstructural thin overlays (25 mm) to improve skid and hydroplaning resistance.	19 mm	Not provided
F	Thin overlays (50 mm) and for wearing courses for new pavement construction or structural overlays on all highways up to 100 mm; being recommended for use on many Oregon roadways, including Interstate highways.	25 mm	a) 1984 Sieve Size, mm — % passing 25 — 99-100 19 — 85-96 12.5 — 60-71 6.3 — 17-31 2 — 7-19 0.43 — ---- 0.08 — 1-6 Mineral Filler — 0.0-1.5 Asphalt Cement --- ---------------------- a) 1998 Sieve Size, mm — % passing 25 — 99-100 19 — 85-96 12.5 — 55-71 6.3 — 15-30 2 — 5-15 0.43 — ---- 0.08 — 1-6 Asphalt Cement — 4-8

Moore et al mention that early mix designs were carried out following the Hveem mix design procedure, and the mix design procedures for open-graded mixes were adopted from mix design procedures from B/C class mixes, by considering the facts that the open-graded mixes have higher voids and they have lower strength and stability when tested in an unconfined mode.

Moore et al state that in 1992, Oregon DOT (ODOT) changed the mix design procedure to include a draindown test and criteria, procedure for determination of bulk specific gravity of compacted specimen from saturated surface dry method to geometric method and specification of modified binder. The new and old ODOT specifications are shown in Table 78.

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Table 78: ODOT Specifications for Open-Graded Asphalt Mix

Criteria	Criteria in 1986	Criteria in 1999
1. Asphalt Film Thickness		
2. Design Air Voids % (DAV) a) 1st Compaction b) 2nd Compaction (min.)	11-13 8	13.5-16.0 n/a
3. Hveem Stability, minimum a) 1st Compaction b) 2nd Compaction	26 30	n/a n/a
4. IRS @ DAV, minimum	75	80*
5. Draindown	n/a	70-80
6. Voids Filled with Asphalt (VFA)	n/a	

Note: These mixes used to be designed with 1% Portland cement as mineral filler to stiffen the hot asphalt during transportation and laydown. The recent switch to the use of PBA-5 and PBA-6 asphalts has eliminated the need for mineral filler.
* TSR on surrogate dense-graded mix.

1.33.4 Construction Practices

Moore et al indicate that standard drum or batch plants are used for producing the open-graded asphalt mix and that conventional paving equipment, with some minor adjustments in construction practices, are used for laydown and compaction in Oregon. They mention that for these types of mixes, draindown of asphalt binder and cooling of the mix are the primary concerns during transportation and placement. The concerns, performance issues and steps taken to avoid them are summarized in Table 79.

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Table 79: Construction Concerns and Special Considerations for Open-Graded Asphalt Mixes

Concern	Performance issue	Special considerations
Excessive draindown of asphalt binder	Results in fat spots in the finished surface which may appear during rolling or within a few weeks of paving	Proper selection of mixing and compacting temperatures, the use of modified binders (PBA-6), and use of fibers; mixture deposited in windrows from "belly-dump" trucks and transferred to the paver via a pick-up machine is the most common method of delivering Class F mix. End dump trucks have been successfully used to deliver Class F mix, but experience has been that the use of end dump trucks depositing directly into the paver is most likely to result in fat spots on the finished surface. To date, a material transfer vehicle (MTV) has not been used on a Class F mix project.
Cooling of mix	Results in "chunks" which, if not removed or broken up prior to depositing into the paver, can result in tears in the mat, differential compaction, and poor ride.	The "chunks" can be minimized by increasing the temperature, tarping loads, and using trucks with insulated beds to deliver the mixture; the recommended mixing temperature for the Class F mix is based on an asphalt viscosity of 700 to 900 cst. For these projects, the maximum mix temperature was 129°C (265°F) at the plant. Minimum allowable temperature during laydown was 96°C (205°F). This is comparable to 163°C (325°F) and 116°C (240°F) for dense-graded mixes. The lower temperatures for the Class F mix help promote thick film coatings during hauling and laydown. Use of PBA-6 binders modified with polymers and/or ground tire rubber have also minimized the draindown. Class F mixes are evaluated in the field. Experienced personnel will adjust asphalt content and mixing temperature during production and laydown to optimize film coatings for durability and temperature for workability needed to achieve a smooth ride.
Compaction	---	A minimum relative density is not specified. The specifications requires a minimum of four coverages with a minimum 7 Mg (8 ton) gross static steel-wheeled roller prior to mix cooling below 80°C. Additional passes may be necessary to eliminate roller marks. Vibratory compaction is not allowed to avoid fracture of aggregate.

Moore et al mentions some special considerations for unusual constructions considerations, as summarized in Table 80.

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Table 80: Special Considerations

Special Case	Concern	Recommendations
Long Hauls	Excessive draindown of the asphalt binder, cooling of the mix, or both.	Jobs far from the asphalt concrete plant are not recommended for Class F mix; although success has been obtained with one-way haul distances of up to 112 km (70 mi), the current policy is to stay below 56 km (35 mi). Weather conditions and asphalt grade may also influence the recommended haul distance.
Inlays	Adequate drainage; mixes must be allowed to drain	Drainage is accomplished by daylighting the mix on the shoulder through a series of outlet trenches. If adequate drainage outlets are provided, Class F mixes may be used for inlays. Although ODOT has used this option on occasion, it is not recommended for standard practice.
Hand work	The Class F mix, because of its coarse texture, is difficult to rake and is not easily placed where an abundance of hand work is necessary.	Usually not specified for tapers, road approaches, or in city streets where there are inlets and manholes to work around.
Night paving	Projects in high traffic areas require night paving. In these cases, the mix must be heated to higher temperatures to get the necessary temperature for compaction; high temperature can cause more draindown of asphalt binder	When F-mix has been placed at night, fibers have been used to minimize draindown related to the higher temperatures.

Moore et al mention that in early work, quality control was ensured by controlling aggregate gradation, asphalt content, moisture and compaction and also by enforcing pay adjustment factors that are used for dense-graded mixes. A study started in 1995 has, however, shown that some of these variables are more important than the others and there are some considerations that must be made for the unique properties of open-graded asphalt mixtures. The primary observations are that fat spots and rutting are caused by excess asphalt and/or excess fines, the important factors that need to be controlled are

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aggregate gradation, asphalt binder content and mix moisture, and that relatively more weightage should be given to asphalt binder content and percent passing the 0.075 mm sieve for pay adjustments.

1.33.5 Maintenance Practices

Moore et al has indicated several maintenance related issues unique to open-graded asphalt mixes. These issues and recommended maintenance practices are summarized in Table 81. They mention that a 1997 survey of ODOT maintenance practices indicates that blade patching with dense-graded hot mix was the most widely used technique, and the most successful with a mean success rating of 8.0 out of a possible 10.0. Mill and inlay and screed patch with dense-graded hot mix were also widely used and reasonably successful. Only three respondents had milled and inlayed with Class F mix and their experiences varied widely with a minimum success rating of 3 and maximum success rating of 10. The authors note that Class F mix (open graded asphalt mix) is not readily available to maintenance personnel in small quantities and that traditional maintenance techniques have used dense graded mixes.

Table 81: Maintenance Issues and Recommendations

Maintenance Work	Issues	Recommendations
Snow plow damage/ type of plow used	In areas where steel plow blades (without the rubber cover) are used, damage to the Class F mix is greater because the plow blade can break and/or remove the large surface rock. This damage then results in greater future raveling, due to the loss of surface integrity.	Fog Seals are applied periodically to minimize raveling; Chip/Sand Seals are normally used to cover gouges caused by snow plows. The gouges do not generally cause structural damage but do create aesthetic problems.
De-icing	Class F mix requires more de-icing chemicals because of the higher amount of voids in the mix; the chemicals enter the pavement structure and, thus, higher concentrations are needed to keep the surface from icing.	--------
Patching	The issue of patching includes both blade/screed and inlay patching. Although successful inlay patches have been made using F-mix, the type of asphalt mix the maintenance forces have available for use is typically a dense-graded commercial mix. Inlay patching with dense- graded hot mix may block the drainage path in the Class F mix causing problems such as black ice and patch deterioration (due to water infiltration). The greatest concern regarding blade patching is getting the 15 mm – 40 mm thick patch to adhere to the existing Class F mix pavement as well as they traditionally do for dense-graded mixes. Maintenance crews report that these thin patches on the Class F mix last about half as long as patches on dense-graded mixes. Even if F-mix were readily available, blade patches with this mix are not feasible because of the inability to “feather” the edge.	Patching is normally used to repair isolated distress areas; generally a Class C mix is used. As long as the patched areas are small, the benefits of the open-graded mixes are not lost.

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1.33.6 Rehabilitation Practices

Moore et al indicate two types of rehabilitation practices for open-graded asphalt mixes: 1) overlays and 2) mill and fill. They mention that overlays are done with either open-graded or dense-graded mixes, and that mill and fill operation consists of milling off the open-graded mix and replacing with another open-graded mix. They mention that adequate care should be taken to insure the new material placed as part of the mill and fill operation is able to drain completely. The authors mention that there are several considerations that need to be made for rehabilitation of pavements with open-graded asphalt mixes, and that the appropriate method of rehabilitation of such pavements is being researched. They indicate that a small number of such projects have been rehabilitated or have been marked for rehabilitation and that important considerations include inlay repairs prior to overlaying, changing the wearing surface mix type, and drainage issues with a middle layer of open-graded asphalt. Moore et al mention that the European experience indicates a preference for mill and inlay with recycling and that this approach eliminates the challenges associated with overlays and porous pavements.

1.33.7 Performance

Moore et al provide an evaluation of performance of different open-graded asphalt mixes and some comparison of their performance with that of dense-graded mixes (such as classes B and C). These evaluations and comparisons, which have been made with respect to three principal benefits of open-graded asphalt mixes – enhancement of friction, reduction of splash and spray and reduction of noise, are summarized in Table 82.

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Table 82: Evaluation of Performance of Open-Graded Asphalt Mixes

Property	Tests, Results and Observation	Inferences and Recommendations
Friction	Friction was measured at a traveling speed of 64 km (40 mph) and expressed as a friction number (FN) (AASHTO T-242). The friction numbers for the pavement management sections used to model the performance were analyzed using standard statistics. Speed gradients were determined for several projects. The speed gradients were determined as the slope of the FN versus speed curve from 64 km (40 mph) to 88 km (55 mph). The tests were performed in a conventional manner on dry pavements and then repeated on the same sections during heavy rainfall	The resulting average friction numbers of 50.5 for the Class F mix and 53.7 for the Class B mix are very close. The data suggest that under dry conditions, both mix types provide acceptable frictional properties; Speed gradients indicate that the Class F mix had a slightly improved speed gradient in dry conditions and a much improved gradient during rainy conditions when free water was present on the pavement.
Splash and Spray	---------	This feature was obvious when driving on the selected projects during wet weather. Although there was no objective test data available related to splash and spray, the observed amount of water splash and spray for the Class F mix projects was much less than for the Class B/C mix projects. Oregon has received numerous comments from motorists noting improved visibility when traveling on Class F mix pavements during rainy weather. For Oregon's unique climatic condition, with nearly 6 months of rainy weather, this advantage may greatly reduce the number of vehicle accidents.
Noise Characteristics	A field investigation was conducted to determine the noise level on both mixes. Two types of noise measurements were taken. The first was roadside noise and the second was interior vehicle noise. In order to remove any geometric variables, test sites were chosen where fairly new pavement types existed, and overlays of Class F mix were planned in the near future. Tests were then performed before and after overlay at identical locations. For all sites in this study, the noise measurements were taken 50 feet from the centerline of the closest directional travel lanes.	The results indicated porous pavements reduced noise in the higher frequency zones. This conclusion was supported mostly from the roadside measurements, and not from those taken in the interior of the vehicle. A possible explanation for this is that the higher frequencies are dampened by the vehicle shell. As high frequency noises have a shorter wavelength, they would be more apt to be reflected off the vehicle's thin shell, and would hide some of the data and make Class F mix pavements appear a little noisier inside than outside.

1.33.8 Structural Design

Moore et al indicate that the 1993 AASHTO Guide for Design of Pavement Structures and other deflection based procedures are used for structural design of open-graded asphalt mixes in Oregon.

Moore et al mention that a minimum thickness of 50 mm has been specified for class F mix (increased from 37.5 mm in the past, to reduce laydown and compaction problems). A maximum thickness of 100 mm (in two 50 mm lifts) has been used. Regarding extensively cracked existing pavement, Moore et al mention that class F mix is allowed

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only if surface repairs are conducted prior to the placement of class F mix. They indicate that surface repairs can include localized inlays, travel lane inlays, a dense HMAC lift below the Class F mix, or other forms of reflective crack control. Ruts of moderate to high severity should be leveled prior to the Class F mix overlay to prevent water from sitting in the deeper sections of Class F mix.

1.33.9 Limitations
Moore et al mention three conditions under which open-graded mix is not recommended for use:
1) Low volume roads with ADT of less than 1,000. They mention that for this level of traffic the technical benefits are not as noticeable due to the low volume of traffic and lack of heavy loads. Moore et al mention that in eastern Oregon, emulsified asphalt concrete (EAC) is used extensively for the low volume highways because of lower cost of EAC and that maintenance patching will be the primary means of sustaining these low volume highways over a longer period of time. Maintenance has reported that patching over Class F mix is difficult (problems with getting the patch to "stick"). The patches also tend to dam the drainage path and reduce splash and spray benefits.
2) Curbed areas or areas requiring handwork. Class F mix is not recommended for use in areas with curbs or where a significant amount of handwork or feathering is required. The mix's aggregate size and aggregate gradation makes handwork difficult around utility appurtenances and at driveways. Also, curbs block the drainage of the Class F mix.
3) Heavily plowed areas where steel plow blades are used. As a result of snowplow damage Class F mix is no longer recommended in areas where plowing is frequent. The snowplows can cause raveling and gouging resulting in a higher rate of surface deterioration. The determination of frequency of plowing is on an individual project basis, but generally involves elevation, any existing plow damage, and existing chain-up areas or snow zones.

1.34 Abe, T. and Y. Kishi. "Development of Low-Noise Pavement Function Recovery Machine." Proceedings of the Ninth International Conference on Asphalt Pavements. Copenhagen, Denmark. August 2002.

1.34.1 General
This paper presented the results of a research study designed to develop a method and associated equipment for returning the functionality (in terms of permeability) of permeable friction courses. The authors describe laboratory and field studies to evaluate methods of cleaning the void structure of permeable friction courses. Comparisons were made between water jets and the use of water jets along with cavitation. Cavitation occurs when a high speed water jet is injected into static water resulting in development of cavitation bubbles. When collapsing, the cavitation bubbles create a high pressure that dislodges debris.

1.34.2 Benefits of Porous Asphalt Mixtures
Though not specifically studied, the authors indicate that permeable friction courses are low-noise pavements.

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1.34.3 Materials and Design
No specifics on materials and mix design were given.

1.34.4 Construction Practices
No specifics on construction practices were given.

1.34.5 Maintenance Practices
Research documented within this paper was conducted to evaluate two methods of de-clogging (removing debris) permeable friction courses. The two methods included a water jet and a water ejection system creating cavitation in water held within the pavement structure. The initial research was conducted in the laboratory to compare the ability of the two methods to create impulsive pressure. The first experiment entailed ejecting water by both methods onto a pressure sensitive film to compare the methods. Results of this testing indicated that the water jet accompanied by the cavitation bubbles had a significantly higher impulsive pressure. The area affected by the water jet accompanied by the cavitation bubbles was much larger than the water jet alone. [The areas were not quantified; however, pictures comparing the two indicated more than 20 times more area affected by the water jet accompanied by the cavitation bubbles.] The second laboratory evaluation entailed a plaster stripping test. Within this test, a 15 mm steel bar was coated with plaster. Both methods of water ejection were then held 30 mm from the steel bar. Results of the testing showed that the water jet in air stripped the plaster from the steel bar a distance of 20 mm. Twenty-six percent of the circumference of the bar was also stripped. For the water jet in water (cavitation), the distance of the steel bar stripped of plaster was 38 mm and 78 percent of the circumference of the bar was stripped. [Pictures of the bar illustrating the effects of both water ejection methods showed a marked difference in the amount of plaster stripped.] The authors stated that the results of these laboratory tests showed that the water jets within water creating cavitation was potentially a better method of cleaning porous pavements; therefore, additional testing was conducted in the field using this method.

Based upon the results of the laboratory testing, the authors developed a small "function-recovery" test for porous pavements. The device allowed the authors to clean the porous pavement using both methods described above. A porous pavement was artificially clogged using the following method:
1. Mix fine sand, silt and plaster in a dry condition at a weight ratio of 35:35:10.
2. The material was spread onto the porous pavement and leveled.
3. Vibration was applied to cause the materials to settle within the pavement void structure.
4. More material was added and leveled using a rake.
5. Steps 2 and 3 were repeated.
6. Water was sprayed onto the pavement and the material allowed to cure.

In order to compare the two methods in the field, a falling head field permeability test was used on the clogged pavement and again after cleaning. Table 83 presents data when comparing the two methods on the artificially clogged permeable friction course. Results indicate that both methods removed the clogging materials from the pavement structure;

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however, on average, the water jet with cavitation method did improve draining characteristics more.

Table 83: Results of Permeability Testing after Functional Recovery

Type	No. Data Points	Avg. Time before cleaning	Avg. Time after cleaning
Water Jet	6	Impermeable	5.14 (seconds/400 ml)
Water Jet w/ Cavitation	6	Impermeable	4.65 (seconds/400 ml)

Following the laboratory and field experiments, the authors concluded that the best function recovery method was utilizing the water jet and cavitation method. This concept was then used to design a truck-mounted Function-Recovery machine. The new machine is truck-mounted (Figure 14) and is comprised of a specially designed high-pressure water ejection system with a high vacuum suction system (Figure 15).

After developing the new equipment, the authors conducted some field trials to evaluate the effectiveness of the new equipment to remove clogging materials within permeable friction courses. This was again accomplished using the falling head field permeability test. Results are illustrated within Table 84.

Figure 14: Truck-Mounted Function-Recovery Machine

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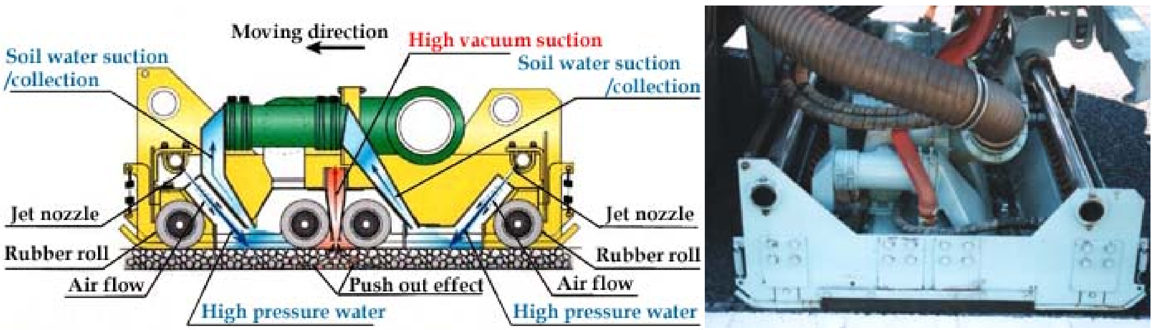

Figure 15: High-Pressure Ejection and Vacuum Systems

See rehabilitation practices below. This paper discusses a water jet cavitation process for rehabilitation of porous pavements. Nothing was discussed about when to use. Measurements were made comparing water jets in air and in water (cavitation) to determine which the better method was.

Rehabilitation Practices

Table 84: Results of Permeability Testing after Use of Truck-Mounted Function Recovery Machine

		Seconds/400 ml								
Measuring Point		1	2	3	4	5	6	7	8	Avg
Sh.	Before cleaning	Imp	Imp	400	125	243	Imp	Imp	133	---
	After cleaning	45	144	37	84	110	149	121	88	97
BWP	Before cleaning	8	8	7	14	10	7	14	14	10
	After cleaning	8	8	6	9	8	5	10	9	8
OWP	Before cleaning	38	Imp	72	2	32	65	73	178	175
	After cleaning	34	24	26	20	16	21	20	30	24

Sh. – Road Shoulder
BWP – Between wheel paths
OWP – outside wheel paths
Imp – 400 ml of water did not flow into pavement during test.

The authors concluded that the new water jet with cavitation equipment is an effective method of general maintenance for permeable friction courses. The self-contained, truck-mounted equipment was successful at improving the drainage capacity of permeable friction courses.

1.34.6 Rehabilitation Practices
Not addressed

1.34.7 Performance

As stated previously, a falling head permeability test was used as an indicator of performance. The test method and equipment is outlined within the Pavement Test Method Manual (1998) of the Japan Road Association. The device is somewhat similar to field permeability devices developed within the U.S. at the National Center for Asphalt

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Technology, University of Arkansas, and Worcester Polytechnic Institute. The device is simply sealed to the pavement surface and water is introduced into a vertical standpipe. A cock-valve is located at the bottom of the device that when opened, water flows from the standpipe into the pavement. The test is run until 400 ml of water flows into the pavement.

1.34.8 Structural Design
Not addressed

1.34.9 Limitations
Not addressed

1.35 Bendtsen, H., C. B. Nielsen, J.Raaberg, and R.A. Macdonald. "Clogging of Porous Bituminous Surfacings – an Investigation in Copenhagen." *Danish Road Institute Report 120*. Road Directorate, Danish Road Institute. Denmark. June 2002.

1.35.1 General
This paper outlines the research of porous pavement clogging and noise reduction undertaken by the Danish government in 1998. The use of porous pavement in Denmark had been halted in the early 1970's because of unsuccessful trials. In the early 1990's two trials using porous asphalt as a surface layer led to its resurgence.

The driving force behind the 1998 research project was the implementation of a new plan called Transport 2005. In this plan the Danish government set goals for reducing traffic noise to 65 dB by the year 2010.

The research conducted by Bendtsen et al consisted of the construction of four porous asphalt test sections. Section 1 was comprised of a 45 mm thick base layer (16 mm maximum aggregate size) with a 25 mm thick top layer (8 mm maximum aggregate size). Section 2 was comprised of a 35 mm thick base layer (16 mm maximum aggregate size) with a 20 mm thick top layer (5 mm maximum aggregate size). Section 3 was comprised of a 65 mm thick base layer (22 mm maximum aggregate size) with a 25 mm thick top layer (5 mm maximum aggregate size). Section 4 was a 30 mm single layer control section of dense mix (8 mm maximum chipping size). All mixes were constructed with a SBS (Styrene-Butadiene-Styrene) modified asphalt.

The research presented in this paper outlines the experimental approach taken by Bendtsen at al. Four test sites were constructed to conduct permeability testing using the Becker's Tube method, noise measurements following ISO 11819-1 (1997) applying the Statistical Pass-By Method, and forensic analysis tests (Plane and Thin Section tests) on cores.

More specifically permeability tests using Becker's tube were conducted at the opening of the road and before/after each semi-annual flushing operation. Bendtsen et al

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proposed classification for the degree of clogging as related to permeability testing as shown in Table 85.

Table 85: Classification of Degree of Clogging

Degree of Clogging of Porous Asphalt	Outflow Time (s)	Permeability Class
New porous asphalt	30	High
Partly clogged porous asphalt (can be flushed clean)	50	Medium
Clogged porous asphalt (cannot be flushed clean)	75	Low

Bendtsen et al further state, "With regards to noise reduction, permeability is also an important property since the pumping of air from vehicle wheels is the cause of some of the noise emanating from moving vehicles. If the voids in the surfacing are clogged, air from tyre-pumping cannot be absorbed and dissipated by the surfacing." Thus noise measurements were also taken in this research. Test section 1 showed no decrease in noise, although that section did have the coarsest aggregate. Section 2 showed a 1.4 dB reduction in noise and Section 3 showed a 2.2 dB decrease in noise between 1999 and 2000.

The forensic analysis tests, Plane and Thin Section tests, were conducted on cores taken immediately after construction in 1999. These cores showed no signs of stripping and all aggregates appeared well coated. Some sections did show the presence of dirt in some of the voids. After the 2000 permeability tests Bendtsen et al decided to take cores from the low permeability areas and. Cores were also taken from areas with good permeability as a check. From the cores with low permeability, it was observed that clogging occurred in the top 20 to 25 mm of the layer. From the cores with high permeability, clogging was still observed in the top 10 to 15 mm.

Results of the research undertaken by Bendtsen et al yielded evidence that the porous pavement does reduce noise levels, permeability in the upper few centimeters of the porous pavement will be reduced over time because of the clogging of voids of asphalt, and the loss of permeability in the porous asphalt can lead to a lessened noise reduction effect as compared with new surfacing.

1.35.2 Benefits of Permeable Asphalt Mixtures

Bendtsen et al mentioned that the main reason for using porous asphalt is its ability to drain water from the surface. This ability reduces aquaplaning (hydroplaning) and reduces splash and spray.

1.35.3 Materials and Design

Bendtsen et al did not discuss materials and design.

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1.35.4 Construction Practices
Bendtsen et al did not discuss construction practices.

1.35.5 Maintenance Practices
Bendtsen et al did not discuss maintenance practices.

1.35.6 Rehabilitation Practices
Bendtsen et al did not discuss rehabilitation practices.

1.35.7 Performance
Bendtsen et al did not discuss performance of permeable asphalt mixes.

1.35.8 Structural Design
Bendtsen et al did not discuss structural design.

1.35.9 Limitations
Bendtsen et al did not discuss limitations.

1.36 Faghri, M. and M. H. Sadd. "Performance Improvement of Open Graded Asphalt Mixes." Report on URI_TC Project No. 536144. October 2002.

1.36.1 General
The objective of this study was to evaluate the effect of additives, fiber and polymer on strength and permeability properties of open-graded asphalt mixes. Faghri and Sadd designed mixes with the Marshal method – using different gradations, cellulose fiber and SBS polymer. They conducted tests to determine air voids, permeability and indirect tensile strength tests. A subset of samples was tested for permeability and strength at a range of temperatures.

Faghri and Sadd mention that the air voids differ significantly when incorporating the different gradations, and that the addition of fiber resulted in decrease in voids in two of the three mixes. Permeability was also reduced with the addition of fibers. The addition of polymer enhanced permeability as well as strength significantly. Faghri and Sadd indicate that for the one mix tested for permeability and indirect tensile strength at different temperatures, an increase in permeability and a decrease in strength were noted with an increase in temperature.

Faghri and Sadd conclude that using a polymer modifier only (as opposed to using fiber or a combination of fiber and polymer) is the best option for optimizing strength and permeability properties.[No durability testing was conducted.]

1.36.2 Benefits of Permeable Asphalt Mixtures
In their introduction Faghri and Sadd indicate that open-graded mixes provide significantly better drainage compared to other mixes, and that the porous structure of the open-graded mixes allow both horizontal (surface) and vertical (through) drainage. They

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mention that such enhanced and rapid removal of water lead to a dry pavement surface during moderate rainstorms and, hence, lead to increased safety of the drivers.

1.37.3 Materials and Design

Faghri and Sadd provide information on different mixes used in this study. In the first phase of this study, to establish base line permeability values, four different gradations were used. However no results were provided for these mixes.

For the mixes in which results were provided, three different gradations (Table 86) were used, referred to as the Arizona, Georgia and SMA mixes. The asphalt contents were determined using the Marshall method. A cellulose fiber and a SBS polymer were used.

Table 86: Study Gradations

Property	Mix		
	Arizona	Georgia	SMA
Gradation Sieve Size (mm)	Percent Passing	Percent Passing	Percent Passing
19	100	100	100
12.5	93	90	93
9.5	85	75	65
4.75	34	24	18
2.36	16	20	8
1.18	7	20	8
0.6	5	14	8
0.3	3	14	8
0.15	2	14	8
0.075	2	9	3
Asphalt Content (PG 64-22)	5.50	5.84	5.80

Note: Fiber – Technocel 1004 fiber; polymer – CITGO SBS polymer; four different mixes were prepared for each gradation – PG 64-22, PG 64-22 with fiber, PG 64-22 with SBS polymer and PG 64-22 with fiber and polymer

Faghri and Sadd provide descriptions of tests conducted on the samples. The information is summarized in Table 87.

Table 87: Tests Conducted on Samples

Test	Method
Bulk specific gravity	Test on paraffin coated specimens, ASTM D 1188-96
Theoretical maximum density	ASTM D2041-95
Voids in total mix	ASTM D3202-94
Permeability	Falling head permeameter
Strength	Indirect tensile strength test, ASTM D4123-82

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In discussing the results of volumetric property tests, Faghri and Sadd indicate that in general the air voids were the lowest in the SMA mixes and the highest in the Georgia mixes. The addition of fiber (alone or with polymer) resulted in lowering of voids in Arizona and SMA mixes, but not in Georgia mixes.
The authors then present the results of permeability tests conducted at different temperatures. They indicate that the SMA mix had very low permeability, while the Georgia mix had the highest permeability. The permeabilities for both Arizona and Georgia mixes were the highest for the polymer only mixes, and lowest for those containing fiber only. Tests were run at 25^{o}C, 35^{o}C, 45^{o}C and 55^{o}C with samples from the Georgia mix. The author point out that the permeability increased with an increase in temperature.

In discussion of results obtained from indirect tensile strength tests, Faghri and Sadd indicate that the strengths for all three mixes increased significantly with the addition of polymer (compared to the base mix containing neither fiber nor polymer). The addition of fiber increased the strength by only a small amount. Samples of the Georgia mix were tested for indirect tensile strength test at 25^{o}C, 35^{o}C, 45^{o}C and 55^{o}C. The authors indicate that the strength decreased significantly with an increase in temperature.

Faghri and Sadd then provide discussion on correlations between permeability and air voids and permeability and strength of the mixes. One correlation between air voids and permeability, they compare their results with those obtained from a study reported by the National Center for Asphalt Technology (NCAT) in 2000. They point out that the trend line obtained from this study is significantly different from the one obtained from the NCAT study and that the range of air voids considered in this study is greater than that considered in the NCAT study. However, in the common range, they point out that the curves are reasonably close to each other.

In discussing the plots of permeability versus strength, the authors contend that there is a significant enhancement in strength and permeability with the addition of polymer, whereas relatively low enhancement in strength and a significant decrease in permeability are achieved with the addition of fiber only.

1.36.4 Construction Practices
No information has been provided on construction practices.

1.36.5 Maintenance Practices
No information has been provided on maintenance practices.

1.36.6 Rehabilitation Practices
No information has been provided on rehabilitation practices

1.36.7 Performance
No information has been provided on performance.

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1.36.8 Structural Design
No information has been provided on structural design.

1.36.9 Limitations
No information has been provided on limitations.

1.37 Giuliani, F. "Winter Maintenance of Porous Asphalt Pavements." XIth International Winter Road Conference. World Road Association (PIARC). Sapporo, Japan. 2002.

1.37.1 General
This paper deals with winter maintenance methods for porous asphalt. Specifically, this paper discusses an anti-icing technique other than the use of salt solutions. Giuliani defines anti-icing techniques as all operations necessary to avoid the formation and development of snow or ice on the pavement surface.

1.37.2 Materials and Mix Design
No specifics on materials and mix design were given.

1.37.3 Benefits
No specific benefits were given.

1.37.4 Construction Practices
No specifics on construction practices were given.

1.37.5 Maintenance Practices
Giuliani states that porous asphalt makes anti-icing operations particularly delicate and expensive. The porous structure, which helps drain water, allows the saline substances generally used in anti-icing operations to drain from the surface. Approximately 30 percent more salt is needed for porous asphalt compared to dense-graded pavements. With respect to traffic safety, late application of anti-icing techniques can lead to safety issues. When winter precipitation occurs without anti-icing operations, tires cannot achieve sufficient traction because the pores of the porous asphalt are clogged with fresh snow which is compacted by passing vehicles forming an ice layer inside the pavement. When this occurs, it can be very difficult to remove this layer of ice.

Giuliani provides an alternative anti-icing technique that consists of installation of a heating system within the pavement during construction. The heating system would use resistive filaments and electricity. This system, based on the Joule effect, would help maintain the porous asphalt surface at a temperature above freezing.

The paper describes a laboratory study conducted to identify the best type of resistive material, the optimal positioning of the filament and the best method of feeding the filaments with electricity. The optimal filament material should result in good heating at very low currents, should have a long service life and have a relatively low cost.

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In order to evaluate the concept, 500 mm square specimens of porous asphalt were fabricated in the laboratory. The lower layer of the specimen was comprised of 120 mm of dense-graded HMA. On top of this lower layer, the resistive filaments were placed. A polymer modified emulsion was then placed on top of the filaments. Finally, 50 mm of a porous asphalt was placed as the upper layer. The resistive filaments were shaped in a sinusoidal form of 24 mm width and 8 mm wavelength. As an initial attempt at the distance between the filaments, a distance of 150 mm was utilized. Thermal images were taken over time to evaluate the ability of the filaments to increase the temperature of the porous asphalt.

Giuliani concluded that the heating elements would increase the road surface temperature above the external ambient temperature. The sinusoidal shaped filaments created a bigger heating surface inside the pavement and could withstand construction better. Placing a system of resistive elements within a pavement is not complicated.

1.37.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.37.7 Performance
No specific performance measures were given.

1.37.8 Structural Design
No specifics on inclusion within structural design were given.

1.37.9 Limitations
No specific limitations were given.

1.38 Greibe, A. P. "Porous Asphalt and Safety." Proceedings of the Ninth International Conference on Asphalt Pavements. Copenhagen, Denmark. August 2002.

1.38.1 General
This paper provides an overview of a research study being conducted in Denmark. The objective of the project was to evaluate the noise reducing effects, durability, performance and safety of porous asphalt. For the study, a section of two-layer porous asphalt was constructed 300m in length and compared to a 100m section of dense-graded HMA.

The majority of the paper provides the results of a literature review that covers a wide range of topics. The first topic is traffic accidents. Greibe cites work in Holland that suggests that there are no differences in safety, whether wet or dry pavements, between porous asphalt and dense-graded HMA. Work in France showed that placement of porous asphalt resulted in 17 percent more accidents but a 25 to 30 percent decrease in wet weather accidents. Greibe noted, however, that this French study omitted pavements where no accidents occurred which may have skewed the results. A similar study in Austria showed a decrease in wet weather accidents with little change on dry pavements.

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Greibe indicated that some of these studies suggested that the benefits of porous asphalt may be somewhat offset by the higher speeds and shorter distance between vehicles resulting from the increased confidence exhibited by drivers on porous asphalt.

The next topic discussed by Greibe was splash and spray. Studies in Europe have indicated that splash and spray may be reduced by as much as 95 percent for porous pavements when compared to dense-graded layers.

Greibe states that new porous asphalt layers have relatively low frictional resistance. This is primarily due to the asphalt binder film coating the aggregates. When a vehicle brakes hard, the friction developed between the tire and the thin film of asphalt binder causes heat which melts the asphalt binder film. This can result in a slippery surface and breaking distances that are 20 to 40 percent longer than on dense-graded layers. The thin asphalt binder layer generally takes 3 to 6 months to wear away.

Less light is reflected by wet porous pavements than dense-graded pavements. This is due to the fact that porous asphalt does not allow water to pool on the pavement surface. Since less light from oncoming vehicles is reflected, pavement markings are more visible.

1.38.2 Benefits of Permeable Asphalt Mixtures
As detailed above, a number of benefits were identified which include: reduced splash and spray, less light reflection and improved wet weather friction.

1.38.3 Materials and Mix Design
No specifics on materials and mix design were given.

1.38.4 Construction Practices
No specifics on construction practices were given.

1.38.5 Maintenance Practices
Greibe states that the winter maintenance of porous asphalt is different than for dense-graded layers. On dense-graded pavement layers, salts mix with moisture on the surface. Porous pavements contain more interconnected voids and the salt disappears in the void structure so more salt is required. An increase of 25 to 100 percent should be expected in salt consumption. The pumping action caused by traffic passing over a porous asphalt layer will continually circulate the salt solution within the void structure of the layer. Therefore, as long as the traffic volume remains high, the driver should not notice any differences between porous asphalt and dense-graded HMA surfaces. Greibe states that since the behavior of salt on porous asphalt surfaces is so different, special locally adjusted salt spreading strategies are required.

Due to the difference in thermal conductivity of porous asphalt compared to dense-graded HMA, the temperature of porous asphalt will fall below freezing before dense-graded layers. Porous asphalt will also stay below freezing longer than dense-grade layers.

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1.38.6 Rehabilitation Practices
No specifics on rehabilitation practices were given.

1.38.7 Performance
No specifics on performance measures were given.
1.38.8 Structural Design
No specifics on inclusion within structural design were given.

1.38.9 Limitations
No. specific limitations were given.

1.39 Iwata, H., T. Watanabe, and T. Saito. "Study on the Performance of Porous Asphalt Pavement on Winter Road Surface Conditions." XIth International Winter Road Conference. World Road Association (PIARC). Sapporo, Japan. 2002.

1.39.1 General
This paper presents results of research conducted by the Japan Highway Public Corporation on porous asphalt. Japan has plans to construct 11,520 km (7,200 miles) of expressway, approximately 6,850 km (4,300 miles) of which have already been constructed. Half of these expressways will pass through snowy or cold regions with a 10 year average maximum snow fall of 30 cm or more.

In Japan, snow and ice control generally takes the form of a special water mixed solution for removal of snow. This poses a potential problem when porous asphalt pavements are used because the snow and ice control solution can drain into the porous asphalt layer.

The authors list the current status on the use of porous asphalt in Japan as follows:
1. Approximately 33 percent of the in-service expressways currently include porous asphalt as the wearing layer.
2. Air void contents within porous asphalt layers are approximately 20 percent.
3. There is approximately an 80 percent reduction in wet weather accidents when comparing porous asphalt to dense-graded layers.
4. Japan uses a permeability requirement for porous asphalt layers. The test criterion is that 400 ml of water must penetrate into the porous layer within 10 seconds.
5. Japan has experienced an average reduction of 3 dB when using porous asphalt as compared to dense-graded HMA layers.

The authors also provided the following technical problems associated with porous asphalt:
1. Methods for snow and ice control need to be established.
2. Methods for restoring permeability need to be established.
3. The durability of porous asphalt needs to be improved in regions where tire chains are used.
4. Methods for rehabilitation of porous asphalt need to be established.

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5. Methods for ensuring an impermeable layer below the porous asphalt layer are needed.
6. Ways to reduce costs for porous asphalt are needed.

The authors conducted a number of experiments to evaluate the performance of porous asphalt in winter conditions. The first experiment entailed comparing the temperature in porous asphalt and dense-graded HMA layers during cold weather. During the daytime, the road surface temperature of dense-grade layers was higher than nearby porous asphalt layers by about 2ºC at the largest difference. At nighttime, the road surface temperature was higher on porous asphalt layers by about 0.5ºC at the largest difference. During snowfalls, the authors indicate that the temperature of porous asphalt was lower than that of dense-graded layers by an average of 0.2ºC.

Another experiment conducted by the authors entailed many visual examinations of pavement surfaces during dry, wet, slush, snow, and ice road conditions. Tables 88 and 89 present the results of the author's examinations of pavement sections listed as either rutted or non-rutted. The upper right part of each of these tables represents the number of cases where the visual surface condition of porous asphalt was considered to be worse than nearby dense-graded surface layers. Alternatively, the lower left portion of each table signifies where the porous asphalt layers were considered in better condition than nearby dense-graded layers. The highlighted cells going diagonally across the tables are instances where the pavement surfaces of porous asphalt and dense-graded layers were considered similar.

Table 88: Road Surface Conditions during a Snowfall (Rutted Sections)

Rutted Sections		Porous Asphalt Pavement					
		dry	wet	slush	snow	ice	total
Dense-Graded Pavement	dry	33					33
	wet	1	297	23			321
	slush		14	96	2		112
	snow		10	2	88		100
	ice		10	3	3	23	39
	total	34	331	124	93	23	605

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Table 89: Road Surface Conditions during a Snowfall (Non-Rutted Sections)

Non-Rutted Sections		Porous Asphalt Pavement					
		dry	wet	slush	snow	ice	total
Dense-Graded Pavement	dry	34					34
	wet	2	247	29	4		282
	slush		13	114	8	3	138
	snow		1	11	109	2	123
	ice		4	5	1	18	28
	total	36	265	159	122	23	605

Based upon Tables 88 and 89, the authors indicated that two cases needed to be highlighted. The first scenario is when the dense-graded HMA surface was wet. In this situation, the snow falling onto the dense-graded surface is being melted while on the porous asphalt it is becoming slush or the snow was collecting on the porous asphalt surface. Therefore, conditions on the porous asphalt layer were worse than the comparing dense-graded surfaces. Based upon the data, these conditions only occurred less than 6 percent of the time during snowfall events.

The second condition noted by the authors was when the dense-graded surfaces were covered with ice and the surface of the porous asphalt layer was either wet, slush or snow. In these instances the freezing of the road surface is difficult to occur on porous asphalt surfaces because any melted snow does not stay on the pavement surface. For the dense-graded surfaces, the melted snow stays on the road surface and can freeze if the temperature and salinity conditions are right.

In another experiment, the authors monitored the salinity concentration on the pavement surface using three different spreading methods: solution, solid and wet salt. Data presented by the authors indicated that the rate of decrease in salinity concentration was less for porous asphalt than for dense-graded HMA. This observation was independent of the manner of spreading the winter maintenance materials. The authors indicate that the small decrease in salinity concentration offered by the porous asphalt allows the amount of anti-icing chemicals for freezing prevention to be reduced. When anti-icing chemical solution is spread, the porous asphalt maintained a higher salinity concentration than the dense-graded layers. When solid or wet salt are used for anti-icing, the salinity concentration of the dense-graded layer stayed higher.

The authors also conducted skid testing using the Japan Highway Public Corporation's locked wheel skid tester at 50 km/hr. Results from this testing indicated that except for when the porous asphalt was covered with compacted snow, the porous asphalt had higher skid resistance coefficients than dense-graded surfaces. When compacted snow was encountered, the friction coefficients were similar for the two surfaces. This indicated that the porous asphalt maintained equal or higher friction coefficients.

The final experiment conducted by the authors was to evaluate wet weather accident data for roadways in which a dense-graded layer was replaced by porous asphalt. In these

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instances, one year worth of accident reports were compared. The data indicated that wet weather accidents decreased 34 percent when dense-graded surfaces were replaced with porous asphalt.

1.39.2 Materials and Mix Design

The only information provided relative to materials and mix design is that the target air voids contents within construction porous asphalt is approximately 20 percent.

1.39.3 Benefits

Two benefits mentioned within the paper were decreased noise levels and reduced wet weather accidents (improved wet weather frictional resistance).

1.39.4 Construction Practices

No specifics construction practices were given.

1.39.5 Maintenance Practices

Research conducted by the authors indicated that there is no significant difference between dense-graded surfaces and porous asphalt surfaces in terms of road surface conditions during a snowfall event. Additionally, there is no significant difference in the salinity concentration between the two surfaces when using anti-icing chemicals.

1.39.6 Rehabilitation Practices

No specific rehabilitation practices were given.

1.39.7 Performance

The authors indicated that porous asphalt surfaces offer higher skid resistance during rain events, even if the surface is slightly frosted.

1.39.8 Structural Design

No specifics on inclusion within structural design were given.

1.39.9 Limitations

No specific limitations were given.

1.40 Kandhal, P.S. "Design, Construction and Maintenance of Open-Graded Asphalt Friction Courses." National Asphalt Pavement Association Information Series 115. May 2002.

1.40.1 General

This document provides an overview for materials selection, mix design, construction, pavement structural design, winter maintenance and rehabilitation for OGFCs. Specific chapters are provided on performance benefits, mix design, mix production and placement, pavement design considerations and maintenance and rehabilitation.

Kandhal begins the document with a brief history of OGFCs. OGFCs evolved through experimentation with plant mix seal coats. During the 1940's, the California Department

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of Highways was experiencing problems with chip seals and seal coats. These problems included bleeding, raveling, loose stone and a relatively short performance life. To combat the problems, the cover aggregate was mixed with a relatively high asphalt binder content in a conventional HMA plant. These plant mix seal coats provided the same frictional benefits as chip seals but were more durable, provided some improvement in ride quality, reduced noise and eliminated loose chips.

During the 1970's, the Federal Highway Administration began a program to improve the overall frictional properties of the roadways within the U.S. The plant mix seal coats were identified as a tool to accomplish this goal. At this time, the term open-graded friction coarse was coined. During this time period, the FHWA published a mix design procedure for OGFC's.

A 1998 survey referenced in the document indicated a number of states had experienced good performance with OGFCs. Of those not experiencing good performance, the primary problems encountered included raveling, delamination and loss of permeability. The survey indicated that the use of polymer-modified binders, relatively high asphalt binder contents (by using fibers) and/or relatively open gradations had alleviated some of these problems.

1.40.2 Benefits of Permeable Asphalt Mixes

Benefits of OGFC mixes were categorized as related to safety or environment. Benefits related to safety include improved wet pavement frictional resistance, less potential for hydroplaning, reduced splash and spray, reduced glare and improved visibility of pavement markings.

Kandhal cited a number of references describing research conducted in the U.S., Canada and Europe that showed the improved wet pavement frictional resistance of OGFC pavements. Much of the research dealt with comparing the speed gradient (or friction gradient) of OGFCs. A frictional speed gradient can be defined as the rate of decrease of a friction number per unit increase in speed. Research by the Bureau of Public Roads during the late 1960's showed that the friction gradient of OGFC pavements was considerably lower than for dense-graded pavements. Table 90 presents data from a Pennsylvania Department of Transportation project that also shows a decreased friction gradient for OGFC layers. Similar work in Oregon and Louisiana also shows decreased friction gradients for OGFC compared to dense-graded layers.

Table 90: Frictional Data (Pennsylvania)

Mix Type	Friction Number		Friction Gradient
	30 mph	40 mph	
OGFC (gravel)	74	73	0.10
OGFC (dolomite)	71	70	0.10
Dense-graded HMA (gravel)	68	60	0.80
Dense-graded HMA (dolomite)	65	57	0.80

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Research in Virginia, France and Canada showed that OGFC layers reduced wet weather crashes. In Virginia, wet weather accidents were reduced from 39 percent of all accidents on State Route 23 to 17 percent of all accidents. On the A7 motorway in France, the number of accidents fell from 52 (1979 to 1985) to none (1985 to 1989) after OGFC was placed on a section of roadway. In Canada, the placement of OGFC on a section of roadway reduced the number of wet weather accidents by 54 percent and the total number of accidents by 20 percent.

During a rain event, water infiltrates into the OGFC layer. Because the water infiltrates into the OGFC layer, a continuous film of water will not be available to cause hydroplaning. Kandhal indicates that hydroplaning also may not occur during prolonged, heavy rainfalls because the pressure developed under a vehicle's tire will dissipate through the porous structure of the OGFC.

Another benefit of the OGFC related to its ability to drain water is the reduction of splash and spray. The use of OGFC almost eliminates splash and spray because water does not pool on the pavement surface. Kandhal references a UK research study that showed a 90 to 95 percent reduction in the amount of splash and spray on OGFC surfaces when compared to dense-graded surfaces.

Due to the lack of water pooling on the pavement surface, drivers do not see the glare caused by the headlights of oncoming vehicles. The reduction in glare contributes to better visibility and helps result in reduced driver fatigue.

The final benefit related to safety mentioned by Kandhal was improved visibility of pavement markings. Kandhal states that pavement markings on OGFC surfaces have high nighttime visibility, especially during wet weather.

The lone benefit mentioned by Kandhal of OGFC surfaces related to the environment is reduction of tire/pavement noise. Kandhal cited numerous research studies that showed OGFC layers reduce tire/pavement noise approximately 3 dB(A) compared to dense-graded HMA. To put a 3 dB(A) reduction in noise into perspective, this reduction can also be achieved by reducing the volume of traffic by half.

1.40.3 Materials and Mix Design

Kandhal presented a mix design system that was based upon research at the National Center for Asphalt Technology. The mix design system includes four primary steps which include: 1) materials selection; 2) selection of moisture susceptibility; 3) selection of optimum asphalt binder content; and 4) evaluation of moisture susceptibility.

Materials needed for selection include coarse and fine aggregates, asphalt binders, and stabilizing additives. Kandhal states that aggregate requirements for OGFC can be similar to those for stone matrix asphalt (SMA). Aggregates should be tough, durable, angular and cubicle. Table 91 summarizes Kandhal's recommendations for coarse aggregate property requirements for OGFC. For fine aggregates, the uncompacted void content (fine aggregate angularity) was recommended with a minimum value being 45

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percent. Asphalt binders should be selected for the project location considering the environment, traffic and expected functional performance for the layer. Kandhal recommends the use of high stiffness binders, generally two grades stiffer than the local climate requires, for hot climates and cold climates having freeze/thaw cycles, medium to high traffic volumes and OGFC mixes with more than 20 percent air voids. The addition of fiber is also desirable to reduce the potential for draindown and to allow relatively high asphalt binder contents.

Table 91: Coarse Aggregate Requirements for OGFC

Property	Recommended Criteria
Los Angeles Abrasion	less than 30 percent
Fractured Faces	90 percent with two or more faces minimum
Flat and Elongated	5:1 – 5 percent maximum
	3:1 – 20 percent maximum

The second step in the mix design system entails developing a design gradation. Similar to SMA, OGFC should have an aggregate skeleton that has stone-on-stone contact; therefore, the voids in coarse aggregate (VCA) of the gradation is determined (Figure 16). A single gradation band was recommended (Table 92). Similar to the Superpave mix design system, Kandhal suggests developing three trial blends using the selected aggregates. For each of the trial blends, the aggregates are mixed with 6 to 6.5 percent asphalt binder using 50 gyrations of a Superpave gyratory compactor. The air voids and VCA of the compacted trial blends are then evaluated and the blend having stone-on-stone contact with high air voids is selected as the design gradation.

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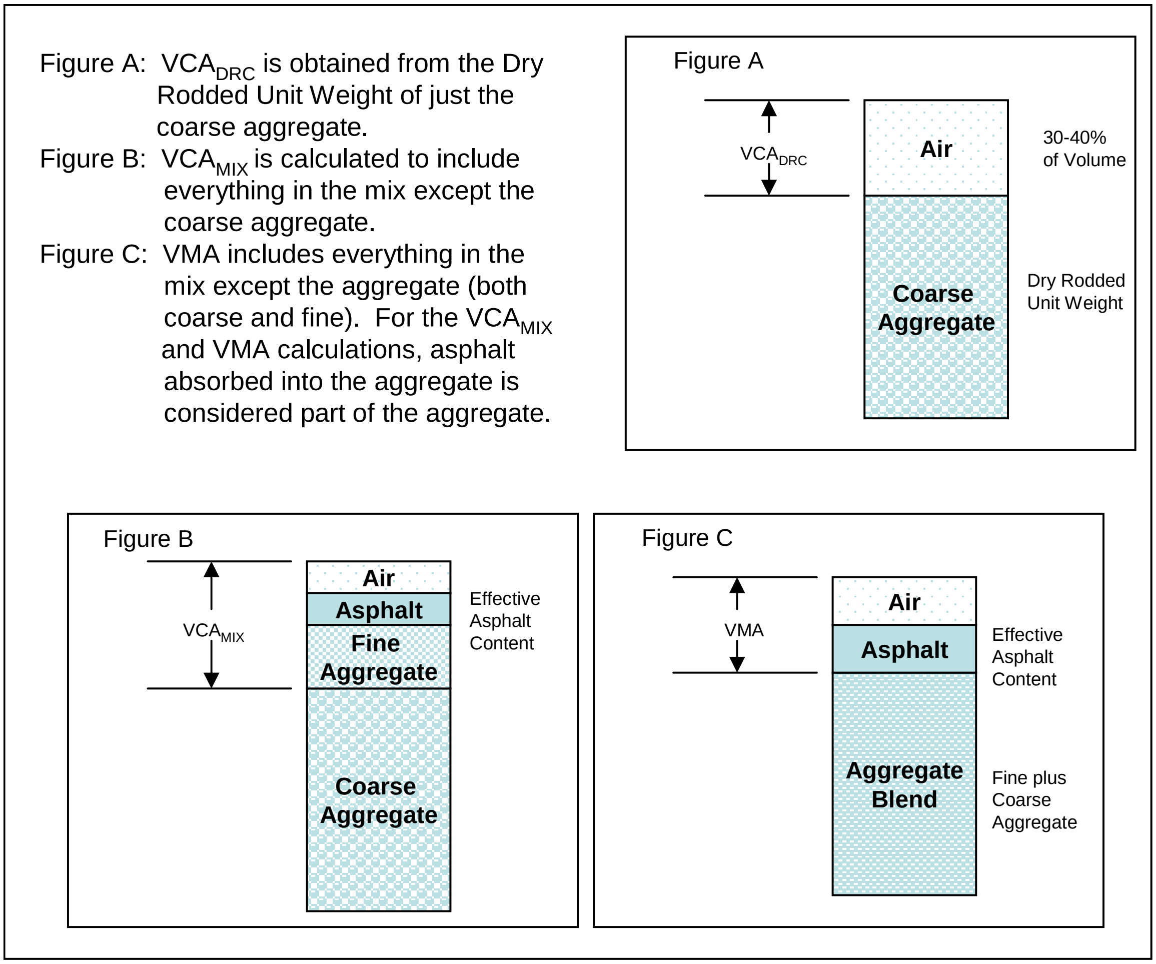

Figure 16: Voids in Coarse Aggregate Concept for Ensuring Stone-On-Stone Contact

Table 92: Recommended Gradation for OGFC

Sieve, mm	Percent Passing
19	100
12.5	85-100
9.5	35-60
4.75	10-25
2.36	5-10
0.075	2-4

Kandhal recommends four properties in the selection of design asphalt binder content: air voids, Cantabro abrasion loss on unaged OGFC samples, Cantabro abrasion loss on aged OGFC samples and draindown. Optimum asphalt binder content should provide at least 18 percent air voids, though Kandhal indicates higher values are desirable.

The Cantabro Abrasion test evaluates the resistance of compacted OGFC specimens to abrasion loss. The test method entails compacting an OGFC specimen to the design compactive effort, allowing the specimen to cool to room temperature, weighing the

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specimen to the nearest 0.1 gram, and then placing the specimen into a Los Angeles abrasion machine without a charge of steel spheres. The Los Angeles abrasion machine is then operated for 300 revolutions at a rate of 30 to 33 rpm at a temperature of 25˚ C. After the 300 revolutions, the specimen is removed and again weighed to the nearest 0.1 gram and the percent mass loss determined. As stated above, there are requirements for both aged and unaged OGFC samples. For the aged condition, samples are placed in a forced draft oven at a temperature of 85˚C for 120 hours.

The final property evaluated during the design of OGFC is draindown potential. For this testing, the draindown basket developed at the National Center for Asphalt Technology is used with a maximum allowable percentage of draindown being 0.3 percent.

Kandhal states that laboratory permeability testing is optional. If utilized, permeability vales greater than 100 m/day are recommended.

Optimum asphalt binder content is selected as a binder content in which all four properties are met. Generally, Cantabro abrasion loss will define a minimum asphalt binder content and draindown/air voids will define a maximum asphalt binder content.

The final step in the mix design system is to evaluate the designed OGFC mix at the selected optimum asphalt binder content for moisture susceptibility. For this, AASHTO T283 is utilized with the following modifications: 1) specimens should be compacted to 50 gyrations with no air void requirement; 2) apply a partial vacuum of 26 inches of Hg for 10 minutes to saturate the specimens; and 3) keep the specimens submerged in water during freeze/thaw cycles. Kandhal recommends 5 freeze/thaw cycles. The retained tensile strength after 5 freeze/thaw cycles should be at least 80 percent.

1.40.4 Construction Practices

Production and placement of OGFC is similar to dense-graded HMA; however, Kandhal provides some differences. The main modification required at the production facility is the addition of a method to incorporate fibers into the production process. Depending upon the type of plant (batch or drum), the addition of fiber can differ. In the case of batch plants, the fiber is added at the pugmill. Bales of loose fibers have been added manually into the pugmill and also the fibers have been blown into the pugmill from a fluffing device. Pelletized fibers can also be placed directly into the pugmill either manually or from a feeder. Within drum plants, fibers (loose or pelletized) are generally blown continuously into the drum. The fiber line is generally placed about 0.3m (1ft) upstream of the asphalt binder line. This is done so that the fibers are captured by the asphalt binder and do not end up in the dust collection system.

During mixing, it is good practice to increase the mixing time to ensure that the fibers are fully dispersed within the mix. When using a batch plant, the dry mixing time should also be increased. Care must also be taken on the screen decks of batch plants. OGFC gradations consist of a large percentage of single size aggregates; therefore, override of the screen decks and hot bins can occur. In order to reduce draindown potential, OGFC should not be stored in surge bins or silos.

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OGFC is transported to the project using normal haul trucks. However, because of the relatively high asphalt binder content and the use of polymer modified binders, several precautions are needed. First, truck beds will need a heavy coat of asphalt release agent to prevent the OGFC from sticking to the truck bed. The heavy coat should not be able to pool in the truck bed; however, pools of release agent will cool the OGFC and cause cold lumps.

In order to maintain mixture heat, trucks should be tarped. Without tarping the truck, there is an increased potential for crusting of the mix during transportation. This potential is further increased during long haul distances.

OGFC should only be placed on an impermeable pavement layer. Kandhal indicates that newly placed dense-graded HMA layers may be permeable to water; therefore, he recommended a uniform tack coat at an adequate application rate to fill and seal the surface. Kandhal references a FHWA recommendation of applying a 50 percent diluted slow-setting emulsion tack coat at a rate of 0.05 to 0.10 gallon per square yard. A slow setting emulsion was suggested because it is more likely to penetrate the surface voids more effectively.

Kandhal states that the use of a material transfer device between the haul truck and paver is optional, but highly recommended. This type of equipment remixes any cold lumps that may result from transportation.

A conventional paver is generally used to place OGFC mixes. A hot screed is important to prevent pulling in the mix.

A conventional steel wheel roller is used to compact OGFC. Pneumatic tire rollers are not recommended because they tend to cause pick up. Vibratory rollers are also not recommended because they can break down aggregate in the compacted mix. Kandhal does state that vibratory rollers may be used at transverse joints.

Because of the open aggregate grading and the use of polymer modified asphalt binders, OGFC mixes are generally harsh; therefore, handwork is often difficult.

1.40.5 Maintenance Practices

General maintenance activities revolve around removing debris from the OGFC layer that builds up over time. Kandhal mentions three methods of cleaning an OGFC pavement; 1) cleaning with a fire hose; 2) cleaning with a high pressure cleaner; 3) cleaning with a specially made cleaning vehicle. Kandhal references a research project in Switzerland that suggested that cleaning with the high pressure cleaner was found most effective of the three methods mentioned above.

Kandhal states that there has been little problem with winter maintenance in Europe. An OGFC layer does have different thermal properties than dense-graded HMA layers as the temperature will tend to be about 2˚C lower. Therefore, frost and ice will accumulate sooner and last longer on an OGFC surface. Because of this, Kandhal states that long

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continuous stretches of OGFC are more desirable than alternate applications of dense-graded HMA and OGFC.

The timing of preventative winter maintenance is very important. Salting is only beneficial on dry pavements when the temperature is less than -10˚C. Referencing work from Austria, Kandhal indicates that a combination of 70 percent dry salt and 30 percent salt water solution applied at a rate of 10 to 20 g/m^2 has been effective for OGFC. The use of brine at the appropriate time has also been found effective in Holland and reduces the amount of dry salt needed to only 15 percent more than that required for dense-graded HMA.

1.40.6 Rehabilitation Practices

Kandhal states that it is generally recommended to mill off the existing OGFC and replacing with a new OGFC or other type HMA.

1.40.7 Performance

No specific performance measures were given.

1.40.8 Structural Design

In the U.S., OGFC layers are generally placed at a thickness of 20mm; however, Oregon has used 50mm for many years. In Europe, thicknesses also range from 20 to 50mm.

Assignment of a structural coefficient for OGFC layers vary throughout the U.S. California assigns a structural coefficient similar to permeable base layers. Oregon assigns the same structural number for both OGFC and dense-graded layers. Some states do not assign a structural number to OGFC.

1.40.9 Limitations

Kandhal provides a number of situations where OGFC should not be used. OGFC should likely not be used on projects that include long haul distances. Long haul distances increase the potential for draindown and/or cooling of the mix. Oregon restricts haul distance for OGFC to 56km (35 miles). OGFC should not be used in inlays. OGFCs should be free draining at the pavement edge; therefore, they should not be used as an inlay. Handwork is difficult with OGFC mixes. Therefore, projects that include a lot of handwork should probably not include OGFC. Kandhal noted that OGFC should not be used in snow zones where extensive snow plowing is required. OGFC may ravel and shove in some critical pavement locations such as intersections, locations with heavy turning movements, ramp terminals, curbed sections and other adverse geometric locations. The final limitation noted by Kandhal has to do with underlying layers. OGFC should not be placed on a permeable layer. Water can infiltrate a permeable underlying layer causing moisture damage.

1.41 Larsen, L.E. and H. Bendtsen. "Noise Reduction with Porous Asphalt – Costs and Perceived Effect." Ninth International Conference on Asphalt Pavements. International Society of Asphalt Pavements. Copenhagen, Denmark. 2002.

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1.41.1 General

This paper presents a cost comparison between various options of noise abatement including noise barriers, building insulation and porous asphalt. The authors state that approximately 20 percent of all homes in Denmark are exposed to noise levels exceeding 55 dB. Six percent are exposed to more than 65 dB. Within Denmark, a noise level of 65 dB requires noise abatement.

Larsen and Bendtsen indicate that noise reducing pavements are the most realistic means for noise abatement. Noise barriers are not feasible along many urban roadways. Insulation of building facades reduces noise within the structure but does nothing for persons outdoors.

For roads with traveling speeds greater than 70 km/hr, a single layer of porous asphalt having an 8 mm maximum aggregate size has been used to reduce noise levels by an average of 3dB compared to typical dense-graded wearing layers. At speeds lower than 70 km/hr, there has been a problem with the void structure of the porous asphalt clogging, whereby, much of the noise reduction is lost.

Another alternative porous asphalt technique for slower speed roadways that was currently being evaluated in Denmark was the placement of two-layer porous asphalt. The two layer porous asphalt system being investigated included a 25 mm top layer comprised of an 8 mm maximum aggregate size porous asphalt overlying 45 mm of 16 mm maximum aggregate size porous asphalt. The authors indicated that the pores within the top layer could be kept open by high pressure cleaning twice a year.

The objective of this study was to compare the cost of two layer porous asphalt to noise barriers and sound insulation as noise abatement techniques. The authors created three scenarios representing city streets, ring roads and freeways.

The unit of measure that was used to define noise levels within the study was the Danish Noise Exposure Factor (NEF). The NEF is an expression for accumulated noise annoyance within housing areas. As NEF is reduced, noise annoyance is also reduced. For each of the three scenarios, the authors estimated costs for reducing the NEF.

Assumptions used for the three scenarios included:

<u>City Streets</u>

- Two lane roadway with an annual daily traffic of 12,000 vehicles including 10 percent trucks.
- Vehicle speed of 50 km/hr
- Closed rows of apartment buildings that are 6 stories tall with shops on the bottom floor. On a 1000 m section of roadway, 655 apartments line the road.
- With no noise abatement, the first floor level of noise would be 68 dB and the noise level on the fifth floor would be 65 dB.

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Ring Road
- Four lane roadway with an annual daily traffic of 30,000 vehicles including 10 percent trucks.
- Three-story apartments on each side of the road. On a 1000 m section of roadway, 399 apartments line the road.
- Vehicle speed of 70 km/hr.
- With no noise abatement, the noise levels would be approximately 73 to 74 dB.

Freeways
- Three lanes in each direction with an annual daily traffic of 60,000 including 10 percent trucks.
- Row houses on each side of the freeway. Half of houses are single story and half are two-story.
- Vehicle speed is 110 km/hr.
- Noise levels are 77 dB on facades of houses within the first row.

Table 93 presents the results of the cost analyses comparing two layer porous asphalt, noise barriers and building insulation.

Table 93: Costs (net present value) and Effect of the Three Noise Abatement Techniques

		City Street	Ring Road	Freeway
Two-Layer Porous Asphalt	30 year cost	Euro 296,000	Euro 360,000	Euro 477,000
	dB reduction	4	5	6
	NEF reduction	85.7	153.2	179.4
	Cost/dB/dwelling	Euro 111	Euro 180	Euro 183
	Cost/NEF	Euro 3,454	Euro 2,350	Euro 2,659
Noise Barrier	30 year cost	---	Euro 1,335,000	Euro 1,590,000
	dB reduction	---	0-12 (avg 3.9)	0-13 (avg 6.2)
	NEF reduction	---	75.5	195.3
	Cost/dB/dwelling	---	Euro 858	Euro 590
	Cost/NEF	---	Euro 17,658	Euro 8,141
Building Insulation	30 year cost	Euro 2,685,000	Euro 1,607,000	Euro 2,890,000
	dB reduction	9	9	9
	NEF reduction	99.0	170.0	123.7
	Cost/dB/dwelling	Euro 449	Euro 448	Euro 738
	Cost/NEF	Euro 27,121	Euro 9,453	Euro 23,363

Based upon the data presented in Table 93, the authors stated that two-layer porous asphalt is an effective means of noise abatement both in terms of reductions in decibels and reductions in NEF.

1.41.2 Benefits of Permeable Asphalt Mixtures
The authors state that the reduction in tire-pavement noise levels is a benefit of using porous asphalt.

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1.41.3 Materials and Mix Design

The only information provided on materials and mix design was that single layer porous asphalt mixes have an 8 mm maximum aggregate size. When two layer porous asphalt layers are used, the top layer has an 8 mm maximum aggregate size and the lower layer of porous asphalt has a 16 mm maximum aggregate size.

1.41.4 Construction Practices

No specifics on construction practices were given.

1.41.5 Maintenance Practices

The authors indicated that two layer porous asphalt as studied in this paper could maintain permeability if cleaned by high pressure water twice a year.

1.41.6 Rehabilitation Practices

No specifics on rehabilitation practices were given.

1.41.7 Performance

No specifics on performance were given except for that the noise reduction capabilities of porous asphalt do diminish over time.

1.41.8 Structural Design

The authors state that two layer porous asphalt pavements include 25 mm of the 8 mm maximum aggregate size porous asphalt and 45 mm of the 16 mm maximum aggregate size porous asphalt. Additionally, the authors state that the total of 70 mm of porous asphalt provides more structural capacity than 30 mm of typical dense-graded HMA.

1.41.9 Limitations

No specific limitations were given.

1.42 Litzka, J. "Austrian Experiences with Winter Maintenance on Porous Asphalt." Proceedings of the Ninth International Conference on Asphalt Pavements. Copenhagen, Denmark. August 2002.

1.42.1 General

This paper describes the evolution of winter maintenance for porous asphalt within Austria. Also included within this paper are the results of a summit, held in Austria during 1999, that describes the typical practices of a number of European countries for the winter maintenance of porous asphalt. Porous asphalt test sections were first placed in Austria during 1984. From 1984 until approximately 1988, porous asphalt was sparingly used within Austria; however, from 1988 until 1991, approximately half of the pavements using porous asphalt were placed. Since 1991, the use of porous asphalt has only been minimal. The paper states that the use of porous asphalt has declined due to two problems: structural life and maintenance. Additionally, the development of dense surface layers that reduce tire-pavement noise, like small nominal maximum aggregate size SMA mixes, has led to the decrease in use of porous asphalt.

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1.42.2 Benefits of Permeable Asphalt Mixtures

The authors did not specifically conduct testing to evaluate the benefits of permeable friction courses; however, benefits of PFCs discussed within the paper include noise reduction and reduced hydroplaning.

1.42.3 Materials and Design

This paper does not specifically deal with the design of friction courses; however, the authors state that only modified binders are used for porous friction courses. Fibers may also be included. Air void contents are specified at greater than or equal to 17 percent.

1.42.4 Construction Practices

This paper does not specifically deal with the construction practices of friction courses; however, the authors state that porous asphalt is placed onto an impervious layer (SAMI). Acceptance testing for porous asphalt pavements include: layer thickness, smoothness, percent compaction, drainage (permeability) and noise emission.

1.42.5 Maintenance Practices

According to the paper, Austria's climate is humid, cool and characterized by cloudy conditions, wind and precipitation throughout the year. In winter (December to February), average temperatures generally range from -0.2˚C to -2.2˚C (32˚F to 28˚F, respectively). The paper also states that there are a large number of freeze-thaw cycles.

Three summaries were provided on the maintenance of porous asphalt: the results of a survey conducted in Austria during 1993/1994, an international exchange of experiences in 1999, and another Austrian survey conducted in 2002.

<u>1993/1994 Austrian Survey</u>

Because of their open structure, porous asphalt surfaces are about 1˚C colder when compared to dense-graded surfaces. Therefore, porous asphalt surfaces remain at a colder temperature longer and reach freezing temperatures earlier than dense-graded surfaces. Because of the extended time for cold temperatures, the consumption of de-icing materials is higher.

During slushy conditions, the performance of porous asphalt surfaces is slightly poorer than dense-graded surfaces. Snowplows tend to push the slushy material into the void structure of the porous pavement. The cold temperatures cause the slushy material to swell such that the slushy materials are again a road hazard. To prevent the slushy material remaining in the void structure of the porous asphalt from swelling, salting of the roadway must be conducted immediately after the snowplows pass. This is in contrast to dense-graded surfaces. The extra salting also leads to increased usage of de-icing materials for porous asphalt surfaces compared to dense-graded surfaces.

The survey states that on dense-graded surfaces, the preventative application of anti-icing materials may delay or even prevent icing. However, on porous asphalt surfaces, immediate and continuing applications of anti-icing materials are required. When road salt is applied to porous asphalt surfaces too late or the anti-icing agents are ineffective,

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the removal of the resulting ice layer is much more difficult on porous asphalt surfaces than dense-graded surfaces.

The authors state that porous asphalt will require 25 to 50 percent more deicing agents than dense-graded surfaces. Winter maintenance crews must be able to respond quickly and flexibly to different weather and road conditions. Weather forecasting systems and electronic road condition surveillance systems are very helpful in this quest.

<u>1999 International Exchange of Experiences</u>
In 1999, an International Exchange of Experiences was hosted in Austria that brought experts from throughout Europe together to discuss issues with porous asphalt. The following presents a summary of the experiences of various countries with respect to the maintenance of porous asphalt.

Germany
Winter maintenance of porous asphalt is generally considered more expensive and slightly more difficult than dense-graded surfaces. The standard quantity of salt applied to porous asphalt surfaces is 10 g/m^2; however, within problem areas the required quantity of salt may reach 40 g/m^2. The availability of weather forecasting systems helps facilitate timely response to winter maintenance activities.

Italy
Dry road salt is generally applied to wet pavements in quantities of 10 to 20 g/m^2 as a preventative maintenance. During snowfalls, the dry road salt is again applied at the same rate. After snowplows have removed the snow from the pavement surface another 10 to 30 g/m^2 of road salt is applied depending upon the road conditions. It was stated that Italy's change from a coarse maximum aggregate particle size porous asphalt (20 mm) to a smaller maximum particle size (16 mm) has led to significant improvement in road conditions during winter months.

Netherlands
Road salt consumption increased by about 25 percent when using porous asphalt. In very severe winter weather, speed limits or road closures have been employed.

Slovenia
It was stated that in Slovenia, salt consumption is up to 100 percent higher for porous asphalt mixtures than for dense-graded surfaces.

Austria
The increase in salt usage for porous asphalt surfaces is about 10 to 15 percent over dense-grades surfaces. Best results occur when using a wet salt application (salt/brine ratio of 2:1).

<u>2002 Survey Within Austria</u>
Statements by the survey respondents indicated that preventative anti-icing measures are difficult and more expensive. Tests with dry road salt did not produce positive results [no

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information or data was provided]. In summary to the 2002 survey, the following statement was made "… even the advocates among the road operators seem inclined to move away from conventional porous asphalt even though its benefits with regard to noise control are generally appreciated."

1.42.6 Rehabilitation Practices
No information is provided on construction practices of friction course.

1.42.7 Performance
No information is provided on the field performance of friction course.

1.42.8 Structural Design
No information is provided on structural design of friction course; however, the paper does state that porous asphalt pavement layers are placed at 40 mm (1½ in).

1.42.9 Limitations
No information is provided on limitations of use.

1.43 Padmos, C. "Over Ten Years Experience with Porous Road Surfaces." Ninth International Conference on Asphalt Pavements. International Society of Asphalt Pavements. Copenhagen, Denmark. 2002.

1.43.1 General
This paper presents a ten year summary on the use of porous asphalt in the Netherlands. Padmos states that the use of porous asphalt is attractive because of the noise reducing capabilities of this mix type compared to dense-graded mixes. He states that approximately 57 percent of the motorway network within the Netherlands is surfaced with porous asphalt. Any time a layer must be replaced, the current policy is for porous asphalt to be used as the wearing surface. Up until the writing of this paper, only single layer porous asphalts were being used. However, there was a push to use double layer porous asphalt wearing surfaces because of the noise reduction capabilities.

1.43.2 Benefits of Permeable Asphalt Mixtures
Benefits mentioned by the author include noise reduction and reduced splash and spray during wet weather. Padmos indicates that porous asphalt reduces noise levels by 3 dB(A).

1.43.3 Materials and Mix Design
Typical gradations used in the Netherlands have a maximum aggregate size of 16 mm.

1.43.4 Construction Practices
No specifics on construction practices were given.

1.43.5 Maintenance Practices
Padmos states that during winter events, all lanes on porous asphalt are closed except for the design lane. The reasoning being that the traffic will help prevent snow and ice

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buildup if traffic is concentrated into one lane. Padmos also states that there is no definitive solution for winter maintenance; however, the addition of brine is a must. He also states that the concentration of traffic within the design lane allows the traffic to “suck” the brine solution out of the void structure of the porous asphalt onto the pavement surface.

1.43.6 Rehabilitation Practices
Raveling is the major cause for rehabilitation. However, Padmos does not provide any techniques for rehabbing porous asphalt.

1.43.7 Performance
Padmos states one of the problems with porous asphalt is the low frictional resistance of these layers immediately after construction. After 3 to 6 months, the asphalt binder film coating the aggregates at the surface wears away and friction significantly increases.

Raveling is the major distress that influences performance. Due to raveling, the service life of porous asphalt is generally 10 to 12 years.

1.43.8 Structural Design
Porous asphalt is constructed to a thickness of 50 mm in the Netherlands. Padmos also states that porous asphalt provides approximately 50 percent of the structural capacity of typical dense-graded wearing layers.

1.43.9 Limitations
No specific limitations were given.

1.44 Ranieri, Vittorio, Runoff control in porous pavements, Transportation Research Record No: 1789, Transportation Research Board, National Research Council, Washington, D.C. 2002.

1.44.1 General
In this paper, Ranieri presents a model linking hydraulic conductivity of porous pavement with the geometrical characteristics of the road section and the rainfall intensity. The author supports the model with verification results from laboratory tests. The tests consisted of simulating rainfall on porous pavements and measuring the depth of water over the impervious layer during the seepage motion. Based on these experiments, Ranieri provides a chart for design of porous pavements. The chart provides a relationship between a function of the maximum depth of the water table over the impervious layer and the length (L) of the seepage path and a function of rainfall intensity and permeability, for different values of slope of the porous pavement.

Note that since this paper contains models, design chart and example, some of these have been scanned and presented as they are in separate figures. The reviewer feels that this is important for full understanding of the material presented in this paper.

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1.44.2 Benefits of Permeable Asphalt Mixtures
No benefit of permeable asphalt mixture is mentioned.

1.44.3 Materials and Design
No information on materials and design has been presented

1.44.4 Construction Practices
No information on construction practices has been presented.

1.44.5 Maintenance Practices
No information is provided on maintenance practices of friction course.

1.44.6 Rehabilitation Practices
No information is provided on rehabilitation practices of friction course.

1.44.7 Performance
No information is provided on performance of friction course

1.44.8 Structural Design
Ranieri starts with a list of models developed for design of hydraulic conductivity related models for structural design– specifically thickness and slope of porous pavements. The relationships are shown in Table 94. Ranieri mentions that all of these models are for dense-graded overlays and are not applicable for porous pavements with open-graded courses, and that the attempt by Ross and Russam (1968, see Table 94) to modify an existing relation with a coefficient to take care of the type of overlay has not been successful.

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Table 94: Models Relating Hydraulic Conductivity to Structural Design of Porous Pavements

Author/Researcher/Year	Topic/Model/Equation
C. F. Izzard/1942	Airport runway runoff (model not provided in this reference).
N.F. Ross and K. Russam/1968	Model relating the height of water film (h) over the grains to the length (L) and slope (i) of the pavement and to the rainfall intensity: $H = 0.015*(\sqrt{LI}/\sqrt[5]{i})$; $H = 0.015*(\sqrt{cLI}/\sqrt[5]{i})$; c = coefficient for the capacity of infiltration within the various kinds of new overlayers.
R. Laganier/1977	$Hr = a \cdot t^{-b}$, where Hr is the height of water brought to its maximum value at equilibrium, t is time in seconds and a, b are coefficients affecting drainage.
F. Giannini and A. Noli/ 1974, L. Domenichini and G. Remedia/1994	Theoretical models (not provided in this reference)
Franklin Institute, 1972, T. J. Jackson and M. R. Ragan, 1974	Improved understanding of rain water discharge through porous pavements, model of full depth porous pavements

Ranieri differentiates porous pavements into two main types: full depth porous pavements and pavements in which only the wearing course is porous. He mentions that both types of pavements can help in avoiding the detrimental effects of water on the road surface by storing and draining water, with drainage being the predominant mechanism in open-graded friction courses.

Ranieri then presents the assumptions of his model of rainwater flow through porous pavements. He contends that rain water flow through porous pavements can be studied as an unconfined aquifer moving within a homogeneous porous medium that lies over an inclined impervious layer and is subjected to constant replenishment along the flow path. He formulates the problem as one of finding a solution given the specific boundary conditions. Citing his work on theoretical groundwater flow model specific for porous roads, Rainier mentions that this paper presents the latest studies with this model.

Rainier then presents the model, solutions and charts based on the solution of the equation describing the model. For convenience, the equations, solutions and chart are shown in Figure 17. The chart relates Hmax/L to 4I/k for different slopes. Specifically, the chart gives the minimum thickness of the porous course so that rain water always discharges within it, if the design rainfall rate (I), the geometric characteristics of the road (longitudinal gradient, cross slope and width) and the permeability k of the porous asphalt are provided.

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Statement of the problem and boundary conditions

The model considers a road carriageway having cross slope i and width L, with a porous friction course of thickness T, permeability k and porosity n; between the friction and the base courses there is a Stress Absorbing Membrane Interlayer (SAMI) (see Figure 1).

During rainfall at a constant rate I, rainwater moving within the porous course can be considered as an aquifer uniformly replenished from the top, flowing in steady flow conditions into a unconfined inclined system. In these conditions the law of motion can be described using the Boussinesq equation the general form of which, referring to the system of Cartesian axes and symbols in Figure 1, is (*14*):

$$\frac{\partial h}{\partial t}=\frac{k}{n}\left[\frac{\partial}{\partial x}\left(H\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(H\frac{\partial h}{\partial y}\right)+\frac{v_0}{k}\right] \tag{3}$$

where: $v_0 = I$ = infiltration rate $[L{\cdot}T_{-1}]$ and h [L] and H [L] are the depth of water measured on the horizontal and inclined layers respectively. With the given hypothesis and boundary conditions, eq. (3) becomes:

$$\frac{d}{dx}\left(Hk\frac{dh}{dx}\right)=-I \tag{4}$$

Solutions of the model

Solving eq. (4), two different solutions are obtained depending on the sign of the quantity $\Delta = i^2-4\frac{I}{k}$:

$$\left[\left|A-\frac{i+\sqrt{\Delta}}{2}\right|^{\frac{1}{2}\left(\frac{i}{\sqrt{\Delta}}-1\right)}\right]\cdot\left[\left|A-\frac{i-\sqrt{\Delta}}{2}\right|^{-\frac{1}{2}\left(\frac{i}{\sqrt{\Delta}}+1\right)}\right]=c\cdot(L-x) \qquad \text{for } \Delta>0 \tag{5}$$

and

$$\log\left[\left(\frac{I}{k}\right)^2\cdot(x-L)^2\cdot\left(A^2-iA+\frac{I}{k}\right)\right]-\frac{2i}{\sqrt{-\Delta}}\operatorname{arctg}\left(\frac{2A-i}{\sqrt{-\Delta}}\right)=c \qquad \text{for } \Delta<0 \tag{6}$$

where $A=\frac{h-iL}{x-L}$ and c is the constant of integration.

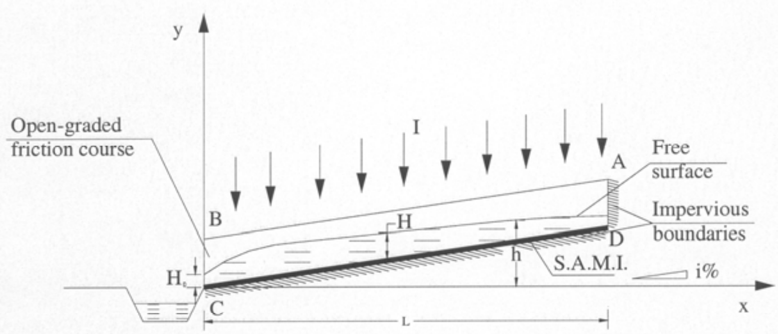

FIGURE 1 **Schematic representation of the runoff model.**

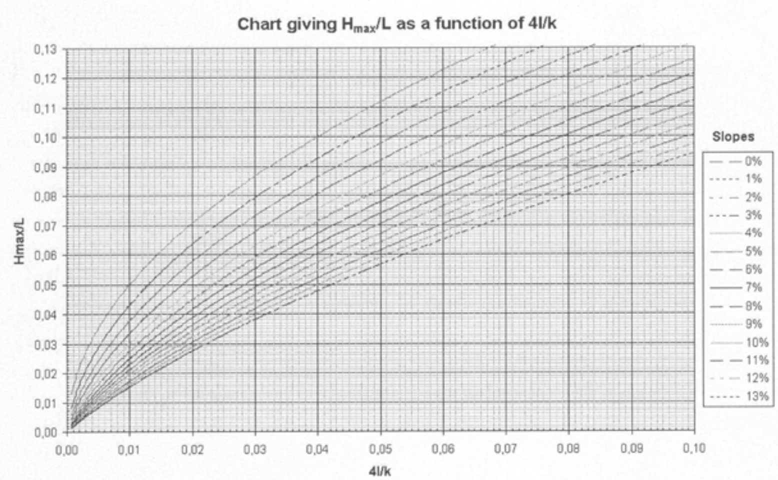

FIGURE 3 **Chart giving the values of H_{max}/L as a function of the ratio 4·(I/k).**

Figure 17. Model and chart from Ranieri's work

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Ranieri also provides details of a laboratory experiment carried out to evaluate his model. Ranieri conducted tests with three different materials that he had used originally - one porous asphalt and two unbound aggregates. Each material was tested for different values of slope i and rainfall intensity I; the piezometric heads along the flow path were observed and the corresponding free water surfaces were plotted. In addition each of these plots was compared to the corresponding surfaces mathematically obtained by the application of the model. The properties of the materials are shown in Table 95.

Table 95: Properties of One Porous Mix and Two Unbound Aggregates

Material	D_{max}, mm	Porosity, %	Permeability, cm/s
A (porous mix?)	0.40	41	0.025
B	Not applicable	22	0.10
C	0.15	42	5

The testing/simulating device consisted of a rain simulator positioned over a 1450 mm x 750 mm reclining basin containing the porous pavement material to be tested. The thickness of this material can be up to 70 mm. Water table depth can be assessed with seven piezometers fixed at the bottom of the basin.

Ranieri indicates that the experimental results fit well with the predicted results (from the model) if a coefficient β is introduced in the theoretical model. The theoretical background of this coefficient, as postulated by Rainier, is that this coefficient is needed to compensate for the fact that the flow during the experiment is not laminar but of transitional nature. Two important points are that the coefficient is not constant, since in this case there is constant replenishment of flow from the top, and that the sole coefficient, β, is sufficient for adjusting the theoretical model.

Ranieri then provide details of experiments carried out to assess the value of the coefficient β. These experiments were carried out with three unbound materials. The properties of these materials are shown in Table 96. From these tests, Rainier provides an equation relating β to i, slope and I/K_d, where I is the rate of rainfall (intensity) and K_D is the Darcy's permeability (coefficient of permeability).

Rainier then provides a design example to use the chart to determine minimum thickness of a porous pavement. This example is shown in Figure 18.

Table 96: Properties of One Porous Mix and Two Unbound Aggregates

Material	D_{max}, mm	Porosity, %	Permeability, cm/s
S	0.85	44	0.015
G	5	46	2.31
P	10	47	4.8

1.44.9 Limitations

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DESIGN EXAMPLE

A road section is given for which $L_c = 7.00$ m is the carriageway width and $i_l = 2.5\%$, $i_c = 3\%$ are the longitudinal gradient and the cross slope respectively. During rainfall, water will flow along the line of maximum slope whose length (L) and inclination (i) are respectively:

$$L = L_c\left[1+\left(\frac{i_l}{i_c}\right)^2\right]^{1/2} = 9.10\text{ m} = 910\text{ cm} \quad \text{and}$$

$$i = \left(i_c^2 + i_l^2\right)^{1/2} = 3.9\%$$

$I/k_D = 0.00035$;
and, from eq. (14):
$\beta^* = \{[-0.013\cdot[\ln(0.00035)]^2 - 0.226\ln(0.00035) - 0.96]\}3.9 + 63.19\cdot(0.00035)^{0.6} = 0.593$
So the value of the quantity $4\cdot I/k$ to be entered in the chart is:
$4\cdot I/k = 4\cdot I/(\beta^*\cdot k_D) = 0.00236$.
As shown in Figure 9, the value $4\cdot I/k = 0.00236$ has to be entered vertically in the chart until the curve relative to the design slope ($i = 3.9\%$). Than, following horizontally, it is possible to read on the ordinate axis the corresponding value of the ratio $H_{max}/L = 0.0095$.

Therefore, in order to guarantee that the water depth would not exceed the thickness of the friction course along the whole runoff path, this must be:
$T \geq H_{max} = 0.0095\cdot L = 0.0095 \cdot 910 \approx 8.6$ cm.

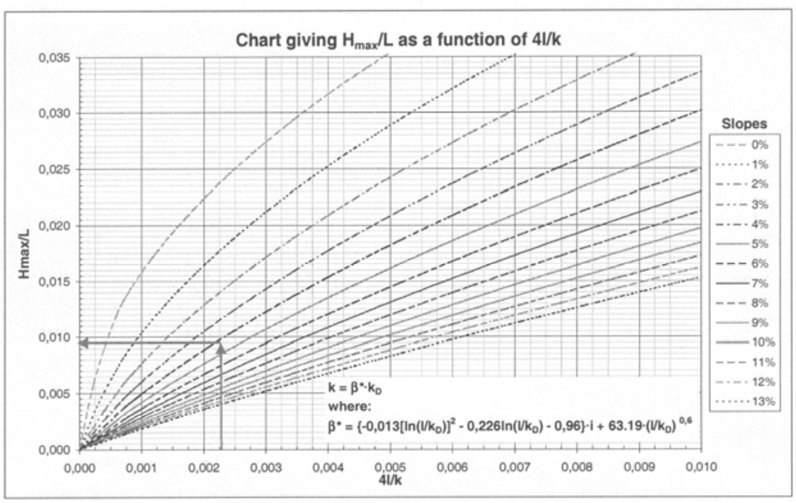

FIGURE 9 **Use of the chart giving the values of H_{max}/L as a function of the ratio $4\cdot(I/k)$.**

Figure 18. Design example

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1.45 Rogge, D. "Development of Maintenance Practices for Oregon F-Mix." Oregon Department of Transportation. FHWA-OR-RD-02-09. Corvallis, Oregon. 2002.

1.45.1 General

This report documents a research study conducted to evaluate maintenance practices for use on OGFC layers. In Oregon, OGFC mixes are referred to as F-mixes and have been used as wearing layer since the 1970's. Experience has shown that standard pavement maintenance practices for dense-graded mixes tend to reduce the drainage capacity of OGFC mixes and change the noise and ride characteristics of the road.

The specific questions to be answered during this research study included:
1. Are fog seals the best preventative maintenance strategy for OGFCs? If so, what is the optimum frequency and procedure?
2. Are better techniques available than those that have historically be used for dense-graded layers?
3. Can OGFCs be obtained in small enough quantities such that they can be used for small repair and patching activities?
4. What maintenance activities should be conducted during snow, ice and freezing fog?

The organization of the report includes an initial survey of Oregon Department of Transportation (ODOT) maintenance personnel to determine the most common distresses encountered with F-mix, then documentation of common repairs for these distresses, and then a conclusions and recommendations.

1.45.2 Benefits of Permeable Asphalt Mixtures

Benefits cited by Rogge include reduced hydroplaning, splash and spray, rutting and tire/pavement noise.

1.45.3 Materials and Mix Design

No specifics on materials and mix design were given.

1.45.4 Construction Practices

No specifics on construction practices were given.

1.45.5 Maintenance Practices

One section of the report dealt specifically with the use of fog seals during maintenance. Of the 78 respondents, 56 percent of the respondents indicated that they did not utilize fog seals while 41 percent indicated that fog seals were used (3 percent did not respond) (Figure 19). Of the maintenance supervisors that did use fog seals; over half indicated that fog seals had been placed on less than 30 percent of their pavements containing F-mix (Figure 20). Of the 78 respondents to the survey, Figure 20 also shows that roughly one-quarter of the maintenance supervisors have utilized fog seals on up to 30 percent of the pavements using F-mix as a wearing course. This led the author to the following statement, "… the survey confirmed that ODOT continues to lack convincing evidence

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about the effectiveness of fog sealing as a preventative maintenance technique for F-mix."

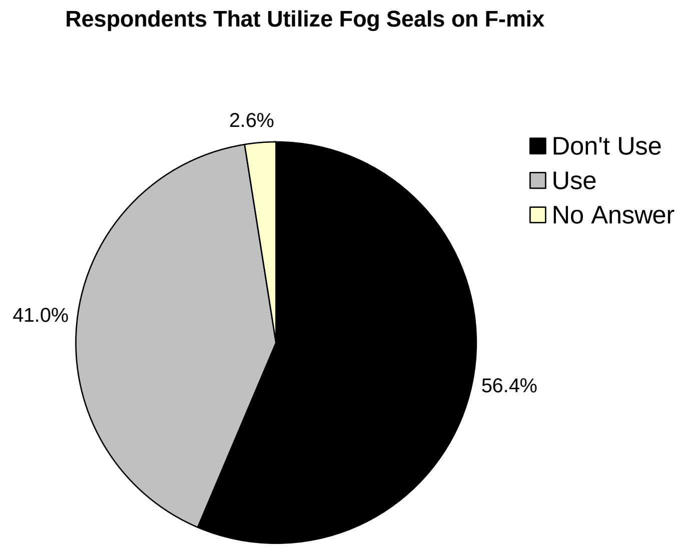

Figure 19: Response to Whether ODOT Maintenance supervisors Utilize Fog Seals

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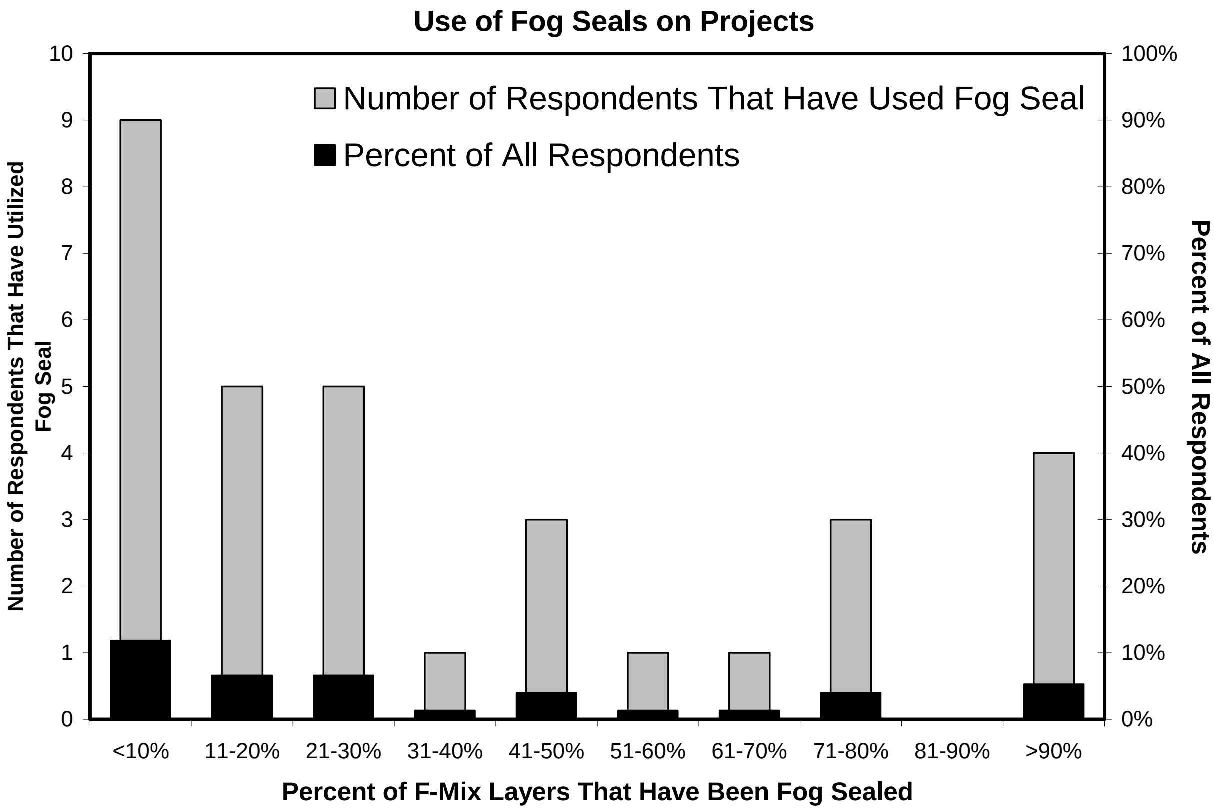

Figure 20: Use of Fog Seals as Preventative Maintenance

Another portion of the survey requested the responding maintenance supervisors to rank the effectiveness of 16 different maintenance treatments. Figure 21 presents the results of this survey. The scale utilized for this portion of the survey was from 1 to 4. A rating of 1 indicates a maintenance technique that is "not at all successful" and a rating of 4 indicated a technique that was "completely successful." Based upon Figure 21, the top six techniques had an average score ranking between somewhat successful and completely successful. These techniques included C-mix (a dense-graded HMA) machine patch, F-mix inlay, B-mix (dense-graded HMA) machine patch, C-mix inlay, C-mix screed patch, and C-mix blade-in. The remaining ten maintenance techniques were rated somewhere between "not very successful" and "somewhat successful." One concern of the results shown in Figure 21 was that it might not be feasible or economically advantageous for the use of F-mix in order to correct minor surface defects during maintenance. Therefore, the author surveyed the ability of ODOT to obtain 60 tons of F-mix for maintenance applications. Results of this survey indicated that 46 of the 78 maintenance supervisors said that they could not get such a small quantity of F-mix. Nine of the 78 indicated that they could order 60 tons while 23 did not know.

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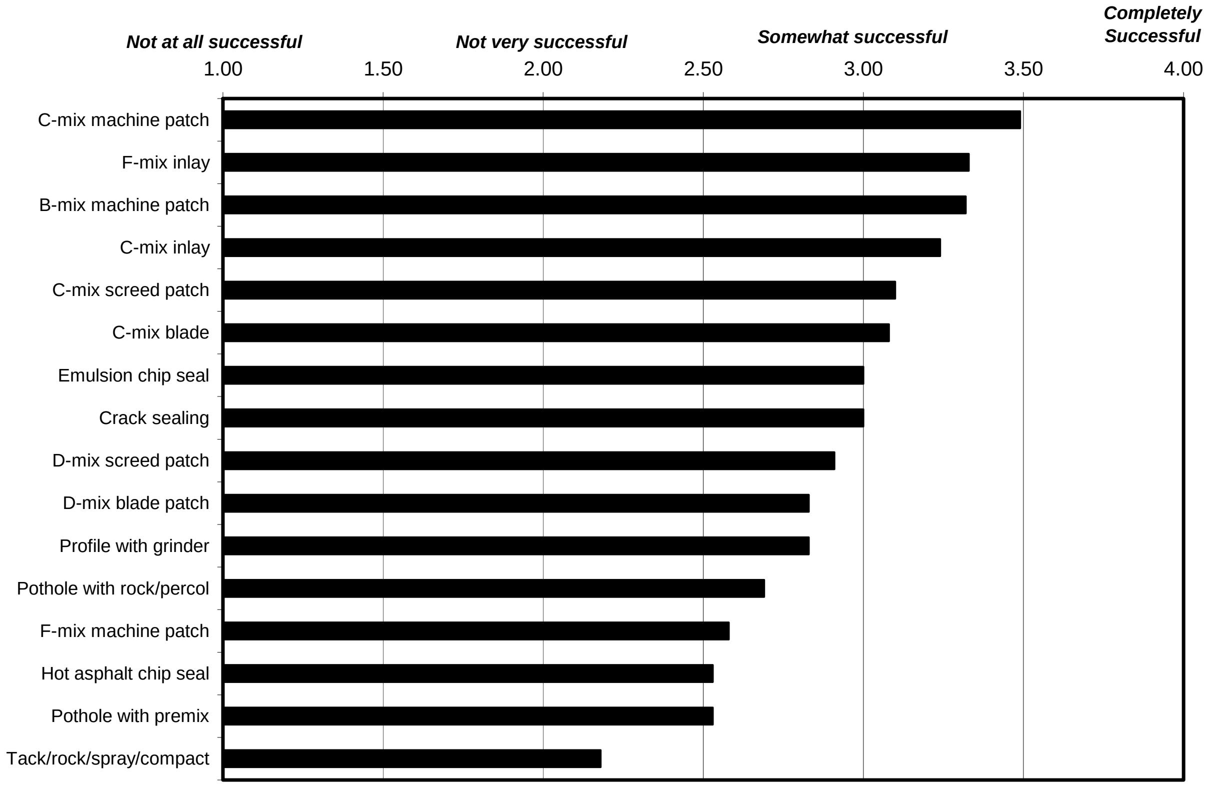

Figure 21: Rankings of Maintenance Techniques for F-Mix

Another portion of the survey dealt with common winter maintenance techniques. Table 97 presents the most common techniques for winter maintenance. Sanding and de-icing agents, either alone or in combination, were the most common method of winter maintenance techniques reported. It was noted, however, that there is a need for more frequent and longer application for the porous F-mix as compared to dense-graded surfaces. Some specific comments related to sanding operations included: "You have to stay on top of it," "It tends to plug it up," Sanding is a last resort," "Sanding is less successful than deicer," and "Cinders are best – they go down into pores more easily."

Specific comments relating the use of deicers in general included: "You have to do it within 60 minutes before a storm. We don't do pre-treating because it's not effective. The material go down into the asphalt," and "They are all effective, but magnesium chloride is more temperature sensitive than CMA."

Specific comments related to the use of magnesium chloride included: "It is more effective than CF7 or CMA," "Pre-treating with magnesium chloride works best as a deicer," "Need to apply it twice as heavy," and "We've been really effective with it."

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Specific comments relating to the use of CMA included: “Use CMA for frosting,” “CMA is temperature and moisture sensitive,” and “Thirty percent solution CMA has proven somewhat successful.”

Table 97: Typical Winter Maintenance Techniques for F-Mix

Treatment	Number
Sanding	20
Liquid de-icer or anti-ice agent	15
Magnesium Chloride	15
CMA	7
Larger quantity of de-icer	7
Run shoes on plows	5
Reduce plow speeds	2
Rubber bits	1
CMA & CF7	1
Magnesium Chloride & CF7	1

1.45.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.45.7 Performance
The initial part of the study was a survey designed to determine the most common types of distresses encountered in F-mixes. Responses to the survey were returned from 78 of 83 maintenance supervisors, for a 94 percent response rate. Results from the 2001 survey are presented in Figure 22. Within this figure, the various maintenance engineers were requested to rank the various distresses by their frequency using a ranking system of 1 to 4. The higher the ranking, the more frequent the distress was encountered. A ranking of 1 indicates that the distress is never seen and a ranking of 4 indicates that the distress is pervasive. Based upon the survey, the top seven distresses were considered closer to scattered than to rare. These seven distresses included tire stud rutting, icing problems, raveling, gouging/scarring, deformation rutting, clogging and potholes. The other seven distresses included in Figure 22 were considered closer to rare than scattered.

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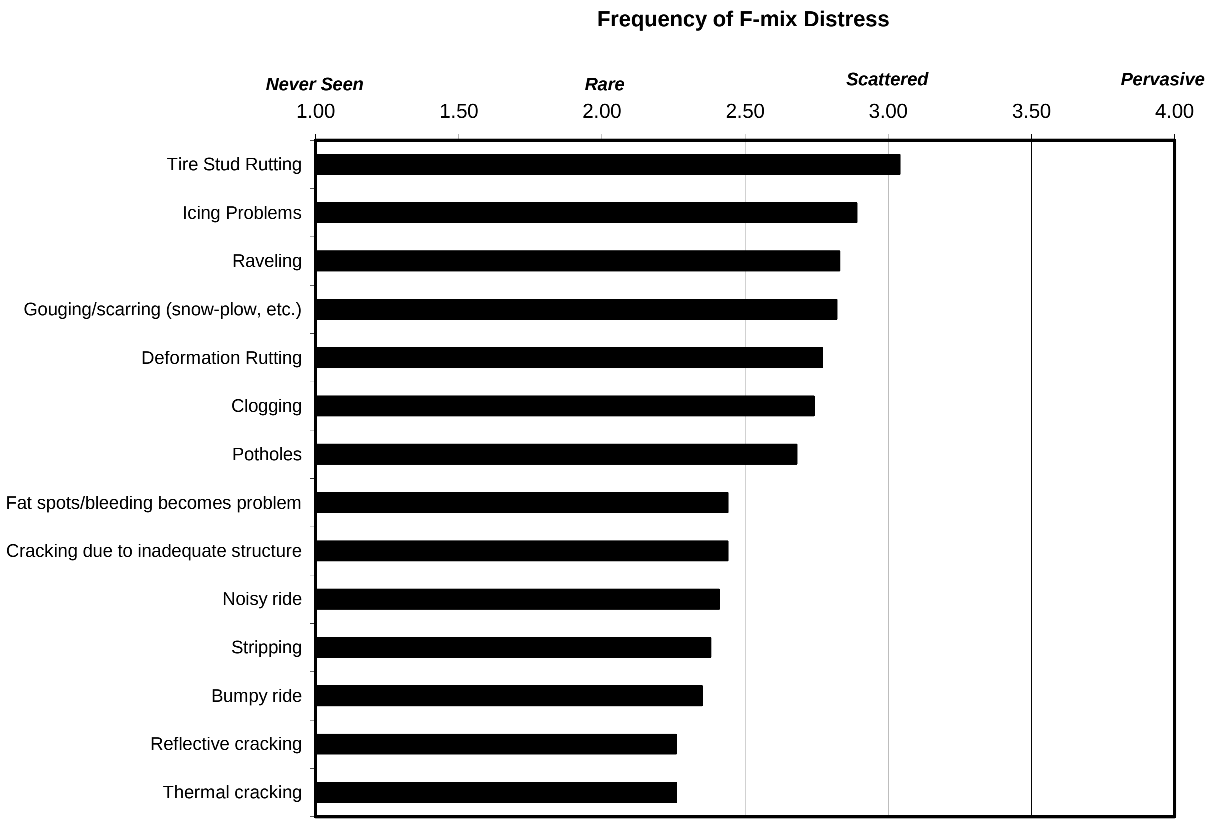

Figure 22: Results of 2001 Survey of ODOT Maintenance Supervisors

1.45.8 Structural Design
No specifics on inclusion within structural design were given.

1.45.9 Limitations
No limitations on use were given.

1.46 Flintsch, G. W., E. de León, K. K. McGhee, I. L. Al-Qadi. "Pavement Surface Macrotexture Measurement and Application." Transportation Research Record No: 1860. Transportation Research Board. National Research Council. Washington, D.C. 2003.

1.46.1 General
This paper does not address any specific feature of porous friction course, but provides results and comparison of results obtained through different types of measurements for macrotexture properties of three mix types, one of which is an OGFC. The authors show correlations of macrotexture measurement results from the CTM meter and the laser profiler method, with results from the sand patch tests for different mixes.

Of the different things present in these comparisons, the most important observation, relevant to this literature review is that the OGFC mix shows the highest macrotexture values (in mm) for both sand patch and laser profiler measurements. (The authors do not

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indicate the OGFC mixes on the plot showing the comparison of results from the CTM meter and the sand patch test).

Flintsch et al also evaluated the applicability of an equation based on material properties for estimating texture depth in non segregated hot mix asphalt (HMA). From a comparison of estimated depths and average results from sand patch measurements, they conclude that the relationship cannot appropriately predict the macrotexture for the mixes studied, especially for OGFC and stone matrix asphalt (SMA) mixes.

1.46.2 Benefits of Permeable Asphalt Mixtures

No benefit of permeable asphalt mixture is mentioned.

1.46.3 Materials and Design

Flintsch et al provides information on several techniques of macrotexture/friction/segregation measurements and gradation and material information on the different mixes (including OGFC) used in this study. The different test methods and equations are shown in Table 98, and the material properties of the OGFC mix are shown in Table 99.

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Table 98: Measurement Methods and Equations

Property	Method
Macrotexture	Static: Sand patch, ASTM E965; Mean: Outflow meter: indirectly estimates pavement texture based on the time for a fixed volume of water to escape from a measured cylinder with a rubber bottom; The Circular Track Meter, or CTMeter; Dynamic: Vehicle-mounted laser device ASTM Standard E1845.
Microtexture	Low speed friction measurement devices such as the British Portable Tester (BPT), the Dynamic Friction Tester (DF Tester), and the locked wheel skid trailer when testing is performed at low speeds; skid trailer measurements conducted using a ribbed tire (ASTM E501);
Skid Resistance/Friction	International Friction Index (IFI): To calculate the IFI, it is necessary to have at least one friction measurement and one macrotexture measurement. The IFI is reported in two parameters: the normalized wet friction value at 60 km/hr (F60) and a speed constant (Sp). A transformation equation has also been established to allow for calculation of the wet friction value at speeds other than 60 km/hr. If the measurements are conducted using a ribbed tire, which is relatively insensitive to macrotexture properties of the surface, then the macrotexture measurement is also used to correct the normalized wet friction value at 60 km/hr (F60). The adjusted friction value and the texture measurement are used to calculate the value for F60.
Non-Segregated ETD of a HMA	NCHRP 441: $ETD = 0.01980*MS - 0.004984*P\,4.75 + 0.1038*C_c - 0.004861*C_u$ MS = maximum size of the aggregate (mm); *P4.75* = Percentage passing 4.75mm sieve; C_c = coefficient of curvature = $(D_{30})^2/(D_{10}D_{60})$; C_u= coefficient of uniformity = D_{60}/D_{10}; D_{10} = the sieve size associated with 10% passing (mm); Equation presented in this paper, based on this study: EPD = -2.896 + 0.2993*NMS+0.0698*VMA, where, EPD = estimated mean profile depth, NMS = nominal maximum size (mm); and VMA = voids in the mineral aggregate (%).

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Table 99: OGFC Properties

Material/property	Type/value
Binder	PG 76-22
Asphalt binder content	5.5
Maximum size, mm	19
Nominal maximum aggregate size (NMAS), mm	12.5
Percent passing the 9.5 mm sieve	81
Percent passing the 4.75 mm sieve	14
Percent passing the 2.36 mm sieve	2
Percent passing the 1.18 mm sieve	1.4
Percent passing the 0.6 mm sieve	1.3

1.46.4 Construction Practices
No information on construction practices has been presented.

1.46.5 Maintenance Practices
No information is provided on maintenance practices of friction course.

1.46.6 Rehabilitation Practices
No information is provided on rehabilitation practices of friction course.

1.46.7 Performance
With respect to performance of OGFC, several observations were made from the data. Flintsch et al conducted macrotexture measurements on OGFC sections using CT Meter and laser profiler, conducted tests with laser profiler to estimate mean profile depths for validation of speed constant equation, used an existing equation to predict texture depth for unsegregated areas in HMA, and finally developed a new equation to predict mean texture depth in unsegregated areas in different types of HMA, including OGFC.

In the comparison of results from different test methods, Flintsch et al showed that the macrotexture of the OGFC was the highest, as evident from results of sand patch tests, CT Meter and laser profiler tests. In their data for IFI speed constant equation validations, Flintsch et al showed that the OGFC sections had the highest macrotexture and the lowest percent normalized gradient (PNG). Note that the gradient is inversely proportional to the pavement macrotexture.

1.46.8 Structural Design
No information is provided on structural design of friction course.

1.46.9 Limitations
No information is provided on limitations of use.

1.47 Kaloush, K. E., M. W. Witczak, A. C. Sotil and G. B. Way. "Laboratory Evaluation of Asphalt Rubber Mixtures Using the Dynamic Modulus (E*) Test." TRB 2003 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2003.

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1.47.1 General

In this paper, Kaloush et al provides the results of a study conducted to evaluate the dynamic modulus of two asphalt rubber mixtures, one of which was an OGFC. The authors conducted testing at low and high temperatures, and developed master curves for these mixes.

Kaloush et al compared the results for the OGFC with results obtained from tests on dense-graded mixes. They concluded that under unconfined conditions the asphalt rubber mixes showed lower modulus values and, hence, better resistance to cracking, whereas under confined conditions they showed higher modulus values and, hence, better resistance against rutting. However, the improvement in resistance at higher and lower temperatures was not as enhanced in the OGFC as it was in the other mix. On the other hand, the difference between modulus values obtained under confined and unconfined was most pronounced for the OGFC mixes.

Kaloush et al concluded that the dynamic modulus test was appropriate for performance evaluation of different asphalt mixes, including asphalt rubber mixes (and OGFC mixes), but recommended that when comparing mixes with both dense and open gradations, the use of confined testing should be made.

1.47.2 Benefits of Permeable Asphalt Mixtures

No information on benefits of permeable asphalt mixtures is provided. However, in the introduction Kaloush et al mention some of the benefits of using crumb rubber in different mixes, including OGFCs. These benefits include adequate resistance against reflection cracking, high temperature conditions of the Arizona desert, cold conditions at higher elevations (e.g. Flagstaff, AZ), reduction in tire noise, and improvement in ride quality.

1.47.3 Materials and Design

Kaloush et al mention that samples of two different types of mixes were obtained during construction for laboratory testing. The mixes were Asphalt Rubber Asphalt Concrete (ARAC) Gap-Graded mixture, and Asphalt Rubber Asphalt Concrete Friction Course (AR-ACFC) Open-Graded mixture. The nominal aggregate size of the ARAC and the AR-ACFC mixes were 19.0 mm and 9.0 mm respectively, whereas the average air voids were 11 and 18 percent, respectively. The mixes were provided with a base binder grade of PG 58-22 grade.

1.47.4 Construction Practices

Kaloush et al indicates that the samples of the two mixes were obtained from a construction project on I-40 (on the Buffalo Range paving project) east of Flagstaff, Arizona. They mention that these ARAC and AR-ACFC mixes, which are typically used in Northern Arizona, are placed in 50 mm and 12.5 mm thickness respectively. With the AR-ACFC mix placed on top of the ARAC mix.

1.47.5 Maintenance Practices

No information is provided on maintenance practices of friction course.

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1.47.6 Rehabilitation Practices

No information is provided on rehabilitation practices of friction course.

1.47.7 Performance

Kaloush et al provide background information on the dynamic modulus test procedure (ASTM D3497) that they used for determination of modulus, E* and phase angle, φ. Test specimens were obtained by coring 150mm tall by 100mm diameter samples from 150mm tall and 150mm diameter Superpave gyratory compacted samples. The test matrix is shown in Table 100.

Table 100: Test Matrix

Testing Condition/samples	Values/numbers
Confinement	0, 10, 20, and 30 psi
Frequency	0.1, 0.5, 1, 5, 10, and 25 Hz
Temperature	14, 40, 70, 100, and 130°F
Number of Samples	6 for ARAC; 12 for ARACFC; 1 formed (molded) crumb rubber specimen consisting of 80% crumb rubber and 20% urethane.
Sequence of Testing	Each specimen was tested in an increasing order of temperature, i.e. 14, 40, 70, 100, and 130°F. For each temperature level, specimens were tested in a decreasing order of frequency (25, 10, 5, 1, 0.5, and 0.1 Hz).

Note: The authors used the specimen instrumentation method that was developed by the Arizona State University Research Team. In this method, the LVDT's were secured in place using brackets and studs glued on to the specimen; guiding rods were added to the instrumentation for alignment especially at high temperatures; the load was varied with temperature to keep the specimen response within a linear range (initial microstrains about 20-25 micro-strain).

For the ARAC mix, Kaloush et al note that the comparison of the results for the unconfined and confined tests show that there is a significant increase in the E* values with confinement at higher temperatures and lower frequencies, compared to the low temperature part of the curve. The difference in E* results between the unconfined and confined tests at higher temperatures become less as the confinement is increased: 400percent increment from unconfined conditions to a 10-psi confinement, 25 percent from 10 to 20-psi confinement, and 11 percent increment from 20 to 30-psi confinement. They mention that the difference between the unconfined and confined tests at lower temperatures is much less, but still significant between the different levels of confinement. Kaloush et al indicate that there were cases where the E* value of the specimens at 100 or 130°F were equivalent. The authors hypothesize that this is due in large measure to the decreased role of the asphalt binder in relationship to the increased role of the rubber particles at higher temperatures. They attempted to prove this by conducting a series of tests on a crumb rubber specimen (see Table 100). From the E* results obtained from this specimen at 70, 100, 130°F, the authors note that the E* values

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remain almost the same throughout the test at the different frequencies (loading time) and the three test temperatures.

For the AR-ACFC mixes, Kaloush et al indicate that the trends of the results were similar to those of the ARAC mixes. They note a significant increase in the E* values with confinement at higher temperatures and lower frequencies, compared to the low temperature part of the curve. The authors mention that the difference in E* results between the unconfined and confined tests at higher temperatures stay significant as the confinement level is increased: 250 percent increment from unconfined conditions to a 10-psi confinement, 61 percent from 10 to 20-psi confinement, and 62 percent increment from 20 to 30-psi confinement. Kaloush et al point out an important property noted for the AR-ACFC mixes only - the difference between the different levels of confinements is negligible at lower (cold) temperatures, but the difference between the unconfined and confined tests is still significant (unlike the ARAC mix) at the lower temperatures.

Kaloush et al provide the results of a study carried out to compare dynamic modulus values of asphalt rubber mixes (used in this study) versus dynamic modulus values of conventional ADOT dense graded mixes with PG 76-16 and PG 64-22 binders. The PG 76-16 mixture showed higher modulus values at all temperature and frequency conditions, whereas the E* values of the asphalt rubber mixes were found to be more comparable to those of the conventional PG 64-22 mixture. The authors note that these comparable values (asphalt rubber mixes showed higher modulus values at higher temperatures and lower modulas values at lower temperatures) were observed despite the fact that there was a significant air void difference between the asphalt rubber and the dense graded mixes. Kaloush et al concludes that this observation supports the superior performance of asphalt rubber mixes against high temperature rutting and low temperature cracking. They also note that the lower modulus values of the asphalt rubber mixes (at al temperatures) compare to the PG 76-16 mixes could very well be due to (at least partially) the impact of higher air voids in the asphalt rubber mixes. They recommend the comparison of results from samples with similar void levels.

Kaloush et al then indicate that a comparison between different dynamic modulus values (dense-graded and asphalt rubber gap and open graded mixes) was conducted by selecting a reference mix. This comparison was carried out for results obtained at 14°F and 100°F, both at 10 Hz. The modular ratio (R) was calculated using the following equation R = E*mix/E*Reference, where R = Modular Ratio, E*mix = Dynamic Complex Modulus value for a given mixture, E*Reference = Dynamic Complex Modulus value for the reference mixture. For low temperature performance, the authors ranked the mixes in terms of increasing E* values (that is decreasing performance). For high temperature performance, the authors raked them in the reverse way – according to decreasing E * values (that is decreasing performance). First, the PG 64-22 conventional mix was taken as the reference mix. Kaloush et al mention that that asphalt rubber mixes have the lowest E* values (lowest modular ratio), at low temperatures. For the high temperature ranking, the asphalt rubber mixes were also found to have the lowest E* values – the authors point out that this may be due to unconfined testing and higher air

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voids. They also contend that these rankings are in contrast to observed excellent high temperature performance of the asphalt rubber mixes.

To refine the comparison, Kaloush et al then used the results of the confined tests. Using results from tests conducted at 20 psi confinement (for both conventional and asphalt rubber mixes), and using the ARAC mix as the reference mix, the authors show that the asphalt rubber mixes show lower modulus values at low temperature and higher modulus values at high temperature (compared to dense graded mixes) and therefore, rank, above conventional mixes, in terms of both low and high temperature performance.

Citing the above results, Kaloush et al argues that since these results are in line with field performance results, the use of confined test results (for E*) is more appropriate for evaluating mixes which are gap- and open-graded courses.

1.47.8 Structural Design
No information is provided on structural design of friction course.

1.47.9 Limitations
No information is provided on limitations of use of friction course.

1.48 Poulikakos, L.D., Takahashi, S. and Partl, M.N. "A Comparison of Swiss and Japanese Porous Asphalt through Various Mechanical Tests." *3rd Swiss Transport Research Conferenc*e. Monte Verita/Ascona. March 19-21, 2003

1.48.1 General
This paper outlines a co-operative research project between Japanese and Swiss agencies to determine if new mix designs (SPPA & JPPA) for porous asphalt performed better than conventionally used porous asphalt mixes (SPA & JPA). The new mix designs were based on the Dry Packing Method (DPM) and Wet Packing Method (WPM). Aggregate gradations of these new mixes (SPPA & JPPA) are shown graphically in this report.

The gradations of the SPPA and JPPA had a target porosity of 22%. The SPPA was constructed 4.1% (by mass) Styrelf 13/80 penetration grade SBS polymer modified Bitumen. The Conventional Swiss mix (SPA) used 4.8% by mass 50/70 penetration graded straight run bitumen with Trinidad NAF 501. Both the new and conventional Japanese mixes used a polymer modified bitumen with 9% SBS. A comparison of the binder is as follows:

Property	JPA and JPPA	SPA	SPPA
Penetration 1/10 mm	45	15	50
Ring and Ball °C	90.1	80.7	56.5

Several tests were carried out by Poulikakos et al to compare the new and conventional porous asphalt mixes. Specifically the following tests were conducted, although not all were discussed in this paper:

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- Laboratory aging of mixes (short and long term)
- Cantabro (particle loss test)
- Water permeability
- Binder penetration
- Binder softening point
- Rheology of the binder using the DSR
- Interlayer shear strength
- Indirect tensile strength
- Shear modulus of mixes using the Coaxial Shear Test (CAST)
- Wheel Tracking

The following observations were made based on these tests:

1. After long term aging binder draindown was observed. Thus long term aging is not an optimal procedure for porous asphalt.

2. The JPA and JPPA mixes maintained a higher porosity after over compaction as compared with the SPA and SPPA. The authors concluded that the difference may be the binder content differences between the mixes, thus emphasizing the importance of correct binder selection.

3. Using a constant head permeameter, the mixes were evaluated for permeability at compaction and over compaction. The SPPA and JPPA mixes did not show an improvement in water permeability.

4. The JPA and JPPA performed well in the wheel tracking tests, which the authors believe is due to the binder in the mix. No significant additional contribution was discovered in the SPPA mix during the wheel tracking test.

5. From the indirect tensile tests it was shown that the JPA and JPPA had higher tensile strength (both wet and dry) than the SPA and SPPA mixes. There was no significant difference in the water susceptibility of the four mixes. Water conditioning had a significant effect on the SPA mix, suggesting that that their resistances to water effects were poor.

6. No significant findings were discovered during the interlayer shear tests. Overall, the SPA and SPPA performed better during this test.

7. The Cantabro tests (particle loss) test results showed that the new mixes reduced the Cantabro loss values by 14%. Moreover, the Japanese mixes showed smaller particle loss then the Swiss mixes.

Overall the research presented by Poulikakos et al suggests that the positive effects of the new mix design packing theory were clearly shown. The highest overall advantage was the reduction in Cantabro loss.

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1.48.2 Benefits of Permeable Asphalt Mixtures
Poulikakos et al state that porous asphalt has the benefits of noise reduction and improved safety under wet conditions.

1.48.3 Materials and Design
Poulikakos et al state porous asphalt is used on top layers and has an air void content of 20% or greater.

1.48.4 Construction Practices
Poulikakos et al did not discuss construction practices.

1.48.5 Maintenance Practices
Poulikakos et al did not discuss maintenance practices.

1.48.6 Rehabilitation Practices
Poulikakos et al did not discuss rehabilitation practices.

1.48.7 Performance
Poulikakos et al did not discuss performance of porous asphalt.

1.48.8 Structural Design
Poulikakos et al did not discuss structural design of porous asphalt.

1.48.9 Limitations
Open structure of porous asphalt exposes surface to the effect of air and water, thus leading to rapid aging of the binder which can lead to particle loss and adhesive failure.

As pavement life proceeds, porous asphalt has voids that become clogged which leads to loss of permeability.

1.49 Tan, S.A., T.F. Fwa and C.T. Han. "Clogging Evaluation of Permeable Base." ***Journal of Transportation Engineering*****. American Society of Civil Engineers. Reston, VA. Volume 129. Issue 3. May 2003. pp. 309-315.**

1.49.1 General
This paper addresses clogging of a permeable asphalt base layer from an experimental and theoretical approach.

Tan et al explain that clogging occurs when particles (sand, sediment, construction material, etc.) collect in porous asphalt layers. They further elaborate that this phenomena is common on public roads and highways due to the increased frequency and type of vehicles.

The experimental portion of the Tan et al research involved laboratory testing of four common base mixes: Modified Base 1, Modified Base 2, US Army Corps of Engineers

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Open Graded Base, and FHWA permeable base [No specific gradation or mix design information was presented in this paper]. Each mix was subjected to a permeability test using the NUS falling head permeameter, a clogging evaluation where 63.5g of clogging material (sand) was placed onto the mix and then water was gently showered so that the clogging material could flow, and a clogging material collection procedure where the clogging material was collected after it traveled through the specimen and quantified. The amount of clogging material collected was then subtracted from the initial amount to yield the amount that actually remained in the specimen. This experimental process was repeated until low to no permeability existed in the specimen.

Tan et al validated their experimental procedure by utilizing Wu and Huang's theoretical model, which was derived from the Kozeny-Carmen equation. Results showed that the theoretical results and experimental results showed close agreement up to the addition of the fifth increment of sand.

Tan et al experiments showed that particle retention increases with gradation openness. Also they found that narrow gradations will yield larger voids and thus allow the mix to retain more particles before losing permeability.

1.49.2 Benefits of Permeable Asphalt Mixtures
Tan et al did not discuss any benefits of permeable asphalt mixtures.

1.49.3 Materials and Design
Tan et al did not discuss materials and design.

1.49.4 Construction Practices
Tan et al did not discuss construction practices.

1.49.5 Maintenance Practices
Tan et al did not discuss maintenance practices.

1.49.6 Rehabilitation Practices
Tan et al did not discuss rehabilitation practices.

1.49.7 Performance
Tan et al did not discuss performance of permeable asphalt mixes.

1.498 Structural Design
Tan et al did not discuss structural design.

1.49.9 Limitations
A clogged permeable layer will have reduced drainage and water storage capacity.

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1.50 Watson, D. E., K. A. Moore, K. Williams and L. A. Cooley, Jr. "Refinement of New Generation Open-Graded Friction Course Mix Design." Transportation Research Record No: 1832. Transportation Research Board. National Research Council. Washington, D.C. 2003.

1.50.1 General

Starting with a discussion on the mix design developed earlier by NCAT researchers, Watson et al points out certain concerns and the need for evaluation of some test methods and specifications for the design of PFCs.

Watson et al then describes the results of a laboratory study carried out with the following objectives: 1.) Determine if the N_{design} of 50 for Open Graded Friction Course is appropriate or not; 2.) How can the method for determination of air voids in OGFC mixes be improved; 3.) What are the implications of using samples compacted with the Superpave gyratory compactor (SGC) (as opposed to Marshall hammer) for Cantabro test; and 4.) Is there any advantage of changing the sieve size in the basket used for evaluation of draindown in OGFC mixes during mix design?

Watson et al indicate that the study consisted of preparation of samples of OGFC with three different types of aggregates and three different types of asphalt binder, and one type of fiber. Samples were compacted at different gyrations with the SGC, as well as with the Marshall hammer, with two different compaction levels. The air voids were determined using dimensional analysis as well as the CoreLok method. Voids in coarse aggregate for the mix were compared to voids in coarse aggregate for dry-rodded condition. Breakdown of aggregates were also evaluated. Conditioned and unconditioned samples were subjected to the Cantabro abrasion loss testing. Finally, baskets with two different opening sizes were used for determination of draindown of the different mixes.

Based on the results, Watson et al concluded that the originally proposed N_{design} of 50 seemed to be appropriate, that future work needed to be done for better prediction of air voids in OGFC mixes, breakdown of aggregates was significantly higher in samples compacted with the Marshall hammer, the Cantabro abrasion loss criteria should be changed to accommodate the reduced stone loss for the samples compacted with the SGC, and that a reduced draindown basket opening size could be used for the draindown testing baskets.

1.50.2 Benefits of Permeable Asphalt Mixtures

In their opening paragraph, Watson et al indicate that OGFC pavements have high air void contents that allow water to be removed from the pavement through the asphalt layer. They mention that this removal of water can help to minimize splash and spray, increase visibility during rain, improve friction resistance and reduce noise levels. Watson et al also mention that OGFCs have primarily been used in the United States (U.S.) for friction resistance.

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1.50.3 Materials and Design

Watson et al indicate that three different types of aggregates and three different types of asphalt binder were used in this study. Samples were compacted with the SGC as well as the Marshall hammer. Table 101 shows the matrix of materials used. Note that the aggregates were blended to meet the medium gradation requirement. The master gradation shown in Table 101 was developed from recent experiences in Georgia – the fine gradation represents mixes originally used (for 9.5 mm NMAS OGFC mixes) in Georgia, whereas the medium and the coarse gradations represent typical porous asphalt mixes used in Europe.

Table 101: Materials and Mixes Used in This Study

Material/property	Type				
Aggregate	Granite, crushed gravel, and traprock				
Aggregate Properties	The traprock is the heaviest aggregate used, followed by granite and crushed gravel respectively. In terms of LA Abrasion loss, the traprock is the toughest, followed by crushed gravel and granite.				
Aggregate gradation, percent passing	Sieve Size	Master Gradation	Fine	Medium	Coarse
	19 mm	100	100	100	100
	12.5 mm	80-100	100	90	80
	9.5 mm	35-60	90	47	35
	4.75 mm	10-25	25	17	10
	2.36 mm	5-10	10	7	5
	0.075 mm	2-4	3	3	3
Asphalt Binder	PG 67-22, PG 76-22 (SBS modified) and PG 76-34 (rubber modified; chemically modified within the refining process).				
Rubber modification process	Chemically modified crumb rubber asphalt (CMCRA) was prepared by using an asphalt of PG 55-34 using ten percent chemically modified crumb rubber by weight of asphalt (minus 180µm – No. 80 – mesh rubber), followed by the addition of 1.7% Styrene Butadiene Styrene (SBS) in continuous stirring mode at an elevated temperature of 170-175ºC. After the complete dispersion of the polymeric material a double action activator/linking agent (0.6%) was added to the mixture of asphalt, crumb rubber and polymer. The CMCRA obtained after modification meets requirements for PG 76-34 with a solubility of 98 percent.				
Stabilizer	Fiber, 0.4% of the total mix weight.				
Mix	With fiber - PG 67-22, PG 76-22, PG 76-34 Without fiber - PG 67-22, PG 76-22				

The test methods that were conducted are shown in Table 102.

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Table 102: Tests for Different Properties

Property	Standard Tests and Procedures Used in this Study
Compaction	Marshall – 25 and 50 blows; SGC – 30, 45 and 60 gyrations
Aggregate Breakdown	Extraction in ignition furnace (AASHTO TP 53-97, without calibration) and washed sieve analysis (AASHTO T 11-91).
Stone-on-Stone Contact	Voids in coarse aggregate in dry rodded condition (VCA_{DRC}) (AASHTO T 19) and voids in coarse aggregate in mix (VCA_{mix}).
Air Voids	Dimensional volume: CoreLok – using double bags.
Draindown	NCAT draindown test; draindown was determined for three binder grades – PG 67-22, PG 76-22 and PG 76-34 and three aggregate types.
Abrasion/ Stone Loss	Cantabro tests (European standard: prEN 12697-17); In this study, samples were compacted using both Marshall hammer and the Superpave gyratory compactor (SGC). The SGC samples were compacted to 50 gyrations and the Marshall samples were compacted to 50 blows per face. Testing was completed using the medium gradation and a PG 67-22 binder and then repeated with a PG 76-22 polymer modified binder. Six samples (rather than ten) were made using each aggregate, the medium gradation, and two asphalt contents. Three of these samples were tested after cooling and are considered unconditioned samples. Three samples were conditioned by placing them in a forced draft oven at 64°C for seven days. The conditioned samples were allowed to cool at room temperature (25°C) a minimum of four hours before testing.
Draindown	With draindown baskets, AASHTO T305-97, with two modifications: 1. Two different basket sieve sizes, and 2. Weighing the basket after the mix was emptied out of the basket.

Regarding compaction characteristics, Watson et al indicate that the PG 76-22 SBS modified asphalt and fiber stabilizer were used to prepare three replicates each for the five levels of compaction. A 6.0 percent binder content was used for all mixes because it was estimated to be near the optimum level needed for these mixes based on previous experience. The compaction slope (gyration versus sample height) for these mixes indicated that the density of these mixtures continued to increase even after 60 gyrations. Watson et al mention that when the change in height per gyration was plotted the resistance to compaction of all three aggregates was seen to be virtually identical. The results showed that most of the density is obtained within the first 20 gyrations. The authors concluded that stone-on-stone contact was beginning to occur at this point and that the increase in density beyond that point was due to aggregate breakdown from excessive compactive effort.

Watson et al made a comparison between the number of gyrations with the SGC and the Marshall hammer. It was found that there was not a close correlation with any of the gyratory compaction levels to the 25 blow Marshall method. However, the 50 blow Marshall compaction effort matched very closely to the gyratory compaction effort at 50 gyrations. Watson et al showed that the SGC results for the traprock and granite mixes

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are equivalent to that of the 50 blow Marshall within a range of 45 to 53 gyrations. The gravel mix had a higher density with the 50 blow Marshall hammer than with the gyratory compactor.

To evaluate aggregate breakdown, Watson et al indicate that samples for each of the five compaction levels were tested for gradation and compared to the actual gradation blend. The results showed that aggregate breakdown (difference in gradation before and after compaction) ranged from zero on the 0.075mm (No. 200) sieve to about ten percent on the 4.75mm (No. 4) sieve. The authors mention that a comparison of breakdown for the granite mixes showed that the SGC breakdown was not directly dependent on the number of gyrations and was not as great as the breakdown caused by the Marshall method. They mention that even as little as 25 blows with the Marshall hammer resulted in greater breakdown than 60 gyrations with the SGC. The authors concluded that the dynamic impact of the Marshall hammer provided more breakdowns of aggregate particles than the kneading action of the SGC. It was also seen that the breakdown from 60 gyrations of the SGC was practically the same as the breakdown from 30 gyrations. Based on this data, Watson et al concluded that the 50 gyration compactive effort previously recommended by NCAT is an acceptable value for design compactive effort.

Watson et al also looked at stone-on-stone contact of different mixes to evaluate the N_{design} criterion. Results from tests using the traprock aggregate indicated that VCA_{MIX} was slightly higher than VCA_{DRC} at the 30 gyration level. This indicates that 30 gyrations were insufficient to compact the specimens to the point of stone-on-stone contact. At both 45 and 60 gyrations, all samples met stone-on-stone criteria. Based on this observation, the authors conclude that a design compactive effort of 50 gyrations is acceptable.

Watson el at indicate that the determination of mixture density and air voids is an inherent problem in the traditional mix design system for OGFC. In this study, air voids were determined based on three methods of determining bulk specific gravity: dimensional analysis, use of the vacuum sealing method (CoreLok), and the use of the Corelok to determine the effective air void content. Bulk specific gravity of the OGFC samples was also obtained using the CoreLok device, with one modification. The use of a single bag frequently resulted in punctures (when testing SGC samples) that would allow air back into the vacuum-sealed bag. Therefore, the double bag method was used on all gyratory samples to prevent bag punctures. A double bag correction factor was used. The standard single bag method was performed on all Marshall samples and bag punctures were not a problem.

Watson et al conducted Cantabro Abrasion loss tests to determine whether samples compacted with the SGC could be utilized in this test. From a comparison of a sample before and after testing, the authors comment that the test is a severe test of mixture durability. Test results on Marshall compacted samples for each of the aggregate sources indicated that none of the samples would meet the maximum requirements. The failing results applied to both the unconditioned and conditioned samples in which 6.0 percent asphalt binder had been added. Only granite mixes met maximum loss requirements only when a polymer-modified asphalt was used.

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Watson et al mention that with Superpave gyratory compacted samples, all mixtures except traprock and gravel with the PG 64-22 binder met the requirements. All the gyratory samples with polymer-modified binder met the maximum stone loss requirements of 20 percent for unconditioned specimens and 30 percent for conditioned samples. They indicate that the results show that 6.0 percent asphalt binder content is insufficient for the traprock and gravel mixtures in order to provide the durability needed for long-term performance. When the asphalt content was increased to 6.5 percent and a polymer-modified binder was used, all mix types met the Cantabro requirements for 50 blow Marshall specimens.

Watson et al indicates that the results emphasize the important contribution of polymer-modified asphalt in providing resistance to stone loss and improving mixture durability. As an example, they mention that there was as much as 55 percent difference in the percent of stone loss for the traprock aggregate when comparing results of unmodified asphalt to that of the polymer modified binder.

Watson et al mention that stone loss by the Cantabro procedure is almost always lower for the SGC samples compacted to 50 gyrations than the Marshall specimens compacted to 50 blows. They comment that for this reason, the criteria developed in Europe for use with Marshall design procedures may not be applicable to the Superpave compaction method used in the U.S., and that the criteria may need to be changed. They mention that the data showed that when stone loss from Marshall samples is compared to stone loss from SGC samples, a corresponding limit for unconditioned samples should be 15 percent maximum loss and that the stone loss for conditioned samples should be near 20 percent maximum.

Watson et al conducted draindown tests with different mixes, using a variation of the standard basket drainage test procedure. They used baskets with 4.75 mm as well as 2.36 mm openings, and they weighed the baskets at the end of the draindown test. The smaller sieve was used because the authors felt that there may be some intermediate aggregate particles that pass through the 4.75mm (No. 4) mesh that would adversely affect test results for binder draindown, by showing an artificially high value for binder draindown. The second modification was performed to determine the asphalt material that remained on the basket and in effect could be considered as part of the binder draindown similar to the Schellenberger drainage test. To evaluate the draindown results an analysis of variance (ANOVA), was conducted. The factors considered were 2 aggregate types, 3 gradations, 3 binder types, 2 fiber conditions, and 4 asphalt contents. Based on previous experience, only traprock and granite were considered as aggregate type factors in this analysis. A regression analysis was also performed which indicated a very strong correlation between the two basket types and two methods used to calculate draindown. An analysis of test results using the standard AASHTO procedure was compared to test results using the modified method where the amount of binder retained on the 4.75mm (No. 4) wire mesh was also considered as draindown. The regression analysis had a coefficient of determination (R^2) of 98.5 percent for this comparison. The authors comment that this indicates that considering the amount of binder retained on the 4.75mm (No. 4) basket may not be a significantly better method of determining draindown than

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the current method. An analysis of the normal method of calculating draindown was conducted to compare the two basket mesh sizes. The regression analysis resulted in a R^2 of 89 percent. This strong correlation indicates the methods are very comparable to one another.

Watson et al mention that since both methods have a good correlation, the only reason to choose one method over another would be the repeatability of test results. The repeatability standard deviation of test results when the 2.36 (No. 8) mesh was used is only 60 percent as much as the standard deviation when the 4.75mm (No. 4) sieve was used. The authors mention that the use of 2.36 mm mesh size provided more repeatable measurements. They explain that probably aggregate particles may have fallen through the 4.75 mm mesh size on some tests but not others.

Watson et al also indicate that further evaluation of the ANOVA statistics showed that the addition of fiber stabilizer was the most significant variable contributing to a reduction in draindown. There was as much as 3 percent draindown in specimens prepared without fiber while the addition of 0.4 percent fiber by weight of the total mix resulted in minimal draindown.

1.50.4 Construction Practices
No information is provided on construction practices of friction course.

1.50.5 Maintenance Practices
No information is provided on maintenance practices of friction course.

1.50.6 Rehabilitation Practices
No information is provided on rehabilitation practices of friction course.

1.50.7 Performance
No information is provided on performance of friction course.

1.50.8 Structural Design
No information is provided on structural design of friction course.

1.50.9 Limitations
No information is provided on limitations of use.

1.51 Wimsatt, A. J. and T. Scullion. "Selecting Rehabilitation Strategies for Flexible Pavements in Texas." TRB 2003 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2003.

1.51.1 General
This paper does not describe any aspect of friction course design or construction, but gives details on observations made during rehabilitation of pavements containing Open Graded Friction Course (OGFC).

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Wimsatt and Scullion describe the rehabilitation selection strategy that was developed as part of a research project in Texas. They also describe results of case studies of three pavement sections in Fort Worth District in Texas, two which contained OGFC. Only these two are discussed in this review.

In the first part of the paper, Wimsatt and Scullion describe a number of procedures for evaluation of pavements and selecting rehabilitation methods. They mention that the TTI Research Report 1712-4, titled "Selecting Rehabilitation Options for Flexible Pavements: Guidelines for Field Investigations" presents an updated summary of the techniques and interpretation guidelines that have been developed by the Texas Transportation Institute over the past two decades in order to conduct an effective investigation. These methods include both nondestructive (Falling weight deflectometer (FWD) and Ground Penetrating radar (GPR)) as well as destructive (Dynamic Cone Penetrometer, DCP) methods. Explanations and equations are provided to describe the use of these techniques.

Wimsatt and Scullion then describe the results of three case studies conducted in the Fort Worth District of Texas. Before starting on the discussion about the three case studies, it is necessary to note that TxDOT's Fort Worth District regularly places seal coats on top of old open graded friction courses. As the authors indicate, the seal coat is used for rehabilitating pavements with substantial rutting distress.

For the first case study reported, FWD testing did not reveal any abnormalities in the existing pavements, which had seal coats and patches over OGFCs. However, results of GPR studies indicated entrapment of water in the old OGFC layers. Observation of the cores by TxDOT personnel indicated that the overlying seal coat AC binder did not completely penetrate the open graded friction course, and voids were still present in this layer that could trap water. This was causing the open graded friction course to become unstable due to freeze/thaw cycles during the winter and due to the asphalt stripping from the aggregate in this layer. So, as a result, the upper 51 mm (two inches) needed to be removed.

In the second case study, GPR data indicated that the open graded friction courses were holding water where maintenance personnel had placed asphalt overlays. The cores obtained indicated that the open graded friction course layer placed in 1988 was disintegrating, which resulted in the surface distress. Therefore, TxDOT personnel concluded that the open graded friction course layers had to be removed, along with any asphalt overlays on top of those layers.

1.51.2 Benefits of Permeable Asphalt Mixtures
No information is provided on benefits of permeable asphalt mixtures.

1.51.3 Materials and Design
No information is provided on materials and design of permeable asphalt mixtures.

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1.51.4 Construction Practices

The first case study was conducted on SH 337, Palo Pinto County. Wimsatt and Scullion mention that this 17.7 km long roadway section consisted of (on average) 203 mm (eight inches) of granular base and two seal coats placed in 1970, followed by a seal coat placed in 1980, an open graded friction course placed in 1989, and another seal coat placed in 2000. The seal coats and open graded friction course together were approximately 51 mm (two inches) thick. However, before the last seal coat was placed, TxDOT maintenance personnel had applied many seal coat surface patches and asphalt patches on this roadway due to the disintegration of the open graded friction course.

Wimsatt and Scullion indicate that in the second case study project (16.9 km long), US 377 Southbound Lanes, Hood County, the pavement was constructed in 1979 with 152 mm (six inches) of lime stabilized subgrade where clay subgrade was present, 254 mm (ten inches) of granular base and a 54 mm (two inch) asphalt surface. Asphalt level up courses and two open graded friction courses were then placed in 1988 and 1992, respectively. The total surfacing was approximately 178 mm (seven inches) thick.

1.51.5 Maintenance Practices

As mentioned in the Construction Practices section, Wimsatt and Scullion indicate that it is the standard practice for TXDOT personnel to use seal coats over distressed OGFC section as part of their maintenance operations.

1.51.6 Rehabilitation Practices

For both sections mentioned above, the authors mention that part of the rehabilitation option was to remove the overlay and the OGFC sections. For the SH 337 section, it was decided to repair the failed base areas and then put 75 mm of HMA. For the US 377 section, the plan was to apply 127 mm of HMA after the removal of the overlay and OGFC layers.

1.51.7 Performance

The authors indicate that both sections, SH 337 and US 377, which had asphalt overlays on top of OGFC layers, were showing major distress. SH 337 was showing ruts up to 51 mm deep, and the section on US 377 was exhibiting substantial alligator cracking and potholes

1.51.8 Structural Design

No information is provided on structural design of friction course.

1.51.9 Limitations

No information is provided on limitations of use.

1.52 Cooper S. B., C. Abadie, and L. N. Mohammad. "Evaluation of Open-Graded Friction Course Mixture." Louisiana Transportation Research Center Technical Assistance Report Number 04-1TA. October 2004.

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1.52.1 General

This report provides a history on the use of OGFC in Louisiana and documents the design of an OGFC project from 2003. Louisiana first developed and used OGFC in the late 1960's and early 1970's. The primary reason for using OGFC in Louisiana was to provide an improved skid resistant wearing surface. In 1973, the Louisiana Department of Transportation and Development issued an Engineering Directive to use OGFC on all roads with an average daily traffic (ADT) greater than 4,000. In 1980, the ADT limit was changed to all roads with ADT greater than 3,000.

During the 1980's, some problems were encountered with Louisiana's OGFC pavements. At this time, many of the OGFC layers were 10 to 12 years old and had begun to fail. Failure was signaled by severe raveling in the wheel paths. Cooper et al stated that the severe raveling problems were related to moisture and temperature. The temperature problems were related to both mix temperature and weather. Moisture problems were related to the drying of aggregates during the production process and predominately associated with a particular aggregate type. To address these issues, a maximum moisture content for the aggregate, limiting OGFC construction to between May and September, and increasing ambient temperature requirements were implemented.

Due to a number of factors, a second moratorium was placed on the use of OGFC in 1984. Just prior to the moratorium, experimental sections of OGFC were placed in which polymer modified asphalt binder were used. The experimental sections included a 4 mile section of OGFC containing a latex modified asphalt binder and 4 miles of OGFC containing an elastomeric polymer modified binder. Cooper et al state the latex modified binder was similar to a current PG 70-22 and the binder modified with the elastomer was similar to a PG 76-22. In addition to theses experimental sections, a control section was placed using an AC-30 asphalt binder. Within one year, this control section began raveling and within two years the wheel paths had completely raveled. Cooper et al stated that the experimental sections (using modified binders) were still performing in 1999 (15 years) when the entire project was rehabilitated.

Based upon the success of these OGFC sections containing modified binders, Louisiana constructed a new OGFC test section in 2003. The remaining portion of the report deals with materials and mix design.

1.52.2 Benefits of Permeable Asphalt Mixtures

The authors indicate that the high air void contents associated with OGFC promotes draining of rainwater from the pavement surface. This characteristic minimizes hydroplaning, reduces splash and spray and improves wet weather skid resistance.

1.52.3 Materials and Mix Design

The authors provide discussion on a number of materials used for the 2003 project including: asphalt binder, aggregates, fibers, anti-strip additives and tack coat (for construction). The asphalt binder used for the project was polymer modified (elastomer) that was graded as a PG 76-22. Sandstone and limestone aggregates were used to

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develop two aggregate gradations. No specifics were provided on the aggregates except that they met the applicable Louisiana specifications for OGFC.

A mineral fiber in a pelletized form was added to minimize draindown potential. A fiber content of 0.1 percent by total mix mass was selected based upon laboratory draindown testing. The contractor did include a liquid anti-strip in the asphalt binder to minimize moisture susceptibility. A SS-1 emulsion was selected for the project and was applied at a rate of 0.07 g/yd^2.

The mix design was based upon the procedure recommended by the National Center for Asphalt Technology. Tables 103 and 104 present two aggregate blends that were evaluated as part of this mix design. Based upon the data shown in Tables 103 and 104as well as the results of Hamburg wheel tracking tests, Design 2 was selected and placed.

1.52.4 Construction Practices
No specific construction practices were given.

1.52.5 Maintenance Practices
No specific maintenance practices were given.

1.52.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.52.7 Performance
No specific performance measures were given.

1.52.8 Structural Design
No specifics on inclusion within structural design were given.

1.52.9 Limitations
No specific limitations were given.

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Table 103: Composition of Mix Design Blends

Material	Percentage	
	Design 1	Design 2
#78 Sandstone, FR I	84.0	67.2
#11 Sandstone, FR I	9.3	7.4
#89 Limestone, FR III		18.7
PG 76-22m	6.6	6.6
Fibers	0.1	0.1
Ad-Here LA 2	0.6 by Wt. of AC	0.6 by Wt. Of AC

Table 104: Composite Blends and Mixture Properties

Sieve Size	Percent Passing				
	Design 1	Design 2	QA Data	LTRC 2nd Truck	Required Gradation
¾" (19mm)	100	100	100	100	100
½" (12.5mm)	90	92	93	91	85-100
3/8" (9.5mm)	58	64	68	66	55-75
No. 4 (4.75mm)	14	16	21	26	10-25
No. 8 (2.36mm)	9	8	11	18	5-10
No. 16 (1.18mm)	7	6	9	16	
No. 30 (.600mm)	6	5	8	15	
No. 50 (.300mm)	5	4	7	14	
No. 100 (.150mm)	3.8	3.4	6	10	
No. 200 (.075mm)	2.8	2.3	3.9	6.1	2-4
G_{mb}	1.916	2.173			
G_{mm}	2.374	2.368	2.381	2.389	
VCA	33.0	23.0			
%Air Voids, AASHTO T166	19.3	8.2			18
G_{sb}	2.558	2.604			
G_{se}	2.619	2.612			
P_{ba}	0.9	0.8			
P_{be}	5.9	6			
Permeability, ft/day	276	453			
Permeability, ft/day LTRC Results	235	278			
Draindown	0.08	0.08			0.3
Design AC	6.6	6.6	7.0	6.8	

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1.53 Flintsch, G.W. "Assessment of the Performance of Several Roadway Mixes Under Rain, Snow, and Winter Maintenance Activities." Final Contract Report. Virginia Transportation Research Council. VTRC-04-CR18. Charlottesville, Virginia. 2004.

1.53.1 General

This report documents a research effort to assess the relative functional performance, including skid resistance and splash/spray, of various HMA surfaces during controlled wet and wintry weather events. The research was conducted in the field at the site of the Virginia Smart Road. The Virginia Smart Road research facility provided a unique opportunity to evaluate full scale pavements during wet and wintry conditions in a controlled manner. Either artificial snow or rain could be applied to the various pavement surfaces in order to research splash/spray during wet weather or various deicing/anti-icing techniques during snow events.

Six different wearing surface types were investigated: five hot mix asphalt and one Portland cement concrete. Three of the hot mix asphalt wearing surfaces were designed as dense-graded using the Superpave mix design system while the remaining two hot mix asphalt wearing surfaces were a SMA and OGFC. The final wearing surface was a tined Portland cement concrete surface. [No specifics were provided on the method of tining.]

Two separate experiments were conducted by Flintsch. First, artificial snow was produced and spread over the pavement surface. Flintsch described an extensive process that creates the snow and spreads the snow in a reasonably uniform manner. The second experiment involved producing artificial rain to provide wet driving conditions.

1.53.2 Benefits of Permeable Asphalt Mixtures

Flintsch concluded that OGFC mixes reduced splash and spray during wet weather events.

1.53.3 Materials and Mix Design

No specifics on materials and mix design were given.

1.53.4 Construction Practices

No specifics on construction practices were given.

1.53.5 Maintenance Practices

The effect of various deicing and anti-icing procedures was evaluated during the wintry weather experiment. Table 105 presents a summary of the various deicing/anti-icing techniques utilized by Flintsch. As shown in Table 105, sodium chloride was utilized in each of the tests. Depending upon the test, sodium chloride was used in a granular, pre-wetted or liquid form. Liquid calcium chloride was also used to pre-wet the granular sodium chloride for some of the tests.

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Table 105: Summary of Deicing/Anti-icing Techniques

Test	Test Date	Applied Chemical	Application Rate	Pre-wetting Rate
I	2-12-2002	Dry solid sodium chloride	200 lb/lane-mile	None
II	3-04-2002	Solid sodium chloride prewetted with liquid calcium chloride	200 lb/lane-mile	10 gal/ton
III	3-06-2002	Solid sodium chloride prewetted with liquid calcium chloride	200 lb/lane-mile	10 gal/ton
IV	3-23-2002	Solid sodium chloride prewetted with liquid calcium chloride	300 lb/lane-mile	10 gal/ton
V	1-27-2003	Solid sodium chloride prewetted with liquid calcium chloride	300 lb/lane-mile	10 gal/ton
VI	2-06-2003	Liquid sodium chloride solution	40 lb/lane-mile	None

The research team utilized a general working procedure to produce and spread the snow, apply the deicing/anti-icing chemicals and measuring performance. The procedure generally included identifying appropriate weather conditions (temperature) for the production and placement of the artificial snow. The snow was then generated and applied to the pavement. A spreader then applied the deicing chemicals to the roadway, except in the instance where the anti-icing chemical was placed prior to the production of the snow. Traffic was then initiated. When the snow accumulation had reached approximately 50mm (2 in.), snow generation was ceased and the pavement plowed twice. Friction was then measured using an ASTM D274 skid trailer using a smooth tire. Results from the skid trailer were used to calculate a Skid Number (SN).

In all cases, the SNs were very low (Table 106). An analysis of variance conducted on the SN values on each of the sections indicated no significant differences between treatment type (deicer or anti-icing materials) or wearing surface type. Flintsch indicated that the lack of significance could have been caused by the make-up of the artificial snow. Naturally occurring snow has an equivalent water coefficient of 1:10 (1 in. of equivalent water for every 10 in. of snow) while the water equivalent for the artificial snow was approximately 1:4. The extra amount of water may have diluted the deicing/anti-icing chemicals and, therefore, reduced their effectiveness. Flintsch also surmised that the amount of traffic (one truck and one car) was not enough to facilitate the spread of the chemicals and the formation of a liquid layer to prevent the bonding between the pavement and snow.

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Table 106: Summary of Friction Numbers Measured on the Test Lane and Control Lane

Test	Experiment	Section (Friction Number)					
		H	I	J	K	L	Conc.
I	Test	15.2	16.5	14.7	15.2	18.7	17.7
	Control	14.9	15.8	15.8	16.5	16.5	13.8
II	Test	18.6	17.6	18.1	21.6	19.0	21.2
	Control	18.5	19.7	16.7	18.4	18.9	18.5
III	Test	18.5	14.0	13.5	16.2	13.4	15.6
	Control	17.2	22.3	16.6	11.0	12.2	14.1
IV	Test	12.6	11.1	13.4	16.8	15.7	16.3
	Control	14.9	25.5	15.2	14.8	14.0	18.2
V	Test	19.7	19.7	87.9[(1)]	25.0	21.7	22.6
	Control	39.9[(1)]	19.9	91.0[(1)]	20.5	22.0	21.0
VI	Test	11.9	15.4	14.7	18.5	13.7	15.1
	Control	15.0	15.4	16.4	15.7	16.0	17.5

[(1)] Sections were not fully covered with snow because of problems with some of the snow towers.

1.53.6 Rehabilitation Practices

No specifics on rehabilitations practices were given.

1.53.7 Performance

A second experiment conducted by Flintsch was conducted to compare the wearing surfaces during a rain event. The experiment involved producing artificial rain and visually comparing each wearing surface through the use of video. Flintsch states that the OGFC enhanced splash and spray performance when compared to the other hot mix asphalt surfaces.

1.53.8 Structural Design

No specifics or inclusions with structural design were given.

1.53.9 Limitations

No specific limitations were given.

1.54 Fortes, R.M. and J.V. Merighi. "Open-graded HMAC Considering the Stone-on-Stone Contact." Proceedings of the International Conference on Design and Construction of Long Lasting Asphalt Pavements. Auburn, Alabama. June 2004.

1.54.1 General

This paper presents the results of a research study to evaluate the use of stone matrix asphalt (SMA) and open-graded friction course (OGFC) for an urban expressway in Sao Paulo, Brazil. The authors describe a pavement that is seven or eight lanes in each direction that carries almost 400,000 vehicles per day in each direction. Approximately 22 percent of the vehicles are heavy trucks and the trucks are limited to the two outside lanes at speeds approximately one-third of small vehicular traffic. The primary problems

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associated with the expressway are rutting, reflective cracking, aggregate polishing and low skid-resistance. Fortes and Meighi also describe a problem with large amounts of splash and spray originating from the heavy trucks during a rain event.

1.54.2 Benefits of Permeable Asphalt Mixtures

The authors did not specifically conduct testing to evaluate the benefits of permeable friction courses, however, numerous literature were referenced that indicated that permeable friction courses could reduce splash and spray, improve frictional properties, reduce the potential for hydroplaning and reduce tire-pavement noise.

1.54.3 Materials and Design

The paper described a mix design procedure that utilizes 50 blows per face of the Marshall compaction hammer for the standard compactive effort. A single granite aggregate source was used for all testing. The authors reported a Los Angeles Abrasion loss of 25.1 percent for the coarse aggregates and a sand equivalency of 66 percent for the fine aggregates. Two asphalt binders were included within the study: an AC-20 and a polymer modified (SBS) asphalt binder. Six different aggregate stockpiles were used to fabricate ten different open-graded gradations which are illustrated in Figure 23.

In order to evaluate the existence of stone-on-stone contact within the OGFC mixtures, the authors used methods developed during NCHRP 9-8 for SMA mixes. The voids in coarse aggregate (VCA) of the aggregate fraction was determined by adding a "low asphalt content" and compacting with 50 blows per face of the Marshall hammer. [Based upon the reference provided, it appears that the authors utilized 2 percent asphalt binder.] Of the ten gradations fabricated within this study, the authors state that only two meet the requirements of having stone-on-stone contact (shown as bold lines in Figure 23). These two gradations were further investigated.

[Based upon the experiences of the NCHRP 9-41 researchers, it is very difficult to design an open-graded friction course that does not have stone-on-stone contact. Evaluation of the gradations in Figure 23 suggests that there are at least three gradations that are coarser and more gap-graded than the two highlighted gradations that the authors indicate did not have stone-on-stone contact. The researchers tried to reproduce the values for VCA of the coarse aggregate fraction provided in the paper as well as other volumetric properties without success. It appears that the authors of this paper used an erroneous equation for calculating the VCA of the coarse aggregate fraction. Equation 2 from the paper is provided verbatim along with descriptions of each equation component below.

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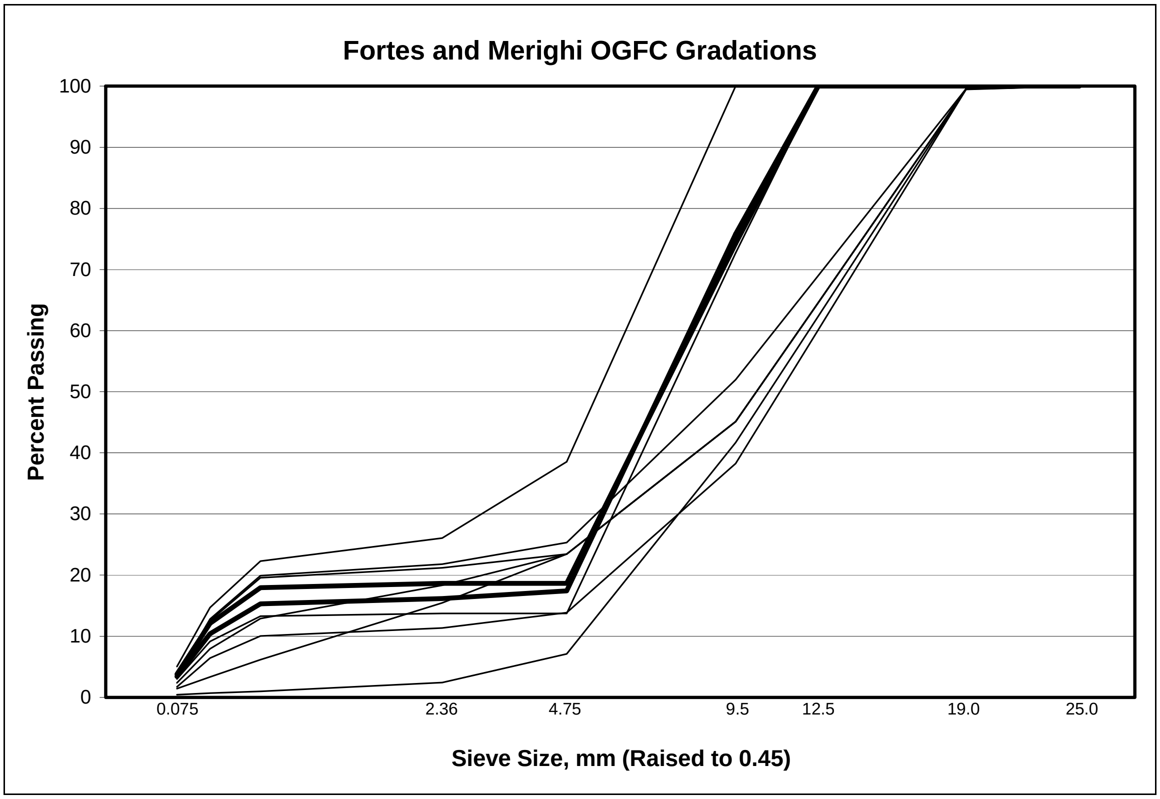

Figure 23: Gradations Used by Fortes and Merighi

$$VCA_{LA} = \frac{\lfloor(\gamma_{agr} \times \gamma_a) - \gamma_{agD}\rfloor}{\gamma_{agr} \times \gamma_a} \times 100$$

where,

VCA_{LA} =	Coarse aggregate voids compacted with low asphalt binder, %
γ_{agr} =	bulk specific gravity of the compacted specimen, kN/m^3
γ_a =	unit weight of water, kN/m^3
γ_{agD} =	unit weight of the coarse aggregate fraction, kN/m^3.

In contrast to the above equation, the following is a correct equation when determining the VCA of the coarse aggregate fraction when using a "low asphalt binder content":

$$VCA = 100 - \left[100\left(\frac{G_{mb}}{G_{ca}} \times (1 - P_b)\right)\right]$$

where,

Gmb =	bulk specific gravity of the compacted mix,
Gca =	bulk specific gravity of the coarse aggregate,
Pb =	percent asphalt binder in the mixture (by total mixture mass).

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Therefore, it appears that the authors were in error when calculating the VCA of the coarse aggregate fraction. Because of these errors, the NCHRP 9-41 researchers tried to recalculate some volumetric properties from the data available within the paper; however, problems were again experienced. Data for the unit weight of the compacted mixtures along with calculated air voids were provided. When the theoretical maximum density was back calculated from these data, the "Rice" was higher than the reported aggregate specific gravities. Therefore, it is uncertain whether other properties are correct within the paper. There remaining part of this review only provides general conclusions provided by the authors.]

The two selected gradations were combined with an AC-20 asphalt binder at three asphalt binder contents (not identical) and a polymer modified binder at a single binder content. These eight mixes were then compared to two typical dense-graded mixes. Volumetric properties of all the mixes were evaluated. Specimens containing the polymer-modified binders had higher air void contents than the OGFC mixes containing the AC-20 asphalt binder. This is likely the result of using identical compaction temperatures for each mix with the Marshall hammer. Tests conducted on the ten different mixtures included Marshall stability (60˚C), indirect tensile strength (25˚C) and laboratory permeability testing. Results of all three tests indicated that the addition of the polymer modified binder improved the performance of the OGFC mixtures. Not surprisingly, however, the Marshall stabilities and indirect tensile strengths of the dense-graded mixes were higher than the OGFC mixes.

The authors also employed an unconfined static creep test at test temperatures of 25, 40 and 50 C to evaluate the potential for permanent deformation for each of the ten mixtures. This testing was conducted because of the rutting problems described previously. Specimens for this testing were 100 mm (4 in) diameter samples. [No specimen heights were provided; however a figure within the paper suggests that the height of the sample was approximately 50 mm (2 in.) in height]. For the static creep test, the authors capped each end of the sample with a resinous material. A load of 0.55 MPa (approximately 80 psi) was placed onto the sample for 1,000 seconds. The load was then removed and the deformation measured for another 1,000 seconds (rebound). This loading configuration was conducted for a total of four steps. Results from this testing indicated that the OGFC mixtures had less potential for permanent deformation than the dense-graded mixes and that the OGFC mixes containing the polymer-modified binders performed best.

1.54.4 Construction Practices

No information is provided on construction practices of friction course.

1.54.5 Maintenance Practices

No information is provided on maintenance practices of friction course.

1.54.6 Rehabilitation Practices

No information is provided on construction practices of friction course.

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1.54.7 Performance
No information is provided on the field performance of friction course.

1.54.8 Structural Design
No information is provided on structural design of friction course.

1.54.9 Limitations
No information is provided on limitations of use.

1.55 Pucher, E., J. Litzka, J. Haberl, and J. Girard. "Silvia Project Report: Report on Recycling of Porous Asphalt in Comparison with Dense Asphalt." SILVIA-036-01-WP3-260204. Sustainable Road Surfaces for Traffic Noise Control. European Commission. February 2004.

1.55.1 General
This report provides an overview for two separate issues: cleaning/maintenance of porous asphalt and the aspects associated with recycling porous asphalt. The report is a synthesis of practices found in the literature by the authors.

1.55.2 Materials and Mix Design
No specifics on materials and mix design were given.

1.55.3 Benefits
No specifics benefits were given.

1.55.4 Construction Practices
No specifics construction practices were given.

1.55.5 Maintenance Practices
The report provides information on three aspects of maintenance: structural maintenance, cleaning (unclogging) and winter maintenance. The authors indicate that raveling is the most common mode of distress for porous asphalt layers. Neither rutting nor cracking (except for reflective cracking) have been perceived as a major problem for porous asphalt. A joint Dutch and Belgian study cited by the authors indicate that typical structural maintenance methods conducted on porous asphalt include fog seal sprays, in-place recycling, overlays and cold-laid porous asphalt for local repairs (including potholes). Minor repairs can be made with dense-graded HMA as long as the size of the repair is not more than 10 percent of the surface area.

A 1990 study referenced by the authors discussed the use of a suction sweeper with water jets for cleaning/unclogging porous asphalt pavements. However, in 1993 a PIARC report stated that cleaning porous asphalt with either high-pressure water or the vacuum sweeping machines have not been encouraging. Improvements in drainage capacity with these two methods were limited to the top portion of the layer and were only temporary. Studies from Austria indicated that cleaning should be started early in the life of a porous

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asphalt layer and by starting earlier in the life that the drainage characteristics could be maintained longer.

The authors noted that porous asphalts require more salt per unit area for winter maintenance than does dense-graded HMA surfaces. Brines are generally ineffective because they have a tendency to drain into the porous asphalt layer. Pucher et al state that there is a tendency toward the use of electronic warning systems to assist in selecting the appropriate time for winter maintenance in France and the Netherlands. The authors state that there is a need for a deicing agent that will stay on the surface of porous pavements and not drain into the void structure. They indicated that Calcium Magnesium Acetate (CMA) mixed with another material [this material was not given] should stay at the pavement surface.

1.55.6 Rehabilitation Practices
The authors state that as of 1997, very little, if any, recycling of porous asphalt had occurred. However, Pucher et al indicate that both cold-mix and hot-mix recycling are options for porous asphalt pavements. Cold-mix recycling would be a process where the reclaimed porous asphalt would be combined with new asphalt and/or recycling agents to produce cold base mixtures. In contrast, hot-mix recycling would be the process of taking reclaimed porous asphalt and combining with new materials through a hot mix production facility. According to a European point of view, hot-mix recycling would be the highest level of value. Due to expected large amounts of porous asphalt planned by the Dutch, they are placing an emphasis on recycling porous asphalt, both in-plant and in-place.

1.55.7 Performance
The authors indicated that the most common distress in porous asphalt pavements is raveling. This problem generally occurs very quickly once the flow of traffic begins and usually originates from placing the porous asphalt at too low of a temperature, incomplete compaction or from segregation of the binder (draindown). Two studies were cited by the authors on service life. One indicted that after 5 to 10 years, porous asphalt slowly degrades. As time increases past this point, the speed of distress development (mostly raveling) also increases. Experience in the Netherlands indicates that the approximate service life of porous asphalt pavements is 10 years while for dense-graded asphalt the service life is 12 years.

1.55.8 Structural Design
No specifics on inclusion within structural design were given.

1.55.9 Limitations
No specific limitations were given.

1.56 Punith, V. S., S. N. Suresha, A. Veeraragavan, S. Raju and S. Bose. "Characterization of Polymer and Fiber-Modified Porous Asphalt Mixtures." TRB 2004 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2004.

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1.56.1 General

Punith et al present the results of a study carried out to evaluate the laboratory performance of different porous mixtures. Performance related to abrasion, moisture-induced damage, fatigue, plastic deformation and the coefficient of friction were evaluated. Punith et al indicate that polymer-modification of the binder enhanced the properties of the porous asphalt mixtures.

1.56.2 Benefits of Permeable Asphalt Mixtures

Punith et al describe the different benefits of porous mixtures (also referred in the text as friction or popcorn mixes). These include improvement of surface frictional resistance, minimization of hydroplaning, reduction of splash and spray, improvement of night visibility and lowering pavement noise levels.

Punith et al indicate that an additional benefit of using fibers in porous mixtures is increased asphalt binder content and, hence, increased film thicknesses and durability. They also mention that the use of modified binder allows a thicker binder film thickness on the aggregate particles and, hence, reduces oxidation and raveling of aggregate. particles

1.56.3 Materials and Design

The different properties of the materials are provided in Table 107. A granite aggregate and three types of asphalt binder – one unmodified and the other two modified (crumb rubber and reclaimed polyethylene) were used in the study. Cellulose fibers were used with the unmodified asphalt binder.

Punith et al mention that crumb rubber (CR) is an elastomeric polymer obtained from waste tires, which is added in crumb form to base asphalt under agitation. They also indicate that because of the presence of elastomer and reduced stiffness at low temperatures, the modification of a binder with CR may thus result in a substantial improved fatigue life by reducing fatigue cracking.

Punith et al indicate that the Reclaimed Polyethylene was obtained from low-Density Polyethylene (LDPE) carry bags collected from domestic waste, and these bags were shredded into approximately 3mm by 3 mm size. They mention that LDPE is plastomeric polymer which is added to base asphalt with a high-speed stirrer rotating at a speed of 3500 rpm for period of 25 minutes. The authors mention that at high service temperatures, the presence of plastomer improves the stiffness of the mixtures and improved resistance to rutting.

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Table 107: Properties of Materials and Information on Mix Design

Materials/Type	Properties
Aggregate/Granite	Specific Gravity: 2.67 Water Absorption, %: 0.42 Impact Value, %: 16.1 Gradation: Sieve Size, mm — Percent Passing 19.0 — 100 13.2 — 95 9.50 — 45 4.75 — 13 2.36 — 10 0.075 — 5
Asphalt Binder/60/70 Pen/Unmodified	Penetration (77^0F): 66 Softening Point: 50 Ductility (cm): +75 Specific Gravity: 1.02 Elastic Recovery (59^oF): 5 %
Asphalt Binder/Modified/12 % Crumb Rubber (CRMB)	Penetration (77^0F): 64 Softening Point: 64 Ductility (cm): 61 Specific Gravity: 1.021 Elastic Recovery (59^oF): 52 %
Asphalt Binder/Modified/5 % Reclaimed Polyethylene (RPEB)	Penetration (77^0F): 61 Softening Point: 62 Ductility (cm): 57 Specific Gravity: 1.019 Elastic Recovery (59^oF): 40 %

Punith et al. mention that the introduction of fibers into bituminous mixtures reinforces the binder system, thus causing an increase in the viscosity of the system. They indicate that the resulting mixture could have greater stability and possibly higher resistance to fatigue cracking, and can also prevent the draindown of the binder in porous mixtures. Cellulose fibers were used at the rate of 0.3 percent based on total mixture weight.

1.56.4 Construction Practices
No information is provided on construction practices**.**

1.56.5 Maintenance Practices
No information is provided on maintenance practices

1.56.6 Rehabilitation Practices
No information is provided on rehabilitation practices

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1.56.7 Performance

Punith et al provide the results of basket drainage testing carried out on mixtures at mixing temperature of 170˚C (338˚F), 160˚C (320˚F) and 170˚C (338˚F) for CRMB, 60/70-grade binder with fiber and RPEB respectively. The use of fiber did lower the draindown significantly, especially at higher asphalt contents, whereas the use of reclaimed polyethylene only (without fiber) did not lower draindown, as compared to the 60/70 grade binder.

For mix design, two types of Marshall compactive efforts were used. In the first type, mixtures at different asphalt binder contents were compacted with 25 blows per face and in the second type, mixtures were compacted with 50 blows on only one face of the specimen by Marshall hammer. Punith et al. mention that the optimum binder content was defined as the binder content that produces the lowest voids in mineral aggregate (VMA). Mix design results are shown in Table 108.

Table 108: Results of Mix Design

Compactive Effort	Mix	Asphalt Binder Content, %	Properties				
			Theoretical Maximum Density	Air voids, %	VMA, %	VCA, %	VFA, %
25 Blow	CR Modified	5.2	2.487	18.9	28.3	34.5	34.3
	60/70 + Fiber	5.3	2.474	19.1	28.8	34.4	33.6
	RP Modified	5.0	2.491	18.5	27.9	34.0	33.5
50 Blow	CR Modified	5.2	2.487	18.8	27.9	33.9	35.1
	60/70 + Fiber	5.3	2.474	18.7	26.	32.5	38.0
	RP Modified	5.0	2.491	17.9	27.5	33.4	34.6

Punith et al indicate that the resistance to compacted porous asphalt mixtures to abrasion loss was analyzed by means of the Cantabro test. They mention that this is an abrasion and impact test carried out in the Los Angeles Abrasion Machine (ASTM C131). In this test, the initial mass of compacted sample is recorded as P_1. The specimen is then placed in the Los Angeles Rattler without the charge of steel spheres. The operating temperature is usually 250˚C (770˚F). The machine is operated for 300 revolutions at a speed of 30 to 33 rpm. The test specimen is then removed and it's mass determined to the nearest 0.1 gram P_2. The percentage abrasion loss (P) is calculated according to: $P = 100\,(P_1 - P_2)/P_1$; the recommended maximum permitted abrasion loss value for freshly compacted specimens is 25 percent.

Punith et al's results show that the abrasion loss was the lowest for the RPEB modified mixes, and the highest for the 60/70 Pen mix, with the CRMB mix falling in between.

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Also, in general, the abrasion loss decreased with an increase in asphalt content and decrease in air voids.

A falling-head laboratory permeability test was used to evaluate the permeability of the different mixes. Punith et al indicates that there was a reduction in permeability with an increase in asphalt binder content. It should be noted that an increase in asphalt binder content also caused a lowering of voids, and that the 25 blows mixes showed higher permeability compared to the 50 blows mixes. Considering all the results, the permeability dropped from a value between 500 and 550 liters per day to approximately 400 to 425 liters per day, for an increase in asphalt content of 4.5 to 6.0 percent.

Punith et al mention that the stiffness of porous asphalt is less than those of conventional, dense-graded wearing courses, and that these mixtures therefore have less ability to distribute traffic stresses than dense-graded mixtures. They mention that the stiffness of the porous asphalt mixtures are generally about half to two-thirds of dense-graded mixtures depending on the amount of voids within the mixture (the higher the void content, the lower the stiffness of the mixture).

Punith et al present the results of different tests conducted in indirect tensile mode – resilient modulus, indirect tensile strength, and calculated tensile strength ratios and fatigue lives for the different mixes. They indicate that the use of binder modified by addition of CRMB and RPEB improve the fatigue resistance of mixtures significantly when compared to mixtures with cellulose fibers for both compactive efforts. Punith et al also show that the tensile strength ratio values of the porous asphalt mixtures with modified binders were significantly higher when compared with asphalt mixtures with fibers.

Punith e al indicate that rutting characteristics were studied using Hamburg Wheel Tracking Device (HWTD) at 45˚C (113˚F), using samples compacted with 5.2, 5.3 and 5.0 percent asphalt binder (by mass of aggregates) for the three different asphalt binders (60/70-grade binder with fiber, CRMB and RPEB). They describe the HWTD as a wheel-tracking device consisting of a loaded wheel and a confined mold in which the 300 mm x 150 x 50 mm specimen for porous asphalt mixtures is rigidly restrained on its four sides. A motor and a reciprocating device give the wheel a to and fro motion of 24 passes a minute with a distance of travel of 300 mm. The solid rubber tired steel wheel bears a total load of 31 kg and indents a straight track in the specimen. The depth of the deformation was recorded at the midpoint of its length by means of a rut depth-measuring device. The contact area between the wheel and specimen is about 5.457 sq cm giving a mean normal pressure 566 kPa.

Punith et al observed that the resistance to plastic deformation was enhanced (compared to mixes with unmodified binder with fiber) with the use of RPEB and CRMB binders. The authors mention that the use of polymeric bitumen (CRMB or RPEB) can diminish the effect of post compaction by traffic, which is sometimes observed in porous asphalt mixtures.

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Punith et al mentions that skid resistance testing was carried out according to the ASTM 303 test method using the pendulum type skid resistance tester. As the test specimens used in this study were of 100 mm diameter and as the values of coefficient of surface friction of test specimens of different types of porous asphalt mixtures were only for comparative purposes, the slider assembly of aluminum backing plate with a bonded rubber strip of 31.8 by 25.4 by 6.35 mm was used to obtain a contact path length of 75 to 78 mm. The test specimens were tested for coefficient of friction values by applying sufficient quantity of water on the top surface area of the test specimens. The average value of coefficient of friction for the porous asphalt specimens was found to be 0.75, and no significant difference was found between the friction values of the different mixes (25 and 50 blow, and different types of asphalt binders)

1.56.8 Structural Design

No information is provided on structural design.

1.56.9 Limitations

No information is provided on limitations of use.

1.57 Tan, S.A., T.F. Fwa and K.C. Chai. "Drainage Considerations for Porous Asphalt Surface Course Design." Transportation Research Record 1868, TRB. National Research Council. Washington, D.C. 2004. pp. 142-149.

1.57.1 General

This paper discusses the development of drainage design charts used to determine the required thickness of a porous asphalt surface course for a given rainfall intensity. Tan et al theorized that the drainage performance of porous asphalt was a function of not only the drainage characteristics of the material, but also geometric design of the roadway. They further theorized that an under designed porous asphalt would not be able to keep the pavement surface dry under wet conditions.

A 3D finite element program (SEEP3D) was utilized to construct and analyze models with varying rainfall intensities, surface course thicknesses, width of pavement, longitudinal slope, and cross slope. These models were validated with laboratory seepage tests.

The analysis of these models showed that the drainage performance was enhanced with increased cross slope, regardless of the longitudinal slope. Similarly the effect of the longitudinal slope decreased as the cross slope and ratio of porous asphalt thickness to pavement width increased.

1.57.2 Benefits of Permeable Asphalt Mixtures

Tan et al did not discuss any benefits of permeable asphalt mixtures.

1.57.3 Materials and Design

Tan et al did not discuss materials and design.

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1.57.4 Construction Practices
Tan et al did not discuss construction practices.

1.57.5 Maintenance Practices
Tan et al did not discuss maintenance practices.

1.57.6 Rehabilitation Practices
Tan et al did not discuss rehabilitation practices.

1.57.7 Performance
Tan et al did not discuss performance of permeable asphalt mixtures.

1.57.8 Structural Design
From the finite element analysis that Tan et al performed, a series of drainage design charts were constructed. These charts allow the user to determine the ratio of porous surface asphalt thickness to width (mm/m) given the cross slope (%) of the road and the maximum rainfall intensity (m/s). Tan et al recommends the following procedure for using these charts:

1. Determine the vertical permeability of the porous asphalt that will be used as the surface course (k_v)
2. Select or determine the maximum allowable rainfall intensity (m/s) for the design storm
3. Adjust the rainfall intensity by multiplying by a factor equal to 20/ k_v. This corrects for the fact that the design drainage charts were constructed assuming the vertical permeability was 20 mm/s.
4. Find the appropriate design chart based on longitudinal slope. Then select the appropriate cross slope curve on this chart and read the ratio of porous surface asphalt thickness to width (mm/m).

Tan et al present drainage design charts for longitudinal slopes of 0, 2, 4, 6, 8, and 10 percent. Each of these charts has curves corresponding to 0, 1, 2, 3, and 4 percent cross slope.

1.57.9 Limitations
Tan et al did not discuss any limitations of permeable asphalt mixtures.

1.58 Watson, D. E., J. Zhang, R. B. Powell. "Analysis of Temperature Data for the NCAT Test Track." Transportation Research Record No: 189. Transportation Research Board. National Research Council. Washington, D.C. 2004.

1.58.1 General
The study reported in this paper was carried out to determine the maximum and minimum temperatures in pavements with different types of asphalt mix surfaces, compare these temperatures with those predicted by Strategic Highway Research Program (SHRP) and

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Long Term Pavement Performance (LTPP) models, and evaluate the effect of mix type, aggregate type, asphalt binder type and layer thickness on the maximum and minimum temperatures.

Watson et al indicates that a number of combinations of materials, mix and layer thicknesses were used in the National Center for Asphalt Technology (NCAT) Test Track near Auburn, Alabama. The different types of mix include Open-Graded Friction Course (OGFC) (3 sections), Stone Matrix Asphalt (SMA) (6 sections), Superpave mix (36 sections) and a mix designed with the Hveem method. High and low temperatures in each and every section were determined from temperature probe data, recorded over a period of two years.

The authors describe the process of predicting temperatures with the use of different models, as well as the effect of temperature on rutting. In this review, only that portion which is relevant to permeable friction courses, is discussed.

Watson et al conclude that the open surface texture of OGFC and SMA mixes result in underlying layers being about 2ºC (3.6ºF) cooler than when conventional dense-graded surface mixes are used.

1.58.2 Benefits of Permeable Asphalt Mixtures
No information is provided on benefits of permeable asphalt mixtures.

1.58.3 Materials and Design
No information is provided on materials and mix design.

1.58.4 Construction Practices
Watson et al mentions that the OGFC mix in the three sections was placed 18 mm (0.7 in) thick.

1.58.5 Maintenance Practices
No information is provided on maintenance practices

1.58.6 Rehabilitation Practices
No information is provided on rehabilitation practices

1.58.7 Performance
Watson et al presents the temperature data at different depths for the different pavements, as well as results of comparison of these data with data predicted from SHRP and LTPP models. Temperature was recorded with probes at the top and mid-depth of the surface course, and at the top and bottom of the binder course in each section. The authors mention that the Datalogger receives temperature data every minute and then records the minimum, maximum, and average pavement temperature every hour.

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The temperature data was used to evaluate measured versus predicted temperatures using SHRP and LTPP temperature models, evaluate the effect of mix type on pavement temperature, and compare the effect of surface layer thickness on pavement temperatures.

Watson et al mentions that most of the sections had a surface layer 50 mm (2 in) thick while some had surface layers of 18 and 38 mm (0.75 and 1.5 in) thick. Since the temperature gauge was located at the interface of layers, the depth of each temperature gauge below the surface was varied based on the thickness of each layer. Pavement temperature at depths up to 250 mm (10 in) was measured at the NCAT test track.

The general conclusions from comparison of predicted versus measured temperatures are shown in Tables 109 and 110.

Table 109: Conclusions from Comparison of SHRP Model Predicted Versus Measured Temperatures

Parameter	SHRP Model	Comments/Explanation
High temperature	1. SHRP high pavement model at 50 percent reliability estimates pavement temperatures fairly close, but in some cases slightly underestimates the measured temperature. At depths of 250 mm (10 in) the model varies from the measured temperature as much as 9.6ºC (17.3ºF). With 98% reliability, this model gave a close prediction of pavement temperatures in the upper layers in 2002, but overestimated the pavement temperatures in 2001. However, the SHRP model still significantly underestimated the pavement temperature at 250 mm (10 in) depth.	Assumption of maximum pavement temperature at a wind speed of 4.5 m/sec may not be proper; research suggests that consideration of wind speed of 1 m/sec to be more appropriate.
Low Temperature	SHRP low temperature model does not closely agree with measured values at either 50 percent or 98 percent reliability. At 50 percent reliability the SHRP model overestimated cold temperatures as much as 10.3ºC (18.5ºF) at a depth of 38 mm.	SHRP low temperature model does not consider the effect of latitude; surface temperature of the pavement was assumed to be equivalent to the low air temperature. This does not consider that heat will be radiated to the pavement surface from underlying layers during the early morning hours when air temperatures are generally lowest.

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Table 110: Conclusions from Comparison of LTPP Model Predicted Versus Measured Temperatures

Parameter	LTPP Model	Comments/Explanation
High Temperature	The LTPP high temperature model at 50 percent reliability provided a close estimation for 2001 within 2.2ºC (4ºF). However, in 2002 it underestimated the pavement temperature as much as 5.6ºC (10.1ºF) except for the temperature at 250 mm (10 in.) depth. At 250 mm (10 in.) deep, the LTPP model gave fairly close predictions in both 2001 and 2002. With a reliability of 98%, the LTPP high temperature overestimated the temperature for all cases.	
Low Temperature	The LTPP models at 50 percent reliability did closely compare to measured values in 2001, but overestimated cold pavement temperatures in 2002.	

In their discussion on the effect of mixes on temperature, Watson et al shows a comparison was made of temperatures at various depths for the OGFC, SMA, and Superpave sections. They show that the layer interface beneath the OGFC (at the middle of the research layers) in July 2001 was 1.7ºC (3.1ºF) cooler than for the SMA and 2.1ºC (3.8ºF) cooler than when Superpave surface mix was used. In 2002, the temperature beneath OGFC and SMA surface mixes was virtually the same at 53.7ºC (128.6ºF) and 53.6ºC (128.5ºF) respectively, while the temperature under the Superpave surface was 55.1ºC (131.2ºF), resulting in a difference of 1.4ºC (2.6ºF). Watson et al mentions that the open surface texture of OGFC and SMA mixes may allow underlying mixes to be slightly cooler than when conventional dense-graded surface mixes are used. The OGFC mix in these sections was placed 18 mm (0.7 in) thick while the SMA and Superpave mixes were placed 38-50 mm (1.5-2 in) thick.

1.58.8 Structural Design
No information is provided on structural design.

1.58.9 Limitations
No information is provided on limitations of use.

1.59 Watson, D. E., L. A. Cooley, Jr., K. A. Moore, K. Williams. "Laboratory Performance Testing of OGFC Mixtures." Transportation Research Record No: 1891. Transportation Research Board. National Research Council. Washington, D.C. 2004.

1.59.1 General
Watson et al describe the results from a laboratory study conducted with OGFC mixes. The objectives of this study were to evaluate certain criteria which had been previously proposed by NCAT researchers. Specifically, they looked at methods of determination of

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air voids, design air voids, compaction of mix samples for abrasion loss testing, use of fiber and polymer modified asphalts for reducing draindown, methods of freeze-thaw conditioning for determination of moisture susceptibility and permeability criterion for mix design.

Watson et al used three different types of aggregates, three different types asphalt binder, one fiber and Marshall and Superpave gyratory compaction procedures.

Watson et al conclude that the CoreLok procedure appeared to be a more accurate method of determining bulk specific gravity of OGFC mixes, the minimum air void content for new-generation OGFC mixtures should be 18 percent based on the dimensional method and 16 percent based on the CoreLok method, the addition of fiber stabilizers significantly reduces the potential for draindown, SGC compacted samples can be used for the Cantabro stone loss test procedure, unconditioned SGC samples should have stone loss of no more than 20 percent, the aging procedure is not necessary for Cantabro test and that one freeze-thaw cycle is sufficient for determination of moisture susceptibility.

1.59.2 Benefits of Permeable Asphalt Mixtures
No information is provided on benefits of permeable asphalt mixtures.

1.59.3 Materials and Design
Watson et al mentions that three different types of aggregates and three different types of asphalt binder were used in this study. The matrix of materials used is provided in Table 111.

Table 111: Materials and Mixes Used by Watson et al

Material/property	Type
Aggregate	Granite, siliceous crushed gravel, and traprock
Asphalt Binder	PG 67-22, PG 76-22 (SBS modified) and PG 76-34 (rubber modified; chemically modified within the refining process).
Stabilizer	Fiber, 0.4% of the total mix weight.
Mix	With fiber - PG 67-22, PG 76-22, PG 76-34 Without fiber - PG 67-22, PG 76-22

1.59.4 Construction Practices
No information is provided on construction practices

1.59.5 Maintenance Practices
No information is provided on maintenance practices

1.59.6 Rehabilitation Practices
No information is provided on rehabilitation practices

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1.59.7 Performance

Watson et al provides the results of different tests conducted on the OGFC mixes. These results were used to develop guidelines/criteria for laboratory mix design. The tests were conducted to answer the following questions:

1. Is using dimensional analysis a good procedure to determine air voids in OGFC mixes? A major problem with the dimensional method is that the specimen is assumed to be a smooth-sided cylinder and does not account for the open texture of the sample; specimens have higher calculated voids with the dimensional method than are actually present.
2. Is changing binder grade a better option compared to adding fiber, for reducing draindown? Is the existing 0.3 percent maximum draindown criterion acceptable?
3. Can samples compacted with the Superpave gyratory compactor be used for testing for abrasion loss (according to Cantabro method)?
4. Is the current criterion of 100 m/day appropriate for the mix design of OGFC?
5. Is there a requirement for five day freeze-thaw conditioning for evaluation of moisture susceptibility OGFC mixes during mix design?

These test methods are provided in Table 112.

Watson et al indicates that there was a very good relationship between air voids obtained from the dimensional and CoreLok methods with an R-square value of 0.84, dimensional method results in air voids were consistently higher than the CoreLok method and that the difference between dimensional and CoreLok air voids increases at an increasing rate as the dimensional air voids increase. From the data the authors infer that if 18 percent air voids is selected as a minimum void content based on the dimensional method, a minimum value based on the CoreLok procedure would be approximately 16 percent.

Regarding draindown, Watson et al mentions that the use of fiber stabilizer significantly improved resistance to draindown, much better than increasing binder grade. There was more than 4 percent drain-down in some specimens prepared without fiber while the addition of 0.4 percent fiber by weight of the total mix resulted in minimal draindown. All aggregate-binder combinations met draindown requirements when a fiber stabilizer was added.

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Table 112: Tests for Different Properties

Property	Standard Tests and Procedures Used in this Study
Air Voids	Dimensional volume, CoreLok
Draindown	NCAT draindown test; draindown was determined for three binder grades – PG 67-22, PG 76-22 and PG 76-34 and three aggregate types.
Abrasion/ Stone Loss/	In this study, samples were compacted using both Marshall hammer and the Superpave gyratory compactor (SGC). The SGC samples were compacted to 50 gyrations and the Marshall samples were compacted to 50 blows per face (per standard procedure). Testing was completed using the medium gradation and a PG 67-22 binder and then repeated with a PG 76-22 polymer modified binder. Samples in this study were conditioned by placing them in a forced draft oven at 64°C for seven days. The temperature was set at 64°C to correspond to the average 7-day high pavement temperature for much of the U.S. based on Superpave criteria. The conditioned samples were allowed to cool at room temperature (25°C) a minimum of four hours before testing.
Permeability	Falling head permeability; method adopted from Florida Department of Transportation
Moisture Susceptibility	Modified AASHTO T283; Use 30 minutes for vacuum saturation under water and five freeze-thaw cycles for the conditioning process. In this study samples were prepared and tested after 1, 3, and 5 freeze-thaw cycles. The intermediate cycles were added to determine if there was a significant difference in the three conditioning methods.

In checking the Cantabro Abrasion loss of OGFC mixes, Watson et al conducted statistical analysis to determine differences (if any) between samples compacted with Marshal hammer and SGC, and between aged and unaged samples (compacted with Marshall hammer). Three replicates were made for each test and results for stone loss were averaged.

Watson et al mention that t-test with the Cantabro test results showed no significant difference between aged versus unaged Marshall specimens at a 95 percent confidence level. They infer that if the aging procedure is to be used for SGC samples, a maximum stone loss value should be 24 percent. Again, the t-test for SGC conditioned versus unconditioned did not indicate any significant difference in the data at the 95 percent confidence interval. These tests were run with and without fiber and with and without polymer modifier. The authors concluded that since there are no significant difference between results for aged and unaged samples for both Marshall and SGC samples, the extra week needed for the aging process need not be spent for mix design of OGFC.

The authors mention that the use of polymer-modified asphalt made a significant difference in results for the Cantabro Abrasion loss test. Abrasion loss was reduced considerably as the PG binder grade increased in stiffness for both conditioned and unconditioned samples. Tests with the PG 76-34 binder had virtually no stone loss for both conditioned and unconditioned samples.

Regarding permeability test results, Watson et al mentions that three replicates were made and results were averaged for each reported test. Sample combinations that met draindown and Cantabro requirements were tested for permeability using a falling head permeameter.

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Permeability tests were performed using the procedure adopted by the Florida Department of Transportation for mix designs with specimens compacted at 50 gyrations with the SGC. They mention that permeability values failed to meet the criterion of 100m per day (minimum) in several cases.

Based on previous research, permeability values of at least 100 meters per day were recommended for acceptable performance. Failing results were typical especially for the fine-graded blends with the exception of the traprock mixtures. Generally, the permeability can be increased by making the aggregate gradation coarser. However, when the coarser gradation was used in this study, it also resulted in a greater potential for draindown problems.

Watson et al indicate that for the fine gradations used in this study the 100 meters per day criterion may be difficult to achieve for some aggregate types. They also mention that the permeability test, however, was highly variable and had a within lab standard deviation of 22.76 m/day. Also, draindown of the asphalt binder through the specimen may account for some of this variability, since draindown caused some of the samples to become sealed around the bottom of the specimen so that they were impermeable. For specimens where this was discovered, samples were remade. The authors also mention that aggregate type was also found to have an effect on permeability results. They explain this with the fact that the total volume of aggregate in the sample will change with a change in aggregate bulk specific gravity.

From the data the authors infer that dimensional air voids would have to be about 20 percent in order to meet the standard permeability of 100 meters/day. The 20 percent air voids is consistent with values being used in Europe. In order to meet permeability of 100 meters/day, the CoreLok air voids would have to be approximately 18 percent.

Watson et al mention that in evaluation of moisture susceptibility, very little overall change was observed (no significant difference was observed in Analysis of Variance, ANOVA) between the three conditioning methods used (1,3 and 5 day freeze-thaw) for samples with PG 76-22 binder and fiber stabilizer. By using only one freeze-thaw cycle for the moisture conditioning process the time needed to perform an OGFC mix design can be reduced by about two weeks over the previous method that recommended five freeze-thaw cycles. They show that both the granite and gravel mixtures exhibited a slight tendency to gain strength with additional freeze-thaw cycles although the traprock showed a slight decrease in strength. These samples averaged more than 100 percent retained tensile strength when compared to unconditioned samples.

Watson et al showed that tensile strength values for mixture using PG 76-34 binder and fiber stabilizer were significantly lower than the samples with PG 76-22 binder, and that, the tensile strength was virtually unchanged as the number of freeze-thaw cycles increased. The tensile strength ratio of conditioned to unconditioned samples was higher than the minimum recommended value of 80 percent in each case. Based on the assumption that 80 percent retained strength is a criterion for identifying good and poor

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mixes, the authors infer that the mixture with PG 76-34 was still resistant to moisture damage although the actual tensile strength was much lower than expected.

1.59.8 Structural Design
No information is provided on structural design.

1.59.9 Limitations
No information is provided on limitations of use.

1.60 Watson, D. E., E. Masad, K. A. Moore, K. Williams, L. A. Cooley, Jr. "Verification of VCA Testing To Determine Stone-On-Stone Contact of HMA Mixtures." Transportation Research Record No: 1891. Transportation Research Board. National Research Council. Washington, D.C. 2004.

1.60.1 General
In this paper, Watson et al describe a laboratory study evaluation of stone-on-stone contact in Open Graded Friction Course (OGFC). Working on the premise that stone-on-stone contact is important for the proper functioning of OGFC, the authors conducted this study to determine whether stone-on-stone contact exists in mixes with different aggregates (same gradation, but using different compaction methods and levels), verify the existence of stone-on-stone contact in these mixes with digital imaging techniques and also to determine the effect of aggregate breakdown (during compaction) on stone-on-stone contact.

Watson et al considered the voids in coarse aggregate in mix (VCA_{MIX}) being less than the dry-rodded voids in coarse aggregate (VCA_{DRC}) as the condition to determine existence of stone-on-stone contact. Using the percent retained on the 4.75 mm sieve as the coarse aggregate fraction (breakpoint sieve), the authors have shown that for this specific 12.5 mm Nominal Maximum Aggregate Size (NMAS) mix, only one of the fifteen different mixes did meet the $VCA_{MIX}<VCA_{DRC}$ criterion. On the other hand, they found all but one mix met the criterion, when the 2.36 mm sieve was used as the "breakpoint" sieve. Watson et al. made a recommendation that the finest sieve to retain more than 10 percent of aggregates be considered as the "breakpoint" sieve for determination of stone-on-stone contact.

With *X*-Ray computed tomography images, Watson et al. used three different image analysis techniques to verify stone-on-stone contact in OGFC mixes: number of contacts, level of contacts, and air void size distribution. The authors found reasonable to good correlation between the results of the three methods. The general conclusions were that there was more stone-on-stone contact in Marshall compacted specimens than in Superpave gyratory compacted specimens and an increased level of compaction did increase the number of "contacts" for mixes with all three aggregates except one.

Watson et al indicate that the amount of aggregate breakdown was higher for Marshall compaction than for Superpave gyratory compaction. They suspect that the higher breakdown caused greater number of intermediate particles and lowered the VCA_{MIX}, or

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that the lower compaction effort was not enough for locking the coarse aggregate particles together.

In general, Watson et al concluded that the finest sieve to retain more than 10 percent of aggregates can be considered as the “breakpoint” sieve for determination of stone-on-stone contact. They also indicate that digital image analysis can provide an independent method of verifying stone-on-stone contact in mixes.

1.60.2 Benefits of Permeable Asphalt Mixtures
No information is provided on benefits of permeable mixes.

1.60.3 Materials and Design
Watson et al used three different aggregates, two different compaction techniques and several compaction levels. The materials and matrix of mixes are shown in Table 113.

Table 113: Materials and Mixes

Material/Property	Type/Test Results	
Asphalt Binder	Polymer modified PG 76-22	
Asphalt Binder content, %	6.0	
Aggregate	Granite, Gravel, and Traprock	
Bulk Specific Gravity (Range)	2.599 to 2.927	
LA Loss, % (Range)	17 – 36	
Gradation	Sieve Size, mm	Percent passing
	19	100
	12.5	90
	9.5	45
	4.75	16
	2.36	7
	1.18	6
	0.6	5
	0.3	4
	0.15	3
	0.075	2.5
Additive	Fiber	
Additive content	0.3 % of total mix weight	
Compaction method	Superpave gyratory and Marshall	
Compaction level for Marshall	25, 50	
Compaction level for Superpave gyratory	30, 45, 60	

1.60.4 Construction Practices
No information is provided on construction practices**.**

1.60.5 Maintenance Practices
No information is provided on maintenance practices

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1.60.6 Rehabilitation Practices
No information is provided on rehabilitation practices

1.60.7 Performance
Watson et al evaluated the performance of the different mixes, in terms of stone-on-stone contact, as determined from VCA calculations and digital image analysis. The paper presents results of volumetric calculations to determine the existence of stone-on-stone contact ($VCA_{MIX}<VCA_{DRC}$), results of digital image analysis in terms of number of contacts, level of contacts and air void distribution, and discussion on the results from the analysis of volumetric and image data.

Watson et al indicate that for Marshall compaction, the VCA_{MIX} decreased with an increase in compactive effort. However, using the 4.75 mm sieve as the breakpoint sieve, the results showed that that only the gravel mix at 50 blows met the requirements of having $VCA_{MIX} < VCA_{DRC}$. The authors speculate that this may be because the rounded gravel particles would more easily move past each other during the compaction process. Similar results were found for mixes compacted with the Superpave gyratory compactor, but in this case, none of the three mixes for the 4.75 mm breakpoint sieve met the requirements of $VCA_{MIX} < VCA_{DRC}$. The authors mention that further reduction of percent passing the 4.75 mm sieve (to lower the VCA_{MIX}) was not possible since the mix already had only 17 percent passing the 4.75 mm sieve. Also, the authors conclude that it is not the aggregate breakdown that is preventing the mixes from meeting the $VCA_{MIX}<VCA_{DRC}$ criteria, since the traprock aggregates, which showed the lowest breakdown, had the highest difference between the VCA_{MIX} and VCA_{DRC}.

Working on the assumption that most of the mixes did not meet the VCA_{MIX} criteria, not because the criterion was not applicable, but because the breakpoint of 4.75 mm should be different, the authors show the VCA_{MIX} and VCA_{DRC} data for the 2.36 mm sieve (as breakpoint sieve). With the 2.36 mm sieve as the breakpoint sieve, all of the mixes except for the traprock 25-blow Marshall mix, were found to meet the $VCA_{MIX} < VCA_{DRC}$. Hence, the authors conclude that the 2.36 mm sieve is a better choice for breakpoint sieve for this OGFC mixture.

In digital image analysis, Watson et al provided the average number of contacts for all the images captured within a specimen per unit volume (between 100 to 150 images/specimen). The packing parameter is calculated from the "Level of Contact Regions" method. A low packing parameter indicates that the particles have a strong level of contact (large area of contacts) and they are more difficult to be separated by the erosion operation compared with the ones that have higher packing parameter. The authors provide the ranking of the specimens according to the size of air voids at which 25 percent, 50 percent, and 75 percent of total air voids are smaller. Based on the hypothesis, that smaller air void size indicates denser packing, the authors mention that the gravel mixes at 50 Marshall blows showed the highest number of contacts or packing.

The authors mention that a comparison between the number of contacts method showed an increase of about 30 percent in the number of contacts for gravel and traprock

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specimens compacted to 60 gyrations compared with the ones compacted to 30 gyrations. There was almost no difference between the SGC granite specimens. The Marshall specimen at 50 blows had 20 percent more contacts than the Marshall specimen at 25 blows. The Marshall specimen at 50 blows had about 210 percent more contacts than the SGC specimen at 60 gyrations.

Watson et al mention that the second method of using the level of contact region showed that the packing level increased by about 60 percent for the gravel SGC specimens compacted to 60 gyrations compared with the one compacted to 30 gyrations. Also, the packing level increased by about 60 percent for the Marshall specimens at 50 blows compared with the SGC specimens at 60 gyrations. This method, however, predicted that the level of packing decreased with increasing the number of gyrations for the granite and traprock specimens.

For the air void distribution method, Watson et al indicates that there was a slight difference in packing for granite and traprock specimens compacted at 60 and 30 gyrations. However, the difference in packing was higher between the SGC gravel specimens compacted at 60 and 30 gyrations. Also, the packing of the Marshal specimens at 50 blows was about 30 percent to 50 percent more than the packing in the SGC specimens at 60 gyrations. The authors mention that this may be caused by the additional aggregate breakdown of the Marshall compaction versus the SGC. There was almost no difference in packing between the Marshall specimens at 25 and 50 blows.

The authors conclude that in general, the Marshall specimens had more contacts than the SGC specimens and that the gravel specimens were more influenced by the increase in the number of gyrations from 30 to 60 than the traprock and granite specimens

In their discussion, Watson et al point out that the fact that the granite mix compacted with the 25-blow Marshall hammer only marginally met requirements seems to indicate that the 25-blow Marshall method may not provide enough compaction energy to adequately lock the coarse aggregate particles together. They also indicate that the digital image analysis procedure showed stone-on-stone contact in samples of all mixes and, hence, the concept of VCA is applicable, but the concept of using the same breakpoint for all gradations is not. They indicate that the slope of the gradation curve should be a factor in making that decision. For breakpoint sieve, the authors provide a recommendation of using the finest sieve size for which there is at least 10 percent of the total aggregate retained.

From the summary of the image analysis results, Watson et al. mention that a minimum number of contacts per specific area may be required to ensure stone-on-stone contact is obtained. For example, a minimum value such as 100,000 contacts per cubic meter may be recommended. They mention that this specification may be an advantage over the VCA method in that with digital imaging the number of contact points can be readily determined while the VCA method only gives a “yes” or “no” answer as to whether stone-on-stone contact exists. However, the authors do mention that more research is needed to compare aggregate packing from laboratory compaction to field compaction

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and performance before establishing a minimum value for number of contact points needed to ensure stone-on stone contact; but this method seems promising.

The authors refer to the use of image analysis for optimizing the gradations for different aggregates with different properties, through the determination of stone-on-stone contacts and establishing both a minimum and maximum number of contacts per cubic meter.

1.60.8 Structural Design
No information is provided on structural design.

1.60.9 Limitations
No information is provided on limitations of use.

1.61 Bennert, T., F. Fee, E. Sheehy, A. Jumikis and R. Sauber. "Comparison of Thin-Lift HMA Surface Course Mixes in New Jersey" TRB 2005 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2005.

1.61.1 General
Bennert et al. provides a comparison of performance and cost-effectiveness of different types of pavement surfaces used in New Jersey. They describe results of tests conducted to evaluate performance related to noise, skid resistance and roughness, and also provide cost information.

The different types of surfaces include modified open-graded friction course (MOGFC), Asphalt Rubber OGFC (AROGFC), 12.5 mm nominal maximum aggregate size Superpave Hot Mix Asphalt (HMA), microsurfacing (specifically, Novachip), Stone Matrix Asphalt (SMA), and unfinished, diamond ground and transverse tined Portland Cement Concrete (PCC).

The authors provide objectives of using different surfaces, descriptions of the different pavement structures for these surfaces, types of field tests, description of winter maintenance activities for the OGFC, and comparison of results and costs.
No particular surface showed the best performance in all tests. For example the transverse tined PCC did not show the high performance in roughness or noise reduction but did show the best performance in skid resistance.

Bennert et al. indicate that the overall performance and cost comparison showed that the OGFC provides the most cost effective surface, followed by the 12.5 mm HMA and the diamond ground PCC. They note that the second best performers, the HMA and the PCC cost significantly more than the OGFC because of its (OGFCs) relatively thin lift thickness and also because the diamond grinding can be conducted only on intact PCC surfaces.

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1.61.2 Benefits of Permeable Asphalt Mixtures

No information on specific benefits of PFCs is given in the paper. However, in discussing the objectives of using relatively thin lifts in New Jersey pavements, the authors refer to certain desirable qualities, that are mostly present in PFCs. For example, Bennert et al. mentions that the New Jersey Department of Transportation (NJDOT) wishes to use a mix that can reduce noise, improve skid resistance, provide a smooth ride and at the same time can be placed in relatively thin lifts (25 mm). The fact that these pavements are on very busy routes rule out the possibility of using cold paving materials (which need time for curing), and NJDOT would like to use a mix that can maximize the coverage area, that is, in per unit of time. These mixes are viewed as “maintenance” mixes and the authors note that one of the desirable qualities is good durability, which would help NJDOT avoid frequent disruption of traffic.

1.61.3 Materials and Design

The mix design information on the open-graded friction courses (and other surface courses) is shown in Tables 114 through 116. Note that the authors provide mix design information on several mixes used in several projects, only three of which have been described in detail (for construction).

Table 114: Mix Design Information

Property	Mix/Project/Design Year			
	AROGFC, I-1 95, Jackson/ 1992	MOGFC, US 24, Union/ 1999	MOGFC-2, I 195, Hamilton Square/ 2001	Novachip, I 195, Hamilton Square/2001
Gradation, percent passing Sieve Size, mm				
19		100	100	
12.5	100	86	91	100
9.5	94	57	74	78
4.75	39	13	18	27
2.36	10	9	7	22
1.18	7	6	4	16
0.6	5	5	3	13
0.3	4	4	3	10
0.15	4	3	2	7
0.075	3.0	2.8	2.0	5.9
Design Method	25 blow Marshall	50 gyration SGC	50 Gyration SGC	50 Gyration SGC
Design air voids, %	18	21.1	23.0	12.0
Additive/Modifier	15% GTR (#80), Wet Process	PG 76-22 (PMA) + 0.4 % Fiber	PG 76-22 (PMA) + 0.4 % Fiber	PG 76-22 (PMA) + Modified Emulsion Tack
Asphalt Binder Content, %	6.60	6.80	6.0	5.00

Note: The projects in which these four mixes were used have been described in detail

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Table 115: Mix Design Information Continued

Property	Mix/Project/Design Year			
	AROGFC/US 9, N/1993	MOGFC, I 78 W/2002	9.5 mm SMA, I 78 E/2002	Micro-S, Type 3, US 202 S, NJ 29/2002
Gradation, percent passing Sieve Size, mm				
19		100		
12.5	100	91	100	
9.5	88	59	93	100
4.75	36	15	45	96
2.36	12	6	25	72
1.18	8	4	18	52
0.6	6	3	16	36
0.3	5	3	15	22
0.15	4	2	13	15
0.075	3.0	1.8	11.6	10.7
Design Method	25 blow Marshall	50 gyration SGC	50 Gyration SGC	---
Design air voids, %	16.5	21.0	4.0	---
Additive/Modifier	15% GTR (#40), Wet Process	PG 76-22 (PMA) + 0.3 % Fiber	PG 76-22 (PMA) + 0.4 % Fiber	Latex modified
Asphalt Binder Content, %	6.50	6.20	6.90	6.00

Table 116: Mix Design Information Continued

Property	Mix/Project/Design Year		
	12.5 mm SP, US 22W/2000	12.5 mm SP, I 78 E/2003	12.5 MM SMA, US 1 N/S/1993
Gradation, percent passing Sieve Size, mm			
19	100	100	100
12.5	88	98	85
9.5	72	86	67
4.75	44	53	38
2.36	30	33	27
1.18	22	24	22
0.6	17	18	19
0.3	11	13	16
0.15	7	8	12
0.075	4.9	4.9	9.7
Design Method	125 gyration SGC	125 gyration SGC	25 blow Marshall
Design air voids, %	4.0	4.2	3.3
Additive/Modifier	PG 76-22 (PMA)	PG 76-22 (PMA)	AC 20
Asphalt Binder Content, %	4.80	4.90	6.00

1.61.4 Construction Practices

Bennert et al. provides description of the different layers, some construction details and costs for each of the sections evaluated. The descriptions are summarized in Table 117.

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Table 117: Description of Pavement Structure and Construction

Description	Project/Type		
	I-195 – Jackson Township/four lane interstate	**US 24 – Union/ four lane interstate**	**I-195 – Hamilton Square/four lane interstate**
Location	30 miles west of Trenton.	15 miles west of NY City.	5 miles west of Trenton.
Original pavement	6.5 inches of HMA, constructed in 1972	9 inches of PCC, constructed in 1974	Not given
Traffic, ESAL for 20 years	27 million	70 million	27.8 million
Intermediate course	3 inches of NJ I-2 HMA base	---	4-5 inches of 12.5 mm & 19 mm Superpave
Cause of rehabilitation	Not Given	Reduce noise	Enhance the structural number
Construction date	Sept. 1992	Paving: Fall, 1999 Reflection cracks noted on surface is Spring, 2000; Saw and seal done;	Novachip – Oct. 2000 / MOGFC-2 –May 2001
Overlay design	Mill 2 inches and fill 4 inches	1 ¼ inches + a heavy application of hot (PG64-22) tack	Mill 2 inches / replace 5-6 inches
Surface course	1 inch of Asphalt Rubber Open Graded Friction Course (AR-OGFC). A #80 mesh Ground Tire Rubber (GTR) was used in the “wet” process.	Modified Open Graded Friction Course #1 gradation	WB – ½” Novachip®; EB ¾” MOGFC-2
Cost of surface course	$7.30/yd^2$	$4.60/yd^2$	Cost of Novachip = $2.62/yd^2$; Cost of OGFC-2 = $1.76/yd^2$

1.61.5 Maintenance Practices

Bennert et al. mention that the cracks in the PCC pavement on the US-24 Union section had caused reflection cracks in the MOGFC between Fall of 1999 and Spring of 2000, and these cracks were subsequently sawed and sealed.

Regarding winter maintenance activities, the authors mention that the NJDOT uses rock salt for deicing, whereas the New Jersey Garden State Parkway (NJGSP) uses liquid magnesium chloride. The authors note that that the NJDOT finds the OGFC mixes to be more difficult to maintain ice-free than the adjacent dense-graded mixes, even though more frequent application of rock salts are made for these mixes. The NJGSP, on the other hand, had good success with the magnesium chloride, although the OGFC needs twice the amount of application as the dense-graded mix. The NJGSP continually monitors forecasts of temperature and measures surface temperatures, and pretreats OGFC surfaces with liquid magnesium chloride to avoid icing. If the magnesium chloride is applied after the OGFC is frozen then it washes off the surface. However, by pretreating, NJGSP has found that the OGFC surfaces are manageable and can be plowed the same as the dense graded mixes.

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1.61.6 Rehabilitation Practices

No information on rehabilitation practices is provided in this paper

1.61.7 Performance

Bennert et al. provides the results of performance of different types of surfaces, including OGFCs, in terms of noise reduction, skid resistance and ride quality. The tests carried out to evaluate these characteristics are shown in Table 118.

Table 118: Details of Tests Conducted

Property	Test Procedure
Noise	The tire/pavement noise was measured using the Close Proximity Method (CPX). In this method microphones are placed near the tire/pavement interface to directly measure the tire/pavement noise levels. It was developed in Europe and is defined by ISO Standard 11819-2. CPX testing was also conducted at three different vehicles speeds to evaluate the effect of vehicle speed on tire/pavement related noise.
Skid Resistance	The wet skid resistance of the pavement surfaces was determined using *ASTM E274-97, Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire*. The tests were conducted at a vehicle speed of 40 mph using a ribbed tire conforming to *ASTM E501-94, Standard Specification for Standard Rib Tire for Pavement Skid-Resistance Tests*.
Ride Quality	Ride Quality measurements were conducted using the NJDOT's Automatic Road Analyzer (ARAN).

Note: The tire/pavement noise, wet skid resistance, and ride quality measurements were all conducted within a 6 month time period of one another. In the case of the ride quality's RQI and IRI, the parameters were measured at the same time. Each measurement was typically taken at either 0.1 or 0.2 mile intervals. For comparison purposes, the measurements were averaged over the distance of the test section.

Bennert et al provided the results of noise level tests, as well as determination of noise gradients in the different types of surfaces. They indicate that the measured noise levels measured at a vehicle speed of 60 mph are used for the main comparison between the different pavement surfaces. A comparison of the thin-lift surfaces (OGFC, Novachip®, SMA, and Microsurfacing) showed that, on average, the OGFC mixes obtained the lowest tire/pavement generated noise levels. The two test sections containing asphalt rubber (AR-OGFC) had the lowest noise levels of all sections tested. The authors note that the AR-OGFC mixes had the finest gradation of the OGFC mixes, perhaps also added to the noise reducing properties.

Bennert et al, indicate that, in order of lowest to highest noise producing surface, the rankings were: 1) OGFC mixes, 2) 9.5mm SMA mix, 3) Micro-surfacing, 4) Novachip®, and 5) 12.5mm SMA mix. They mention that a recently (at the time of the study) placed 12.5mm Superpave mix had an unexpectedly low tire/pavement noise of 97.1 dB(A), although the older 12.5mm mixes were almost 2 dB(A) higher. The PCC diamond grind treatment showed tire/pavement noise characteristics comparable to those of the Micro-surfacing and older surfaces of 12.5mm Superpave and Novachip. The PCC pavements generated the largest tire/pavement noise, approximately 10 dB(A) higher than the OGFC mixes.

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Bennert et al. indicate that the noise data also showed a general increase in tire/pavement noise as the age of the surface type increases. For example, the five year older Novachip surface had a 1.2 dBA higher noise level than the younger section. Similar trends also occurred for the 12.5mm Superpave and the NJDOT MOGFC- 1 mixes. The authors note that the increase in tire/pavement noise may be attributed to both the natural aging/stiffening of the asphalt material, as well as some surface distress that may have been caused by excessive traffic levels.

Bennert et al. indicate that the measured noise gradients usually fell within 0.15 to 0.3 dB(A) per mph. The thin-lift material that achieved the lowest noise gradient was the Novachip® and the NJDOT MOGFC-2 mix. In general, the NJDOT MOGFC mixes had noise gradients ranging from 0.15 to 0.16 dB(A) per mph, with the AR-OGFC sections having noise gradients ranging between 0.19 and 0.20. The Micro-surfacing had the largest noise gradient of 0.3 dB(A) per mph with the non-treated PCC pavements yielding the second largest noise gradient. The diamond ground PCC surface had a noise gradient comparable to the HMA mixes.

Bennert et al. indicate that the PCC surface with no finish showed the lowest wet skid resistance, whereas the PCC surface with transverse tine and one of the AROGFC courses showed the highest wet skid resistance. The diamond ground PCC section had the third best wet skid resistance. The authors were surprised to see that the 12.5 mm Superpave mixes performed quite well under the wet skid resistance test, out-performing the more open/gap-graded mixes. Using a recommendation that roadway surfaces should have a minimum wet skid resistance ranging from 35 to 40, when using a ribbed test tire, to maintain vehicle safety in wet weather conditions, the authors note that only one of the PCC with no finish would be considered marginally acceptable.

Bennert et al indicate that pavement surface treatments that are optimal for selection should have both low tire/pavement noise and high wet skid resistance. Using this criterion, they note that best performing surface in the study was the asphalt rubber OGFC located on I-195 W, while the poorest performing surface was the PCC without treatment. The 12.5 mm Superpave mixes had the second best ranking based on having low tire/pavement noise and high wet skid resistance. Overall, when comparing only the thin-lift mixtures, the OGFC mixtures had the best performance with the Micro-surfacing being the second most optimal surface type. The authors indicate that the 12.5 mm nominal aggregate sized SMA mixed was the worst performing thin-lift surface treatment.

Regarding ride quality measurement results, Bennert et al. mentions that the of the 17 test sections evaluated for RQI, only 8 of the sections obtained a rating of Very Good. The remaining sections, except for the 2 transverse tined PCC sections, were rated as Good. They note that it was difficult to determine which surface type provided the “best” RQI rating since the age of the pavement surface, as well as the contractor’s quality control, may have influenced the results. As example, Bennert et al. points out that after 3 years of service, the Novachip® had a RQI rating of 4.47. However, after 8 years, the Novachip® had an RQI of 3.51. Overall, the 12.5mm Superpave mix and the diamond

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ground PCC had the best RQI ratings, with the OGFC mixes having the best RQI rating for the thin-lift HMA mixes.

Since the RQI is derived from the user's perception of ride quality, Bennert et al hypothesized that the tire/pavement noise may contribute to the user's rating of the pavement surface. With a plot of RQI versus tire/pavement noise at 60 mph, the authors show that a user's pavement roughness perception correlated well ($R^2 = 0.81$) to the tire/pavement noise generated during the user's ride.

Bennert et al. also provide the IRI data for the different sections. They note that the IRI values followed a trend which was similar to the trend in the RQI data. They mention that on average, the pavement surface that obtained the smoothest rating (lowest roughness from IRI) was the 12.5mm Superpave mix, with the diamond ground PCC having the second best IRI measurement. The thin-lift surface treatment that had the smoothest rating was the newer Novachip® surface, with the NJOGFC-1 mix being comparable. The roughest pavement surfaces measured in the study were the transverse tined PCC pavements.

Bennert et al. also point out a strong correlation exists between the pavement smoothness (or roughness) and the measured tire/pavement generated noise. They mention that this correlation is most likely because the pavement surface macro-texture that is being measured with the ARAN's lasers for IRI has a direct influence on the tire/pavement noise.

In making overall summary, Bennert et al. mention that the pavement surface type that performed the best under the noise, wet skid resistance, and ride quality testing (RQI and IRI) was the 12.5mm Superpave mix, with the diamond ground PCC performing the second best overall. The OGFC mixes performed comparably and was ranked third overall. The worst performing surface type in the study was the PCC with no surface treatment, which obtained on average the highest noise levels, the lowest wet skid resistance, and the poorest ride quality ratings for both the RQI and the IRI measurements.

However, in subsequent sections, Bennert et al provide cost information on the different mixes, and from the comparisons, the benefits of OGFC courses become more apparent. This is because, in both examples, the cost of OGFC was found to be significantly lower compared to the alternative courses. In the case of US 24, Union, where MOGFC and the diamond ground PCC performed equally well (overall), the cost of the MOGFC-1 was \$4.60/yd^2, whereas the cost of the Diamond Grind was estimated at \$6.00/yd2. The authors conclude that the MOGFC-1 provided the required reduction in noise and at only two-thirds the cost of the Diamond Grinding, without sacrificing ride quality. Similarly, in the I 195 section in Hamilton, where Novachip, MOGFC-2 and a 12.5mm Superpave mix were the possible alternatives, the MOGFC-2 provided comparable performance at two thirds the cost of the Novachip and one third the cost of the Superpave mix.

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1.61.8 Structural Design
No information on structural design is provided in this paper

1.61.9 Limitations
No information on limitation on use of PFC is provided in this paper

1.62 Brousseaud, Y. and F. Anfosso-Lédée. "Silvia Project Report: Review of Existing Low Noise Pavement Solutions in France." SILVIA-LCPC-011-01-WP4-310505. Sustainable Road Surfaces for Traffic Noise Control. European Commission. May 2005.

1.62.1 General
This report provides an overview of low noise pavement solutions in France. The report was written for the SILVIA (Sustainable Road Surfaces for Traffic Noise Control) project which is funded by the European Commission.

Mostly single layer porous asphalt is utilized in France; however, double-layer porous asphalt is sometimes used. Typically, porous asphalt will have a 6mm or 10mm maximum aggregate size gradation and be placed on the roadway 30 to 40mm in thickness.

1.62.2 Materials and Mix Design
In France, porous asphalt is divided into one of two classes. Class 1 porous asphalt is designed at 20 to 25 percent air voids while Class 2 is designed at 25 to 30 percent air voids. Typical porous asphalt has a 10mm maximum aggregate size gradation with a gap in the grading between 2 and 6mm. The material passing the 0.075mm sieve is generally about 3 to 4 percent of the aggregate fraction.

In most applications, a polymer-modified binder is used. Typical asphalt binder contents range from 4.6 to 5.0 percent. Acrylic, glass or cellulose fibers are added to reduce draindown potential.

Mix designs are conducted with a Gyratory Shear Compactor. A total of 25 gyrations are used.

1.62.3 Benefits
The primary benefit of porous asphalt discussed in this paper was the reduction of tire/pavement noise. Porous asphalt in France has typical noise levels of approximately 71 to 73 dB(A) when measured by the Statistical By-Pass method. For comparison, typical dense-graded HMA has a noise level of approximately 76 dB(A).

Skid resistance was briefly mentioned as a benefit. The authors indicated that speed has nearly no effect on the braking force coefficient.

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1.62.4 Construction Practices

The authors suggest that prior to placement of a porous asphalt layer, a water drainage study should be conducted on the existing layer. The water drainage study should entail evaluating the permeability and the grade and cross slope of the existing layer. Existing layers that are permeable or areas that will hold water should be avoided.

Because the porous layer and underlying layer are only in contact through the coarse aggregate of the porous asphalt, a good quality tack coat is needed. The authors recommended a tack coat with 400g/m^2 of residual binder.

1.62.5 Maintenance Practices

The primary problem with general maintenance is clogging of the porous asphalt void structure. The authors state that roadways with low traffic volumes (especially heavy trucks) can clog at an early age.

With respect to winter maintenance, the authors state that experience is the only true method of developing a winter maintenance program. Porous asphalt presents unique winter conditions because frost emerges at a higher ambient temperature as the surface temperature of porous asphalt is 1.5 to 2.0˚C lower than typical pavements. Also, typical de-icing practices are not as effective as porous asphalt as on typical dense-graded pavement surfaces.

1.62.6 Rehabilitation Practices

The authors mentioned, but did not elaborate, on three rehabilitation techniques: replacement with a new porous asphalt, overlaying (with or without seal coat), and recycling in-place or in a plant.

1.62.7 Performance

Permeability was mentioned as a performance measure for porous asphalt. Table 119 presents typical results from a permeability related test. No specifics are provided for the permeability related test; however, it appears similar to other devices used in Europe that simply determine the time required for a certain volume of water to infiltrate into a layer (expressed as cm/sec). Results of this test are reported as percolation speed. An interesting observation of Table 119 is that as the maximum aggregate size of the gradation increases (10mm compared to 6mm) the permeability increases (percolation speed increases).

Table 119: Typical Percolation Speed of Porous Asphalt in France

Type of Porous Asphalt	Class 1		Class 2	
	0/6mm	0/10mm	0/6mm	0/10mm
Percolation speed (cm/sec) (NF P 98-254-3)	0.6	0.8	0.9	1.2

1.62.8 Structural Design

The only information provided on structural design is that a typical thickness of porous asphalt is 30 to 40mm.

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1.62.9 Limitations
No specific limitations were given.

1.63 "Quiet Pavements: Lessons Learned from Europe". *Focus*. U.S. Department of Transportation. Federal Highway Administration. Washington, DC. April 2005.

1.63.1 General
This paper outlines the findings of a scanning tour team (comprised of representatives of the FHWA, state transportation agencies, private industry, and academia) that visited Denmark, Netherlands, France, Italy, and the United Kingdom to report on these countries experience with quiet pavements. Please note that this article refers to other quiet pavement types besides porous asphalt.

The tour team summarized that European countries were more comprehensive than the U.S. in addressing noise reduction. One of the three major noise reduction technologies being used in Europe was porous asphalt mixes. These are mainly used on rural roads and highways with moderate winter conditions.

France has used single layer porous asphalt that has reduced noise by 6 to 9 dB as compared with a dense-graded pavement. Meanwhile, Italy has found that porous asphalt can achieve a life of 80-90 percent of a dense-graded mix.

Other highlights from the tour team were: Quiet pavements in Europe cost 10 to 25 percent more than traditional pavements and federal regulations do not currently recognize quiet pavements as a noise mitigation strategy.

1.63.2 Benefits of Permeable Asphalt Mixtures
No benefits of permeable asphalt mixtures were given.

1.63.3 Materials and Design
No materials and designs were given.

1.63.4 Construction Practices
No. construction practices were given.

1.63.5 Maintenance Practices
No maintenance practices were given.

1.63.6 Rehabilitation Practices
No rehabilitation practices were given.

1.63.7 Performance
No performance guidelines were given.

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1.63.8 Structural Design
No structural design guidelines were given.

1.63.9 Limitations
No specific limitations were given.

1.64 Frick, K. "Evaluation of New Patching Material for Open-Graded Asphalt Concrete (OGAC) Wearing Courses." Technical Memorandum TM-UCB-PRC-2—5-9. California Department of Transportation. June 2005.

1.64.1 General
This Technical Memorandum provides an overview of an internal research project conducted by the California Department of Transportation to evaluate a commercial product for patching OGFC layers. The patching product was a combination of rapid-setting urethane polymer and a blend of hydrophobic polymer-coated open-graded aggregates. The product is mixed on the paving project using a 5-gallon bucket mixes. [Pictures within the document show a concrete mixer being used to mix the material.] Materials used in the mixing process are blended at ambient temperature. The urethane polymer is not a viscoelastic material like asphalt binder and takes about two hours to cure. Frick indicated that the process is easy and requires minimal equipment and training.

Aggregates used in the patching material were open-graded to allow water to pass through the patch. In order to create the open-grading, a large percentage of pea gravel was placed in the aggregate blend. Frick states that the inclusion of the rounded pea gravel was not a concern because the polymer used in the mix is not dependent on temperature.

1.64.2 Benefits of Permeable Asphalt Mixtures
Frick identifies the OGFC's ability to drain water from the pavement surface as a benefit of OGFC. Because of the ability to drain water from the pavement surface, safety to the traveling public is improved as the potential for hydroplaning and splash/spray is reduced.

1.64.3 Materials and Mix Design
No specifics on materials and mix design of PFC were provided.

1.64.4 Construction Practices
As stated previously, the patching material is mixed on-site. For the projects described in the Technical Memorandum, no specifics were provided on how the material was placed onto the roadway. However, based upon a photograph within the document, the material was simply spread by hand over the patched area.
1.64.5 Maintenance Practices
No specifics were provided on maintenance practices.

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1.64.6 Rehabilitation Practices
The subject of this Technical Memorandum is considered to provide an alternative for minor rehabilitation. The patching material was placed on two test sites. First, the material was placed over a dense-graded layer of HMA at the Pavement Research Center, located at the University of California Richmond Field Station. This first test site was used to assess workability and ease of application of the material. Additionally, the drainage characteristics of the placed patching material were evaluated.

For the first test site, the patching material was placed in a lift of approximately 40 mm over an area of 0.2 m^2. In order to simulate a pothole, a wooden frame was used to provide confinement. Frick stated that the patching material was easily mixed and placed and provided adequate drainage materials.

The second trial site was a highway where an area of distress was located. The patching material was spread full lane approximately 15 mm thick for several meters. Frick considered this test section to be severe because the thickness was less than desired and there was no confinement of the mix since it was placed full lane width. Results from this second trial site resulted in portions of the patching material raveling within a few months. No mention of whether an attempt was made to bond the patching material to the underlying layer.

1.64.7 Performance
Performance of the open-graded patching material was not acceptable as the material began to ravel within just a few months.

1.64.8 Structural Design
No specifics on inclusion within structural design were given.

1.64.9Limitations
No specific limitations were given.

1.65 Graf, B., Simond, E. "The Experience with Porous Asphalt in Canton Vaud." VSS Publication Strasse and Verkehr. Route et Trafic. April 2005.

1.65.1 General
The positive experience with porous asphalt within a 15 year period in Canton Vaud of Switzerland is discussed. Porous asphalt was first constructed in Canton Vaud in 1991. Currently, one third or 65km of the surfaces in the Canton are covered with porous asphalt. In addition the use of porous asphalt on bridges was first introduced in 1993. Currently 17 bridges are surfaced with it. The paper discusses noise reduction binder use and winter maintenance.

165.2 Benefits of Permeable Asphalt Mixtures
Simond and Graf point out that one square meter of porous asphalt with a thickness of 4cm has a void volume of ca. 9 liter of which 7 liters are interconnecting voids capable of draining 7liters of water.

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Main benefits:

- Reduction of rolling noise of vehicles
- Reduction in splash and spray
- No aquaplaning
- Reduction in reflection from head lights under rainy weather
- Reduction in stress level for the driver

Disadvantages

- Winter maintenance
- Reduction in mechanical strength
- Reduction in service life

1.65.3 Materials and Design

The structure of porous asphalt and dense graded asphalt from Switzerland is presented in Figure 24.

Figure 24: Interconnected Voids of Porous Asphalt

Seven test sections have been built in Switzerland in 1988/99 with one variable that of binder as follows:

- 4 sections with commercially available polymer-modified binder
- one with B80/100 with SBS polymer modified
- one with B55/70 and Trinidad additive
- one with special binder with rubber additive

A 300m dual layer section was constructed in 2000 with a lower layer of 5cm of porous asphalt with max grain size of 22mm and binder content of 3.9 percent and a upper layer of 2.5 cm with max grain size of 8mm and binder content of 5 percent. As a result of this construction the permeability increased by 25 to 30 percent and noise emission fell by

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1dB in comparison with the single layer porous asphalt. Binder draindown in the lower layer was noted in the cores and it was concluded that the binder content of the lower layer should be decreased. The dual layer is used in the median for its permeability

1.65.4 Construction Practices

Normal construction practices is recommended

1.65.5 Maintenance Practices

Winter maintenance practices:

- Timing of the application of salt very important as when the snow depth increases it is harder to solve the freezing problem
- More salt should be applied in the first application of the season
- Application of salt should be repeated regularly
- The effect of weather on the porous pavement should be closely watched

The winter maintenance of porous asphalt is somewhat more complicated than dense asphalt however not a single
Weather related accident has been reported on the porous asphalt in canton Vaud.

1.65.6 Rehabilitation Practices

Not addressed in this paper

1.65.7 Performance

Reduction in noise through the use of porous asphalt versus dense-graded asphalt at various locations is reported in Table 120. On the average a reduction of 6db is recorded.

Table 120: Reduction in Noise Levels at Various Locations

Installation date	Location	Reduction in noise emission after the installation of porous asphalt [dBA]
1991	Pertit	4.1 ... 6.2
1993	Morges	5.4 ... 8.6
1999	Lonay	6.2 ... 8.4
1999	Bex	4.5 ... 6.0

Long-term development of noise reduction was recorded for two sites as shown in Figure 25, indicating that at these sites the noise reduction capability of porous asphalt was maintained even after 9 years.

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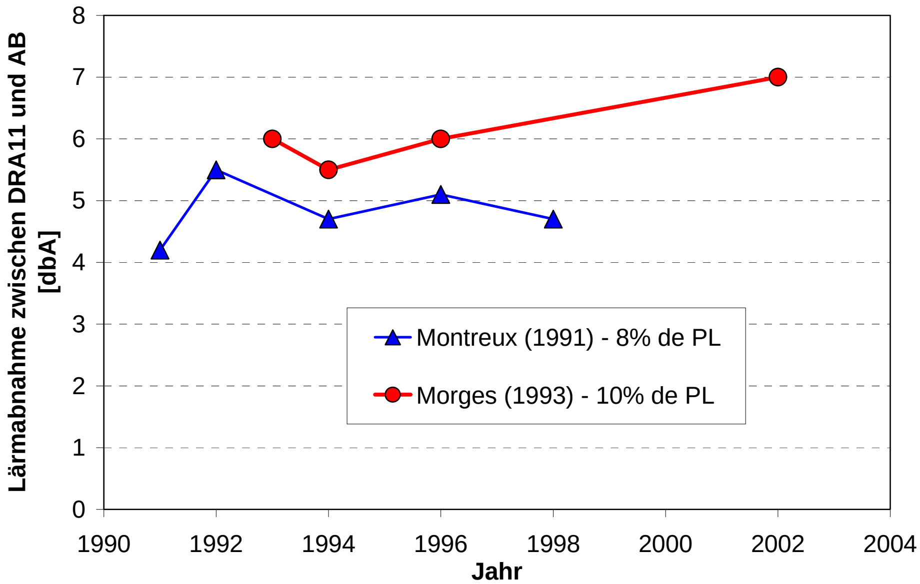

Figure 25: Change in Noise Levels with Time

The seven test sections performed well under an especially heavy snowfall in 1988/99 chains were used. Some raveling was experienced. The section with Trinidad showed the worst performance.

1.65.8 Structural Design
Not discussed.

1.65.9 Limitations
In accordance with Swiss standards porous asphalt is not used over 600 m elevation.

1.66 Hardiman, C. "The Improvement of Water Drainage Function and Abrasion Loss of Conventional Porous Asphalt." Proceedings of the Eastern Asia Society for Transportation Studies. Volume 5. pp. 671-678. 2005.

1.66.1 General
This paper describes a research effort designed to improve the permeability and durability of porous asphalt mixes. Hardiman evaluated porous asphalt mixes having maximum aggregate sizes of 10, 14 and 20mm.

1.66.2 Benefits of Permeable Asphalt Mixtures
Benefits listed in the paper included reduced hydroplaning, improved skid resistance at higher speeds, reduction in noise, reduction in glare at night and good resistance to permanent deformation.

1.66.3 Materials and Mix Design
Materials described by Hardiman included crushed aggregates, a combination of hydrated lime and cement as mineral filler and two asphalt binders, a penetration graded 60/70 and

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a polymer modified. Table 121 presents the design gradations for the 20, 14 and 10mm maximum aggregate size.

Table 121: Aggregate Grading Used in This Investigation

Sieve (mm)	Percent Passing by Weight Gradation 1	Gradation 2	Gradation 3
20	100	100	100
14	77.9	100	100
10	55.8	74.5	100
4.75	25	49	59.3
3.35	10	10.5	11.5
0.425	5.5	6	6.5
0.075	4	4	4

Hardiman evaluated both a single layer porous asphalt and double layer porous asphalt in this laboratory study. An interesting method of creating the laboratory specimens of the double layer porous asphalt was utilized. For the single layer porous asphalt specimens, 50 blows per face were applied to the sample. For the double layer porous asphalt, sufficient mix was placed in the Marshall mold to produce an approximately 50mm of the lower layer with 25 blows of the Marshall hammer. Then, the upper layer was placed on top of the lower layer and compacted with 25 blows in the same direction. Finally, the opposite face was compacted with 50 blows. In all cases, the 20mm maximum aggregate size gradation was used as the lower layer of porous asphalt in the double layer system. The 10 and 14mm maximum aggregate size mixes were used as the upper layer.

Testing of the various mixtures included permeability and Cantabro Abrasion testing. Results of the permeability testing showed that permeability increased as the maximum aggregate size of the gradation increased. Mixes with a 20mm maximum aggregate size were more permeable then the 14 or 10 mm maximum aggregate size mixes. Results of testing on the double layer system showed that permeability was controlled more by the top layer than the lower layer.

Results of the Cantabro Abrasion testing indicated that the smaller maximum aggregate mixes had lower abrasion loss values. This would indicate the smaller maximum aggregate size mixes are more durable.

1.66.4 Construction Practices

No specific construction practices were given.

1.66.5 Maintenance Practices

No specific construction practices were given.

1.66.6 Rehabilitation Practices

No specific rehabilitation practices were given.

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1.66.7 Performance
No specific performance measures were given.

1.66.8 Structural Design
No specifics on inclusion within structural design were given.

1.66.9 Limitations
No specific limitations were given.

1.67 Lane, R. "Cleaning Open-Grade Asphalt To Improve Safety." International Conference on Surface Friction/ 2005 Papers. www.surfacefriction.org.nz. Christchurch, New Zealand. 2005.

1.67.1 General
This paper discusses a trial maintenance program designed at cleaning the void structure of open-graded porous asphalt (OGPA). This work was conducted in New Zealand.

1.67.2 Benefits of Permeable Asphalt Mixtures
Lane provided several benefits of the OGPA mixtures used in New Zealand, namely, reduced hydroplaning potential, reduction in splash and spray, improved wet weather friction, improved reflectively of pavement markings and reduced traffic noise.

1.67.3 Materials and Mix Design
No specifics on materials and mix design were given.

1.67.4 Construction Practices
No specifics on construction practices were given.

1.67.5 Maintenance Practices
This paper primarily deals with a trial program to evaluate a method for cleaning debris from OGPA pavements. Though the method and equipment utilized were not discussed in detail, the author indicates the method as "low pressure captive blasting."

The OGPA Cleaning Trial was conducted jointly between the Transit New Zealand and Pavement Treatments, Ltd on a total of 13 sites. In order to evaluate/quantify the level of cleaning that took place, a pavement permeability test was utilized. Another aspect of the test program was to evaluate the point in an OGPA's life when maintenance is required. Of the 13 pavements cleaned and evaluated, they ranged in age between 1 and 6 years.

No raw data was provided to quantify the improvement in permeability of the OGPA layers; however, a figure within the paper did show the improvement in permeability after removing debris for the void structure. Figure 26 is a replication of the figure contained within the paper. Values of Figure 26 were scaled in order to create this figure.

Based on Figure 26, Lane indicated that the optimum timing for the first cleaning is between 2 to 3 years in age. Generally, the sections evaluated that had an age less the 3

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years still maintained an adequate level of permeability. Lane explained the method of measuring permeability of a pavement as a permeability index test. A certain volume of water was allowed to penetrate into the OGPA pavement and the time required for all of the water to penetrate the pavement reported as permeability. All of the sections showed a decrease in time (increase in permeability) after cleaning the pavement. For older pavements, the data also showed that cleaning a pavement a second time would again improve the drainability of an OGPA pavement.

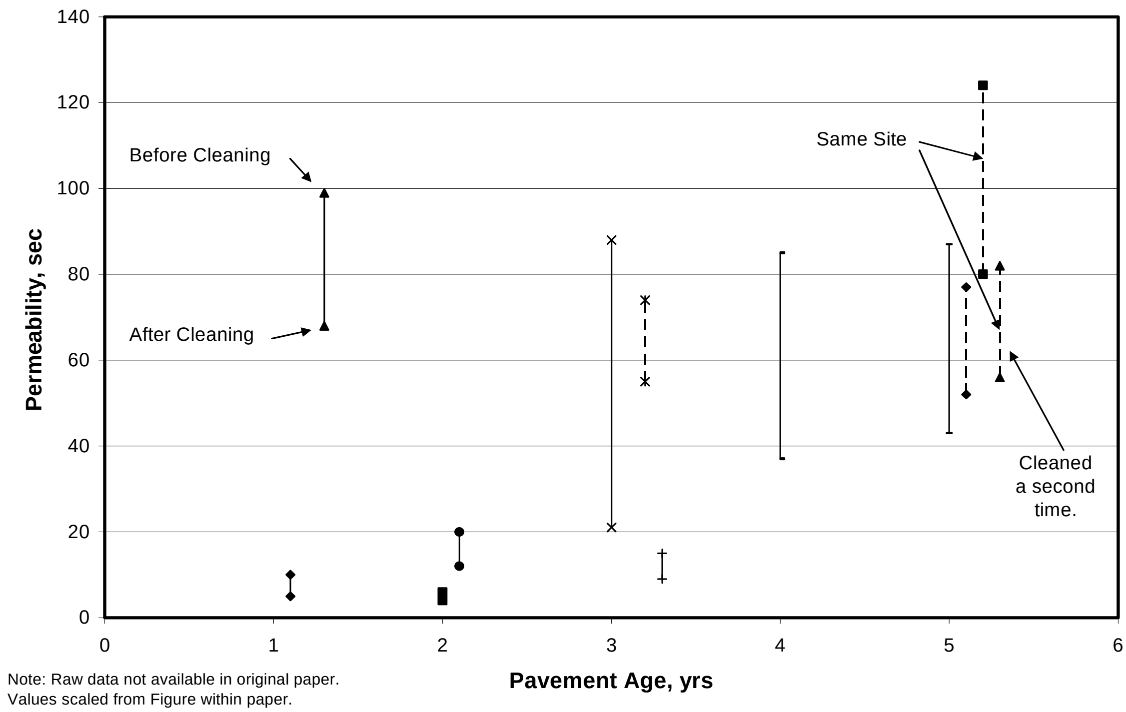

Figure 26: Effect of Cleaning OGPA

1.67.6 Rehabilitation Practices

No specifics on rehabilitation practices were given.

1.67.7 Performance

The performance measure utilized by Lane was permeability. Though not fully described within the paper, the test method seems similar to some used in Europe in that the index of permeability has units of time.

1.67.8 Structural Design

No specifics on inclusion within structural design were given.

1.67.9 Limitations

No specific limitations were given.

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1.68 McDaniel, R. "Case Study: A Porous Friction Course for Noise Control". *North Central Superpave Center News*. North Central Superpave Center. West Lafayette, Indiana. Volume 4, Number 3. Spring 2005.

1.68.1 General

This paper overviews the testing completed on a test section of Porous Friction Course (PFC) placed in Indiana in 2003. This test section was constructed concurrently with a section of SMA and conventional Superpave HMA. All three of these test sections were evaluated for pavement noise, friction and performance.

All there test section mixes were constructed with steel slag aggregate from the same source and a SBS- modified PG76-22. Also, all mixes were designed for a traffic level of 10-30 million ESALs. The gradation of the mixes were all different (No specific data was given, although a gradation curve was presented), and the PFC and SMA had cellulose fibers added.

First noise measurements were taken by the pass-by (CPB) and close proximity (CPX) methods. The results of the testing showed that the PFC mix had the lowest noise levels. The conventional Superpave mix had the next lowest noise levels (CPX test showed 3.6 dB higher than PFC, CPB was 4.2 dB higher than PFC), and the SMA had the largest noise levels (CPX test showed 4.8dB higher than PFC, CPB was 5.0 dB higher than PFC).

Next the three test sections were evaluated for friction, surface texture, and splash and spray. The PFC surface texture was visually inspected and evaluated to be more open than the SMA and conventional HMA. The difference in surface texture were confirmed and quantified with a Circular Texture Meter. The frictional properties of the PFC mix were found to be higher than the HMA and SMA as quantified by the International Friction Index. Splash and Spray was only visually observed. Based on these observations, the PFC was significantly better at reducing water on the surface and increasing visibility.

Overall, McDaniel concluded that, "PFC may offer an effective and economical way to reduce noise while maintaining, or even improving, friction and visibility."

1.68.2 Benefits of Permeable Asphalt Mixtures

McDaniel mentioned that the PFC greatly reduced splash and spray as compared with the SMA mix.

1.68.3 Materials and Design

McDaniel did not discuss materials and design.

1.68.4 Construction Practices

McDaniel did not discuss construction practices.

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1.68.5 Maintenance Practices
McDaniel did not discuss maintenance practices.

1.68.6 Rehabilitation Practices
McDaniel did not discuss rehabilitation practices.

1.68.7 Performance
McDaniel did not discuss performance of permeable asphalt mixes.

1.68.8 Structural Design
McDaniel did not discuss structural design.

1.68.9 Limitations
McDaniel did not discuss limitations.

1.69 McDaniel, R. S. and W. Thornton. "Field Evaluation of a Porous Friction Course for Noise Control." TRB 2005 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2005.

1.69.1 General
This paper describes the design, construction and test performance of a porous friction course (PFC) placed on I-74, east of Indianapolis, in 2003. McDaniel and Thornton indicate that this section of PFC was placed adjacent to and compared against a stone matrix asphalt (SMA) and a Superpave HMA section.

The authors indicate that the PFC showed significantly better (early life) performance in terms of noise reduction, increasing surface texture and friction, and reducing splash and spray. They conclude that PFC provides an economical way to enhance different performance related properties of HMA.

McDaniel and Thornton indicate that the Indiana DOT had experienced clogging problems with OGFC in the past, and that it remains to be seen whether the new PFC, with its higher void content, would exhibit better (long term) performance. Although the initial performance of PFC seems to be promising, they suggest the use of long term performance evaluation for making decisions regarding its use in future.

1.69.2 Benefits of Permeable Asphalt Mixtures
McDaniel and Thornton indicate a number of benefits of PFC in their literature review. Based on mostly reports on the European experience, the authors mention that PFCs have a significantly higher void content compared to OGFCs (18-22 percent compared to 10-15 percent), and that PFCs have proven to be durable, to possess good surface friction and to decrease splash and spray during rain. They indicate that aggregate quality and gradation provide enhanced microtexture and macrotexture in PFCs. They also mention that the microtexture, which is the fine scale texture of the aggregates, influence frictional

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properties at low speeds, and that the macrotexture influences drainage of water and affects rate of decrease in frictional properties with increasing speeds.

1.69.3 Materials and Design

McDaniel and Thornton indicate that the PFC used in this study was made up of primarily steel slag aggregates, and consisted of SBS modified asphalt binder and cellulose fiber. The PFC mix was designed with a target air void content of 18-22 percent, using 20 gyrations of the Superpave gyratory compactor.

After design, the PFC samples at optimum asphalt content were subjected to the Cantabro test for evaluation of resistance against abrasion. In this test, unaged and aged samples compacted with the Marshall hammer were subjected to 300 revolutions in a Los Angeles abrasion machine. The mass loss during this process is determined on the basis of percentage of original mass of the specimen. McDaniel and Thornton indicate that the allowable mass loss of unaged and aged samples are taken as 25 percent and 30 percent, respectively, by some European and South African specifications.
The specific results of mix design and Cantabro test for the PFC are shown in Table 122. The properties of SMA and HMA are also shown in Table 122.

Table 122: Materials, Mix Design and Test Results

Materials/Properties	**Mix**					
	PFC		SMA		HMA	
Design Traffic	---		10-30 million ESALs		10-30 million ESALs	
Aggregates	90 % steel slag, 10 % sand		80 % steel slag, 10 % stone sand, 10 % mineral filler		Coarse aggregate – 50 % steel slag and 50 % dolomite; dolomitic manufactured sand	
Gradation	Sieve Size, mm	Percent Passing	Sieve Size, mm	Percent Passing	Sieve Size, mm	Percent Passing
	12.5	100	12.5	100	12.5	100
	9.5	83	9.5	85	9.5	94
	4.75	28	4.75	39	4.75	64
	2.36	12	2.36	27	2.36	46
	1.18	9	1.18	21	1.18	---
	0.6	6	0.6	18	0.6	17
	0.3	5	0.3	15	0.3	---
	0.15	3	0.15	13	0.15	---
	0.075	2.4	0.075	10.1	0.075	5.5
Asphalt Type	SBS modified PG 76-22		SBS modified PG 76-22		PG 76-22 (source different from those used for SMA and PFC)	
Asphalt Content	5.7		5.5		5.7	
Fiber	Cellulose fiber, 0.3 %		Cellulose fiber, 0.1 %		---	
Gyration	20		100		100	
Voids, %	23.1		4.0		4.0	
VMA, %	---		17.7		15.5	
Cantabro Loss, unaged	15		---		---	
Cantabro Loss, aged	24.9		---		---	

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1.69.4 Construction Practices

McDaniel and Thornton indicate that the PFC mix was constructed in August 2003, using a material transfer device (MTV), which is commonly used in HMA construction in Indiana. One pass from each of two steel wheel rollers was found to be sufficient for compaction. No problem was reported during construction. The authors caution that over rolling can lead to aggregate breakdown and mention that relatively little compactive effort should be sufficient to bring the coarse aggregate in contact for proper construction of PFC.

McDaniel and Thornton mention that similar rollers and roller passes were used for construction of the SMA section. No information is provided on the construction of the HMA section, but the authors indicate that there was no problem reported during the construction of the HMA section.

1.69.5 Maintenance Practices

No information has been provided regarding maintenance practices.

1.69.6 Rehabilitation Practices

No information has been provided regarding rehabilitation practices

1.69.7 Performance

McDaniel and Thornton provide information on performance of the PFC section, in terms of results of different tests conducted to evaluate noise, surface texture and friction and splash and spray.

For all the different types of results, the PFC showed lower noise than the SMA and the HMA. The different results are shown in Table 123. McDaniel and Thornton show in a figure that the overall sound pressure level is the lowest for the PFC. They explain the effect of reduction of noise with an example. For a line source of noise (a busy roadway is approximated as a line source), noise is attenuated by 3 dB per doubling of noise from the source. A 3dB reduction in noise source level will lead to a 50% decrease in the distance at which a given noise level is measured relative to the original sound. For

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Table 123: Results of All Sound Measurements

Method	Speed		Average CPX Sound Pressure Levels (Time averaged level over the length of pavement, LAEQ)		
			PFC	SMA	HMA
CPX	72 kph		89.7 dBA	94.2 dBA	93.0 dBA
	97 kph		92.6 dBA	97.6 dBA	96.4 dBA
	Average		91.2 dBA	95.9 dBA	94.7 dBA
	Difference from PFC		0.0 dBA	4.7 dBA	3.5 dBA
Pass-By	Speed	Vehicle			
	80 kph	Impala	68.1 dBA	74.8 dBA	72.6 dBA
		Volvo	70.1 dBA	75.5 dBA	75.2 dBA
		Silverado	71.6 dBA	77.0 dBA	74.5 dBA
		Average	69.9 dBA	75.8 dBA	74.1 dBA
	110 kph	Impala	71.7 dBA	78.5 dBA	NA*
		Volvo	74.3 dBA	80.5 dBA	NA
		Silverado	74.4 dBA	79.4 dBA	NA
		Average	73.5 dBA	79.5 dBA	NA
	Difference from PFC		0.0 dBA	6.0 dBA	---

* Could not be tested due to speed limits

Results of surface texture measurements are shown in Table 124. McDaniel and Thornton indicate that the PFC and SMA showed significantly more texture than the HMA, with the PFC showing the maximum depth. The variability in measured values was also found to be higher for the PFC and the SMA. The authors indicate that this is expected since these mixes have gap-graded aggregate structures, and a lower texture is expected in the SMA due to the presence of the mastic of asphalt binder and fibers.

Table 124: Results of Surface Texture Measurement

Mix	Mean profile Depth, mm (Standard Deviation)
PFC	1.37 (0.13)
SMA	1.17 (0.14)
HMA	0.30 (0.05)

The results of dynamic friction measurement are presented in Table 125. McDaniel and Thornton indicate that for the DFT measurements, the PFC and the HMA show comparable values whereas the SMA showed the lowest values. The authors point out that the SMA is expected to have lower values because of the presence of the mastic of asphalt binder and filler in the gap-graded aggregate structure and that these values should increase the traffic wears off the asphalt binder film and exposes the steel slag aggregate.

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Table 125: Results of Friction Measurement

Mix	Average Dynamic Friction Tester (DFT) Number (Standard Deviation) 20 kph	40 kph	60 kph	International Friction Index (F_{60})
PFC	0.51 (0.03)	0.45 (0.03)	0.42 (0.03)	0.36
SMA	0.37 (0.01)	0.31 (0.01)	0.29 (0.01)	0.28
HMA	0.52 (0.01)	0.47 (0.01)	0.44 (0.01)	0.19

In terms of the IFI measurements, the PFC showed the highest friction, followed by the SMA and the HMA. Since these values are calculated from the mean profile depth, the trend is similar to the trend of the mean profile depth.

Since no quantitative measurement was made for splash and spray, no results are given. However, based on visual evaluation, the authors indicate that sight condition for the driver was improved significantly (even when passing or passed by semi trailer trucks) with the use of the PFC, as compared to the SMA section. McDaniel and Thornton illustrate the better draining capabilities of the PFC section with the help of a figure showing a close-up view of the PFC and the HMA section, side by side.

1.69.8 Structural Design

No information on structural design is given in this paper.

1.69.9 Limitations

No information on limitations of the use of the PFC is given in this paper.

1.70 Scofield, L. and P. Donavan. "The Road To Quiet Neighborhoods In Arizona." TRB 2005 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D.C. 2005.

1.70.1 General

Scofield and Donavan describe the initial results of a ten-year $2 million program of research on producing quieter pavements in Arizona. The scope of this work to date consists of the determination of noise levels in existing roadway network, use of different types of tinings in new concrete pavements, the use of Asphalt Rubber Friction Course (ARFC), and the determination of noise levels in the new pavements using different techniques.

The authors describe the background of the quieter pavement work in Arizona, and the resultant research program, which has been implemented through the use of different experimental pavements and tinings in the metropolitan Phoenix area. They describe the two main techniques used for determination of noise levels – the close proximity (CPX) method (using a trailer) and the sound intensity (SI) techniques (which can be used without a trailer). The results from these techniques compared well with each other, and were used for determination of noise levels in not only ARFC layers over concrete pavements, but also new concrete pavements tined in different ways.

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Scofield and Donavan indicate that the 25 mm (one inch) ARFC over concrete pavements did produce the lowest noise levels, about 5 dB(A) less than random transverse tined concrete pavement (which produced the highest noise level). The ARFC was also found to have reduced the noise level by 5dB(A), on an average, at neighborhood residences.

1.70.2 Benefits of Permeable Asphalt Mixtures

No benefit of permeable asphalt mixtures have been mentioned separately in this paper.

1.70.3 Materials and Design

No information on materials and design has been provided in this paper.

1.70.4 Construction Practices

Scofield and Donavan mention that traditionally 12.5 mm thick ARFCs have been placed over HMA pavements in Arizona, and as part of this new research program on producing quieter pavements, 25 mm thick ARFCs are being placed over concrete pavements. No other details of construction have been provided.

Scofield and Donavan indicate techniques used for lowering of noise levels in existing pavement. These techniques include uniform longitudinal tinings, random transverse tinings and different diamond grinding techniques using different spacing between grinding blades, and difference in the amount of head pressure and beam length used in constructing the sections.

1.70.5 Maintenance Practices

No information on maintenance practices has been provided in this paper.

1.70.6 Rehabilitation Practices

Scofield and Donavan do not mention any specific rehabilitation activities to improve the condition of existing pavements

1.70.7 Performance

Performance of the different types of surfaces, in terms of results from noise testing studies, has been provided in this paper. The authors give a detailed description of the different types of tests, data analysis and the amounts of noise reduction obtained from the different pavements.

The authors present the results of their study in three different sections – development of a near field noise measurement system, determination of noise levels in existing pavements of different types but of similar ages, and the determination of noise levels in new concrete pavements with open surface and those with ARFC surfaces.

Scofield and Donavan indicate that the AZ DOT have used both the CPX and the SI method (for near field noise measurement) and have decided to adopt the SI system because of its ability to measure noise without using a trailer. The authors describe the studies conducted to compare the results from these two test methods.

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Initially, the CPX and SI methods were compared by conducting tests with a Goodyear Aqua Tread tire, at 60 mph on three roadway locations - SR 138 in California and sections of I-8 and I-10 in Arizona. The SI system was mounted on the driver side trailer wheel (in the CPX trailer) and leaving the CPX system mounted on the passenger side of the trailer. Five hundred foot long pavement segments were tested at each location. Data was reduced and averaged in terms of overall A-weighted level and 1/3 octave band level. Scofield and Donavan indicate that there was a good correlation between the two systems even though they measured different wheel path locations. In addition to obtaining A-weighted values, spectral comparisons of CPX and SI data were also made, and the results indicated a slight spectral shift between the CPX and SI results, particularly below a frequency of approximately 1600 Hz.

Scofield and Donavan mention that to overcome the added time required to use the original SI system (made by GM, which required making two passes with the vehicle), I&R has developed a modified design which would allow both leading and trailing edge contact patch areas to be measured concurrently with one pass of the trailer. Although this would violate the draft ISO standards for close proximity measurements, Arizona DOT is working on developing a trailer which can be used for both CPX and SI measurement, to increase the speed and reduce the variability of data collection.

Scofield and Donavan then provide results of field and laboratory tests conducted to independently evaluate the trailer modifications. From the field testing (which used a noise generator, a microphone and speaker), the authors conclude that most of the differences are due to experimental variations. In the laboratory testing, where the effect of isolation through an enclosure was evaluated, a 10dB(A) expected difference between tests conducted with open and closed door were not obtained, and the authors caution against the use of this system. In the third testing, the isolation of side-to-side tire noise when using two different tire types on the same axle, was evaluated, both in the field and in the laboratory. The laboratory testing consisted of mounting a speaker in place of the tire at one position and replacing the tire with a microphone at the other position. The results of this testing indicated the minimum 10 dB(A) separation requirement was met. To further verify the isolation of the side-to-side tire noise, on-road tests were conducted using an ASTM friction tire and a Goodyear Wrangler SUV type tire. The baseline testing was conducted using the friction tire on both sides. The far side Friction tire was then replaced with the Wrangler SUV type tire. The results indicated the 10 dB(A) requirement had been met.

Scofield and Donavan indicate that three to twelve year old pavements constructed in similar environments as Phoenix were identified and tested using both the CPX and SI systems. These pavements had 12.5 mm thick ARFC surfaces over HMA pavements. No significant correlation between pavement age and overall noise level was found. The authors indicate that due to equipment problems, some of the older and quieter pavements did not get tested.

Scofield and Donavan indicate that testing was conducted on existing concrete pavements prior to overlay with ARFC. Both longitudinal and random transverse tinings were

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present in different sections of SR202. These sections were tested with controlled pass-by technique. For each of the roadside test locations, 32 vehicles representing three classes of vehicles were driven past each of the three points at approximately one-minute intervals for the passenger vehicles and two minute intervals for the medium and heavy trucks. Each of the passenger cars were driven by at 60 MPH and then again at 70 MPH Measurements were obtained at 25 ft and 50 ft from the centerline of the travel lane at all three-test locations.

Scofield and Donavan mention that the results indicated that the uniform longitudinal texture produced approximately a 5 dB(A) reduction over ADOT's standard uniform one inch transverse texture. It also produced approximately an 8-9 dB(A) reduction over the Wisconsin random transverse texturing. All three tining methods resulted in approximately a 2 dB(A) increase between 60 and 70 MPH at the 50 ft measurement location. Inside the test vehicle, it was found that the random traverse tined surface produced levels higher than either of the two other surfaces between about 200 and 800 hertz apparently due to the details of the tine spacing. The authors mention that the experience with random transverse tining of AZ DOT is different from Wisconsin DOT, as the tinings were of different nature – AZ DOT tinings had an upward (positive) tine of 5/32inch, whereas Wisconsin tines had typical depths of 2/32 to 3/32 inch.

In addition to existing concrete section, Scofield and Donavan mention the testing of new concrete pavements, which had several different types of grindings. The four grinding techniques are essentially based upon altering the spacing between grinding blades, and in the amount of head pressure and beam length used in constructing the sections. Testing was conducted using the CPX method and using a Goodyear Aqua Tread tire at 60 mph. The noise levels in these experimental sections were found to be significantly lower than in pavements constructed with other concrete texturing methods.

Scofield and Donavan point out that the current procedure of loudness measurement using the 1/3 octave analysis with A-weighting does not consider the tonal properties (spikes) that may exist. In a figure showing results of analysis conducted in two ways – using 1/3 octave and 1/24 octave, the authors illustrate that in the 1/3 octave spectrum, a tonal spike at approximately 1500-1600 Hz is not apparent, whereas in the other one, the tonal spike is readily apparent. The authors mention that this is important, since this tonal spike represents the "tire whine" associated with uniformly-tined PCCP and represents an important part of the annoyance factor. They illustrate that using 1/24 octave the benefit of using ARFC is more apparent - there is also a significant reduction in the spectrum in the range between 1000 and 2000hz, a range where hearing is quite sensitive.

Overall, Scofield and Donavan mention that the ARFC produced the lowest noise level (91.8 dB(A)), followed by the Whisper Grinding (95.5 dB(A), the longitudinally tined PCCP (99.1 dB(A)), ADOT uniformly transverse tined PCC)P (102.5 dB(A)), and the random transverse tined PCCP (104.5 dB(A)). The ARFC was found to have reduced the noise level at residential neighborhoods by 5dB(A). The authors mention that the 25 mm thick ARFC layers do produce quieter pavements compared to the traditionally used 12.5

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mm thick ARFCs. They are not sure whether this is due to the fact that the 25 mm thick ARFCs are on new pavements, while the 12.5 mm ARFCs are on old pavements.

1.70.8 Structural Design
Scofield and Donavan do not provide any information on structural design.

1.70.9 Limitations
Scofield and Donavan do not provide any information on limitations.

1.71 Sholar, G. A., G. C. Page, J. A. Musselman, P. B. Upshaw and H. L. Moseley. "Development of the Florida Department of Transportation's Percent Within Limits Hot-Mix Asphalt Specification." TRB 2005 Annual Meeting CD-ROM. Transportation Research Board. National Research Council, Washington, D.C. 2005.

1.71.1 General
Sholar et al. provides a description of test data and methods used for the development for a percent within limits (PWL) specification for construction quality control of dense-graded hot mix asphalt HMA and open graded friction courses (OGFC) in Florida. The acceptance and payment for OGFCs are based on asphalt binder content and percent passing certain critical sieve sizes.

Sholar et al mentions that data from recently constructed projects were used to develop the tolerances in this specification, and a system has also been developed for acceptance and payment of small quantity lots. They mention that a system has been developed to use independent sampling and statistical tests for acceptance and payment.

Note that since this paper is focused on quality control and not specifically on permeable friction courses (PFC), only the portion relevant to PFC is reviewed and presented in the following paragraphs.

1.71.2 Benefits of Permeable Asphalt Mixtures
No information on benefits of PFCs is given.

1.71.3 Materials and Design
No information on materials and design of PFCs is given.

1.71.4 Construction Practices
No specific information on construction practices is given. However, since the development of PWL and the data related to variability in asphalt binder content and gradations are related to construction of PFC, these are reviewed and presented below.

Sholar et al mention that the development of the PWL system for open-graded friction course mixtures was done in the same way as done for dense-graded Superpave mixtures. However, the material properties used for payment, standard deviations and specification limits are unique for OGFCs.

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Sholar et al indicate that the Florida Department of Transportation (FDOT) selected four asphalt material properties that were believed to relate most closely to the performance of open-graded friction course mixtures. For each of these properties, proper weightage was determined on the basis of engineering judgment to determine the relative contribution to the overall quality of OGFC. The four properties and the relative ratings are: 1) asphalt binder content (40%), 2) percent passing the 3/8 inch sieve (20%), 3) percent passing the No. 4 sieve (30%), and 4) percent passing the No. 8 sieve (10%).

Sholar et al indicate that FDOT considered a sublot for OGFC mixes as 500 tons of asphalt mixture, with four sublots making up one lot of 2000 tons. They mention that the PWL system is utilized only for LOTs that contain three or more sublots. Production resulting in less than three sublots is treated separately as “small production”. The frequency of test for each asphalt mix property is one random test per sublot.

Sholar et al provide a table with standard deviations of the critical OGFC mix properties on the basis of quality control data obtained from 5 projects, constructed by 7 contractors. Table 126 shows the variance and standard deviation of the different properties.

Table 126: Variability of Test data from OGFC Projects (25 projects, 319 data points)

Property	Variance	Standard Deviation
Asphalt Binder Content, %	0.057	0.0238
Percent passing the 3/8 inch sieve	8.950	2.992
Percent passing the No. 4 sieve	4.422	2.103
Percent passing the No. 8 sieve	1.090	1.044

Sholar et al indicate that the FDOT used the method presented in the AASHTO Quality Assurance Guide Specification and NCHRP Report 447 to develop the specification limits for each asphalt mix. They mention that in this method, a 100 percent payment to the contractor is made if 90 percent of the test results are within the upper and lower specification test limits. Therefore, the specification limits must be established such that if a Contractor’s test data has representative variability and the mean of the test results is equal to the target, then 100 percent payment can be achieved. Sholar et al indicates that the standard deviations shown in Table 126 were multiplied by 1.645, since in a normal distribution 90 percent of the data is expected to lie between ± 1.645(standard deviation) from the mean value.

In the derivation of the specification limits, Sholar et al mention that the median instead of the average standard deviation values were used. This was done in view of the fact that in the distribution of variance values, the median variance and the average variance were not found to be equal, and the DOT believed that the median value was more representative than the average value.

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Sholar et al presents the derived specification limits (shown in Table 127), and mentions that the mathematically derived specification limit values were increased (and adopted, as shown in the column "Implemented Specification Limits") on the basis of discussion with construction industry representatives, FHWA representatives and other Department personnel.

Table 127: Specification Limits for OGFC

Property	Median Standard Deviation	Calculated specification Limits (standard deviation * 1.645)	Implemented specification limits
Asphalt Binder Content, %	0.24	0.39	±0.45
Percent passing the 3/8 inch sieve	2.99	4.92	±6.00
Percent passing the No. 4 sieve	2.10	3.46	±4.50
Percent passing the No. 8 sieve	1.04	1.72	±2.50

Sholar et al indicate that for calculation of payment, the FDOT used the method presented in the AASHTO Quality Assurance Guide Specification. They indicate that for each asphalt material property, the following equation is used to determine the pay factor: Pay factor (%) = 55 + 0.5 x PWL, where: PWL = percent within limits
The authors mention that with this system, the maximum pay factor that can be achieved for 100 percent of test results within the specification limits is 105, i.e. a five percent bonus in payment. They also mention that FDOT has specifications that have clauses for dealing with low pay factor material.

Sholar et al mention that after the pay factor has been determined for each asphalt material property, a composite pay factor (CPF, with a maximum of 105) is calculated by multiplying the respective weights for each material property by the individual pay factors. There are also specifications to protect against low pay factor material.

The authors mention that there are three ways for a Contractor to increase the percent within limits to increase the pay factor: 1) reduce the difference between the mean of the test data and the established target value, 2) reduce the variability of the test results, or 3) a combination of one and two.

Sholar et al indicate that FDOT has developed small quantity pay tables that are similar in principle to the pay tables used prior to the development of the PWL system. Payment is based on the deviation of a test result or average deviation of two test results from the target value. Larger deviations from the target result in lower pay factors. No provision has been made for bonus in the small quantity pay table, since the intent of the

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specification is to use the PWL concept as often as possible and to discourage the production of small LOTs resulting in only one or two test results.

Sholar et al mention that the derivation of the values contained in the small quantity pay tables is based partially on the specification limits used in the PWL system and partially on negotiations with industry. Even though very little difference was found between variability of properties between large and small productions, provision for larger variability was kept in the table for small production in view of the contractors' concerns that the variability for small quantities would be difficult to control. The pay table for open-graded friction course mixtures is shown in Table 128.

Table 128: Pay Table for Small Production

Property	Pay Factor	1-Test Deviation	2-Test Average Deviation
Asphalt Binder Content, %	1.00	0.00-0.50	0.00-0.35
	0.90	0.51-0.60	0.36-0.42
	0.80	>0.60	>0.42
Percent passing the 3/8 inch sieve	1.00	0.00-6.50	0.00-4.60
	0.90	6.51-7.50	4.61-5.30
	0.80	>7.50	>5.30
Percent passing the No. 4 sieve	1.00	0.00-5.00	0.00-3.54
	0.90	5.01-6.00	3.55-4.24
	0.80	>6.00	>4.24
Percent passing the No. 8 sieve	1.00	0.00-3.00	0.00-2.12
	0.90	3.01-3.50	2.13-2.47
	0.80	>3.50	>2.47

Notes:
(1) Each density test result is the average of five cores.
(2) In the event that the density of a LOT is less than 93.00% of Gmm, the Department will assess the pavement's permeability in accordance with FM 5-565. If the coefficient of permeability is greater than or equal to 125 x 10^{-5} cm/s, the Engineer may require removal and replacement at no cost or may accept the pavement at 90% pay. The Contractor may remove and replace at no cost to the Department at any time.
(3) If approved by the Engineer, based on an engineering determination that the material is acceptable to remain in place, the Contractor may accept the indicated partial pay. Otherwise, the Department will require removal and replacement at no cost. The Contractor may remove and replace at no cost to the Department at any time.

Sholar et al provide information on the verification procedure used by FDOT. This procedure is necessary since the contractor's quality control test data is used in the calculation of the payment amount for a given lot of hot-mix asphalt material. For verification, a DOT representative directs the contractor to obtain three split samples at a randomly selected point within the production of each sublot. The three split samples are required for the contractor, for verification testing and for resolution testing. The contractor performs tests on each sublot split sample. The verification technician

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performs tests on one randomly selected split sample per lot. For both the contractor and verification technician, tests are conducted to determine the following six asphalt material properties: 1) maximum specific gravity of the asphalt mixture, 2) bulk specific gravity of the gyratory compacted asphalt specimen at the plant, 3) percent asphalt binder content, 4) percent passing the No. 8 sieve, 5) percent passing the No. 200 sieve, and 6) bulk specific gravity of roadway cores. The contractor and verification test results are required to meet between-laboratory precision values established in FDOT test methods.

Sholar et al mentions that if all of the test results compare well, then the lot will be accepted with payment calculated based on the contractor's test data. If any of the test results do not compare favorably, the verification split samples from the remaining sublots of the lot will be tested only for the asphalt property(s) that did not compare favorably. A comparison will then be made between all of the contractor and verification test results for the asphalt property(s) that did not compare favorably. If there is only one unfavorable comparison, then the lot will be accepted and payment will be based on the contractor's test results. If there are two or more unfavorable comparisons, then a laboratory identified by the Department will test all of the resolution samples for the asphalt property(s) that did not compare favorably. If all of the resolution test results compare favorably with the contractor's test results, then the lot will be accepted with payment calculated based on the contractor's test data. If any of the resolution test results do not compare favorably with the contractor's test results, then the lot will be accepted with payment calculated based on the resolution test results.

Sholar et al indicates that the Florida DOT has developed an Excel spreadsheet (available on the DOT website) that performs all of the required calculations and pay factor determinations. It also generates random numbers to identify sampling points during production. The spreadsheet is password protected to prevent manipulation of tolerances and formula.

1.71.5 Maintenance Practices

No information on maintenance practices is provided in this paper**.**

1.71.6 Rehabilitation Practices

No information on rehabilitation practices is provided in this paper

1.71.7 Performance

No information on performance is provided in this paper

1.71.8 Structural Design

No information on structural design is provided in this paper

1.71.9 Limitations

No information on limitation on use of PFC is provided in this paper

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1.72 Van Doorn, R. "Winter Maintenance in the Netherlands." Ministry of Transportation. Public Works and Water Management. Compiled from COST344 Snow and Ice Control on European Roads and Bridges Task Group 3. Best Practices. March 2002.

1.72.1 General

This report provides information on the current winter maintenance activities for pavements in the Netherlands. Because of the influence of the North Sea, the climatic conditions of the Netherlands are more tempered than other Scandinavian countries. Extremely high or low temperatures are generally not experienced. Some general statistics for the Netherlands are provided in Table 129.

Table 129: General Climatic Statistics for the Netherlands

Amount of rain per year	760 mm (30 in.)
Number of days below 0˚C per year	60 days
Number of days with snowfall per year	30 days
Number of days with freezing rain per year	1 day

1.72.2 Benefits of Permeable Asphalt Mixtures

No specific benefits were given.

1.72.3 Materials and Mix Design

No specifics on materials and mix design were given.

1.72.4 Construction Practices

No specifics on construction practices were given.

1.72.5 Maintenance Practices

Table 130 presents the recommended average rate for spreading road salt in various winter climatic conditions from the Netherlands. This table suggests that more road salt is required for porous asphalt.

Table 130: Recommend Spreading Rates

	Amount of NaCl (g/m^2)	
Condition	dry salt	prewetted salt
Preventative (before a problem exists)	-	7 (1)
Fog Moisture	10	7
Icing	15-20	7-10
Glazed Frost (2)	20	15
Snow (after removal with plows) (3)	20	-

(1) On porous asphalt 14 g/m^2 is used (two application of 7 g/m^2)

(2) When the glazed frost situation stays for several hours, 20-40 g/m^2 dry salt should be used.

(3) Precautionary treatment: 15-20 g/m^2 pre-wetted salt.

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Van Doorn indicates that too much salt placed on porous asphalt under dry conditions can lead to slipperiness. Also, when temperatures drop below -15˚C, the salt can freeze. In these instances, calcium chloride, CaCl, is sprayed on the pavement surface.

1.72.6 Rehabilitation Practices
No specific rehabilitation practices were given.

1.72.7 Performance
No specific performance measures were given.

1.72.8 Structural Design
No specifics on inclusion within structural design were given.

1.72.9 Limitation
No specific limitations were given.

1.73 Wagner, C. and Y.S. Kim. "Construction of a Safe Pavement Edge: Minimizing the Effects of Shoulder Dropoff." TRB 2005 Annual Meeting CD-ROM. Transportation Research Board. National Research Council. Washington, D. C. 2005.

1.73.1 General
This paper presents the results of a study conducted to evaluate the constructability of a safety edge along the pavement edge. [This paper does not specifically deal with porous friction courses; however, because PFCs are daylighted at the pavement edge, the use of a safety edge may have benefit. In order to minimize the vertical pavement edge dropoff, agencies have limited the thickness of PFC layers. An effective pavement edge that provides safety to the traveling public, while not degrading the drainage properties of the PFC pavement, would be a benefit. Such a pavement edge would likely allow agencies to apply thicker layers of PFC, if so desired.]

The authors indicate that vertical pavement edge dropoff is a significant safety hazard. An errant vehicle that has left the roadway at the pavement edge can have difficulty reentering the driving lane when the tire traverses the vertical edge. The authors indicate that when a tire traverses the vertical pavement edge (or called scrubbing by the authors), resulting forces of the vertical pavement edge act to prevent the tire from reentering the travel lane. This results in the driver increasing the angle of reentry until a critical angle of reentry is reached. As the tire reenters the travel lane, a sudden, steep angled reentry is caused. This action can cause a driver to veer into the adjacent lane, where there is a potential for a collision with other vehicles.

1.73.2 Benefits of Permeable Asphalt Mixtures
No specific benefits were given.

1.73.3 Materials and Mix Design
No specifics on materials and mix design were given.

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1.73.4 Construction Practices

As mentioned previously, this paper did not specifically deal with porous friction courses. However, the paper does describe two pieces of equipment that can be used to construct a safety edge. The safety edge is essentially constructed as a tapered edge approximately 6 to 8 inches in length. One piece of equipment was developed by the Georgia Department of Transportation (GDOT) and the second equipment was designed and built by TransTech Systems, Inc. Both pieces of equipment are generally a wedge that mounts onto a paver. The GDOT device mounts only the end gate of the paver screed. Mixture passes underneath the device and results in a taper from the layer thickness to the desired height of the vertical pavement edge. The authors indicate that the GDOT safety wedge demonstrated the ability to create a safety edge from a dropoff of 0 to 6 inches.

The TransTech Systems, Inc. Safety Edge Maker (SEM) mounts onto a variety of different type pavers. The system has a self adjusting spring that allows the device to follow the roadside surface. A self adjusting screw allows for setting the initial height and the taper to create the safety edge.

The authors concluded that both pieces of equipment successfully constructed safety edges. This was demonstrated with a very fine 9.5 mm nominal maximum aggregate size mixture and a coarser 12.5 mm nominal maximum aggregate size mixture.

1.73.5 Maintenance Practices

No specific maintenance practices were given.

1.73.6 Rehabilitation Practices

No specific rehabilitation practices were given.

1.73.7 Performance

No performance measures were given.

1.73.8 Structural Design

No specifics on inclusion within structural design were given.

1.73.9 Limitations

No limitations on the use were given.

1.74 Martinez, F. C. and R. A. Poecker. "Evaluation of Deicer Applications on Open-Graded Pavements." FHWA-OR-RD-06-12. Oregon Department of Transportation. Salem, Oregon. April 2006.

1.74.1 General

This report documents a research study to evaluate the effects of deicing/ant-icing liquids (magnesium chloride and calcium magnesium acetate) on the friction characteristics of open-graded pavements to determine if their use is associated with any changes in the road surface traction that would result in slick conditions. The intent was not to evaluate the effectiveness of the deicing/anti-icing liquids in preventing snow and ice from

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bonding with the pavement or melting the snow and ice, rather there were concerns that the liquids may affect the frictional properties of the pavement surface immediately after application and prior to snow and ice forming.

1.74.2 Benefits of Permeable Asphalt Mixtures
No specific benefits are given.

1.74.3 Materials and Mix Design
No specifics on materials or mix design were given.

1.74.4 Construction Practices
No specifics on construction practices were given.

1.74.5 Maintenance Practices
The authors devised a research approach that would allow for the testing of both dense-graded and open-graded pavement surfaces at the following points in time:

1. First baseline skid tests. These tests were to be conducted at the end of summer to provide a baseline of performance.

2. Second baseline side tests. These tests were to be conducted immediately after the first rainfall of 6mm or greater.

3. Early winter skid tests. These tests were to be conducted between 30 minutes and two hours after the deicing/anti-icing liquids were applied to the pavement surface at the beginning of the winter season.

4. Late winter skid tests. These tests were also to be conducted between 30 minutes and 2 hours after treatment, but were to be conducted late in the winter season.

All friction testing was conducted with a K. J. Law locked-wheel pavement friction tester. Unfortunately, coordination issues did not allow the researchers to obtain either the first baseline skid tests or the early winter skid tests. Only half of the sections were tested to obtain the late winter skid tests.

Based upon the data collected, the authors concluded that the application of the deicing/anti-icing liquids had little effect on either the dense-graded or open-graded pavement friction. The authors did, however, indicate that the data set was relatively small and further research was needed.

1.74.6 Rehabilitation Practices
No specifics on rehabilitation practices were given.

1.74.7 Performance
Pavement friction was utilized as a performance indicator.

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1.74.8 Structural Design
No specifics on inclusion within structural design were given.

1.74.9 Limitations
Open-graded mixes are not used in high elevation snow zones that require a significant amount of snow plowing.

1.75 "Open Graded Friction Course Usage Guide." California Department of Transportation. Division of Engineering Services. Materials Engineering and Testing Services-MS #5. Sacramento, California. February 2006.

1.75.1 General
This guidelines document prepared by the California Department of Transportation provides an excellent review of OGFC practices in California. The document includes a history of OGFC use in the US, information on materials and mix design, production and construction practices and maintenance practices.

The document states that the initial interest in open-graded mixes grew out of an effort to avoid the shortcomings associated with chip seal construction. These shortcomings primarily involved broken windshields from loose aggregates and time constraint problems associated with setting aggregates during a sudden rainstorm. Oregon began experimenting with open-graded mixes during the 1930's in an effort to improve the frictional properties of pavements. However, problems were encountered that were due to draindown issues. The draindown problems led to areas of asphalt binder rich locations and areas of low asphalt binder. In the 1940's, the use of open-graded mixes began to increase within the western US. As early as 1944, California began to use this mix type for drainage interlayers and as an alternative to chip seals and slurry seals. In the 1980's, Oregon accelerated the use of open-graded mixes when they developed their "F-mix."

The authors of this guidelines document define an OGFC as a sacrificial wearing layer. OGFC consists of a relatively uniform aggregate grading with little or no fine aggregates and mineral filler. These mixes are specifically designed to have large air void contents.

The guidelines state that OGFC can be used in new construction, major rehabilitation projects and maintenance overlays. Layers of OGFC are sacrificial and are used in areas that experience high traffic volumes and moderate to heavy rainfall. When used as a maintenance overlay over a structurally sound pavement, OGFCs can renew the existing surface in terms of functional performance (i.e., ride quality). When used as a maintenance overlay, OGFC layers can rectify/retard the following distresses: raveling, oxidation, skid problems, hydroplaning, splash and spray, flushing and bleeding, and reflective cracking.

The following are specific situations where placement of OGFC wearing layers should be considered:

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- Hydroplaning: OGFCs should be used in areas that have shown a high potential for wet weather accidents. These mix types will reduce hydroplaning potential by providing drainage channels for water to flow beneath the pavement surface. As a result, good tire-pavement contact will be achieved.
- Wet and Nighttime Visibility: OGFCs should be used in areas that have shown a high potential for wet weather and nighttime accidents. OGFCs will minimize the potential for splash and spray. Because the rain infiltrates into the pavement, less water is available at the pavement surface which reduces the amount of reflected light for oncoming traffic.
- Skid Resistance: OGFCs should be used when an increased wet friction number is required for high speed areas. Due to the macrotexture within OGFC mixes, they result in higher friction numbers at higher speeds. At medium speeds, OGFCs and dense-graded surfaces may have similar frictional properties; however, the friction numbers do not decrease as rapidly with increasing speed on OGFC surfaces.
- Location: OGFC should be used when the project is located adjacent to an existing OGFC such as lane addition projects. Placement of dense-graded layers adjacent to OGFC layers may impede the flow of water through the OGFC layers creating a "bath tub" effect.
- Cross Slope: The guidelines state that OGFC should be considered when the cross slope is less than 2 percent and there are two or more lanes in one direction. However, the guidelines also state that OGFC should not be considered a solution to cross slope problems.
- Roadway Smoothness: OGFCs should be considered as an alternative to chip seals and slurry seals when correcting minor surface irregularities. OGFCs should not be used to correct major rutting problems or depressions.
- Oxidation Reduction: OGFCs should be considered in desert areas as a protective layer against oxidation of the underlying dense-graded layer. In this instance, the OGFC is a true sacrificial layer.
- Mitigation of Flushing and Bleeding: OGFCs should be considered as a temporary solution to mitigating flushing and bleeding problems. OGFCs will improve frictional properties and the void structure of this mix allows absorption of the free asphalt binder from bleeding pavements. The guidelines emphasizes that this only provides a short term solution.

OGFCs should not be used when life cycle cost analyses suggest significant savings with other surface types. OGFC is more expensive per ton than dense-graded HMA layers. However, OGFCs are generally placed at a lower unit weight (thickness) which partially offsets the higher costs per ton of OGFC. Using modified asphalt binders increases the costs of OGFCs but use of these binders does tend to increase the life expectancy of OGFCs.

1.75.2 Benefits of Permeable Asphalt Mixtures

The document states that the most important benefit realized from OGFC is the increase in roadway safety during wet weather. The increased safety is due to providing maximum tire to surface contact during wet weather and a strong contrast in pavement

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markings. When rain falls on an OGFC surface, the void structure allows the water to penetrate into the layer which allows for maximum contact between the tire and pavement surface. Also, without the water on the pavement surface, the potential for hydroplaning and splash and spray is greatly reduced.

1.75.3 Materials and Mix Design

Guidance is provided on the selection of materials as well as mix design considerations. Coarse aggregates used in OGFC mixes should be polish resistant. They should also be hard, angular and abrasion resistant to provide stone-on-stone contact, deformation resistance and low susceptibility to clogging.

The aggregate gradation controls the amount of macrotexture and drainage capacity of OGFC layers. Most of the US utilizes a ⅜ in nominal maximum aggregate size gradation while Caltrans allows both ⅜ in and ½ in gradations. In some instances where very high rainfall is expected, a 1 in gradation is sometimes allowed.

Since OGFCs have very high in-place air void contents, air and water are able to come in contact with the asphalt binder coating the aggregates. Therefore, some states allow the use of soft asphalt binders to increase durability by prolonging the time for the binder to age-harden. Alternatively, other states use harder asphalt binders to permit higher mixing temperatures, thus, increasing asphalt binder film thickness to resist age-hardening. Caltrans currently allows the use of unmodified asphalt binders as well as polymer and rubber modified asphalt binders within OGFCs. In general, unmodified binders are used in normal applications and modified binders are used in cooler climates. The document states that OGFC durability is a function of initial binder viscosity and film thickness. Thicker asphalt films improve durability and overall performance.

To select optimum asphalt binder content, several tests are utilized. First, an approximate asphalt binder content is determined using California Test 303, Method of Test for Centrifuge Kerosene Equivalent and Approximate Bitumen Ratio (ABR). This method utilizes the centrifuge kerosene equivalent (CKE) to determine the approximate asphalt binder content. Next, California Test 368, Standard Method for Determining Optimum Bitumen Content (OBC) for Open Graded Asphalt Concrete, is used to determine optimum. This test entails placing loose mixture prepared at the ABR into "extraction thimbles." Next a 4 kg mass is placed on top of the mix and the assembled test samples are allowed to stay in an oven at 275°F (135°C) for 30 min ± 5 sec. After the prescribed time period, the mix is removed from the assembly and the amount of asphalt binder remaining in the assembly is measured [a measure of draindown]. This process is again conducted at asphalt binder contents ± 0.7 percent asphalt from the ABR. After all testing, the asphalt binder contents and average drainage at each asphalt content are plotted with asphalt binder content on the y-axis (form provided in the test method). Optimum asphalt binder content is defined as the asphalt binder content that provides 0.4 g of drainage.

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Caltrans does not design to a minimum air void content. However, the guidance document states that mixes designed as described above would have design air void contents near or above 18 percent.

The role of fine aggregates within OGFC is simply to fill the voids between the coarse aggregate particles. However, the fine aggregates should not bulk (push apart) the coarse aggregates. The percent passing the No. 4 (4.75 mm) sieve is limited to 15 percent, by volume.

Caltrans specifies the optimum mixing temperature within its standard specifications. Use of anti-strip materials is made on the District level in California.

1.75.4 Construction Practices

OGFC can be produced in plants typically used to produce dense-graded HMA. No specific modifications are required. Mixing temperature must be closely controlled in an effort to minimize the potential for draindown problems.

Because of the potential for draindown problems, OGFC should not be stored in a silo for more than two hours. When storage silos are used, it is essential to observe the mix to identify if draindown is occurring.

Draindown can also occur during mix transportation. During transport, best practices should be used to prevent loss of heat and to protect the mix from the weather. Best practices could include tarping the loads, selecting minimal haul times, utilizing material transfer devices, dumping directly into the hopper, etc. For projects with long haul times, the use of polymer modified binders was recommended to allow for higher mixing temperatures.

Only non-petroleum type release agents should be used to coat truck beds. Petroleum based release agents will soften the mix and accelerate deterioration of the pavement.

To ensure proper bonding of the new OGFC layer to the existing pavement, a good tack coat is required. The existing pavement surface should be structurally stable prior to placing the OGFC layer. Cracking and rutting should only be minimal, if existent, prior to placing OGFC. Substantial cracking may reflect up and rutting will provide areas where water will accumulate.

Table 131 provides guidance on the relationship between ambient temperature and placement temperatures. Additionally, the wind must also be considered when constructing OGFC layers. Wind may reduce the temperature of the existing layer and the OGFC. Handwork should be minimized. Transverse joints are difficult to construct with OGFC. The guidance document states that butt joints work best. Longitudinal joints are constructed in a similar manner as dense-graded HMA layers. Caltrans utilizes a method specification to compact OGFC layers. These specifications indicate that two complete coverages with a static steel-wheel roller should be made for compaction. Vibratory and pneumatic tire rollers should not be used. Vibratory rollers will break

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down aggregates within the OGFC and pneumatic tire rollers tend to pick up the OGFC. Also, the kneading action caused by pneumatic tire rollers tends to close the voids at the surface of the OGFC. Table 132 presents temperature limits for OGFC which also includes minimum temperatures for rolling.

Table 131: OGFC Placement Temperatures

Anticipated Atmospheric Temperature	Best Practices
In General	• Follow the requirements in the Standard Specifications and SSPs • Continuously monitor temperature and wind. • Ensure that a tack coat is applied uniformly and at the proper rate. • Compaction of the OGFC pavement is complete after 2 complete coverages of the roller. • Stop operations if the critical temperatures* stated in Table Y cannot be met.
Temp. > 70°F	• For OGFC with conventional asphalt binder, complete rolling before the temperature of the OGFC reaches 195°F. • For OGFC with polymer modified binder, complete rolling before the temperature of the OGFC drops below 250°F. • For OGFC with asphalt rubber binder, complete the first coverage of the initial breakdown compaction when the temperature of the OGFC is greater than 275°F. Complete breakdown compaction before the temperature of the OGFC drops below 250°F.
55°F < T ≤ 70°F	• For OGFC with conventional asphalt binder, complete rolling before pavement reaches 220°F. • For OGFC with polymer modified binder, complete rolling before the temperature of the OGFC drops below 250°F. • For OGFC with asphalt rubber binder, complete the first coverage of the initial or breakdown compaction when the temperature of the OGFC is greater than 280°F. Complete breakdown compaction before the temperature of the OGFC drops below 260°F.
45°F ≤ T ≤ 55°F	• Polymer modified binders must be used. • For OGFC with polymer modified binder, complete rolling before the temperature of the OGFC drops below 250°F.
Temp. < 45°	• OGFC should not be placed.

* Critical temperatures are the minimum atmospheric temperature, minimum pavement temperature, maximum aggregate temperature at plant and the mix laydown temperature range.

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Table 132: OGFC Temperature Limits

Type OGFC	Minimum Atmospheric Temperature	Minimum Pavement Temperature	Maximum Aggregate Temperature at Plant	Recommended Minimum Breakdown Rolling Temperature	Recommended Minimum Finishing Rolling Temperature
Conventional (Normal)	70°F	*	275°F	N/A	195°F
Conventional (Cold Temp.)	55°F	*	275°F	N/A	220°F
Polymer Modified	45°F	*	325°F	N/A	250°F
Asphalt Rubber (Normal)	65°F	65°F	325°F	275°F	250°F
Asphalt Rubber (Cold Temp.)	55°F - 65°F	55°F	325°F	280°F	260°F

1.75.5 Maintenance Practices

The document lists the following as possible distresses encountered in OGFCs

- Shear failures in high stress areas
- Cracking due to fatigue
- Cracking due to reflection from below
- Raveling due to oxidation and hardening of the binder
- Raveling due to softened binder from oil and fuel drippings
- Raveling due to lack of compaction.
- Delamination due to improper tack coat application.
- Clogging of voids from mud, sand, etc. causing loss of permeability
- Rich and dry spots due to draindown of binder during transportation and placement.

Proper selection of materials, mix design, construction and project will help to minimize the occurrence of the above listed distresses.

When distresses occur, the method of maintenance utilized should avoid obstructing the lateral flow of water through the layer. For instance, crack sealing or patching small failed areas with dense-graded HMA can reduce the flow of water through an OGFC layer. When large areas of patching are involved, OGFC should be used to replace OGFC.

OGFC has different thermal properties than typical dense-graded HMA layers. The document states that thermal conductivity is up to 70 percent less than typical dense-graded layers. For this reason, frost and ice will accumulate sooner and last longer on OGFC. This fact is important for maintenance forces to know and understand since it may alter winter maintenance activities. Also, OGFC will require more extensive de-icing measures due to the open nature of the mix.

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1.75.6 Rehabilitation Practices

Caltrans currently only allows removal and replacement for rehabilitating failed or aged OGFC layers.

1.75.7 Performance

No performance measures were given.

1.75.8 Structural Design

Caltrans has assigned a gravel factor of 1.4 to OGFC mixes. However, the document also states that the contribution of the OGFC to structural capacity is not considered.

1.75.9 Limitations

The guidelines provide several instances on where OGFCs should not be utilized which include:

- <u>Unsound Pavements</u>: OGFCs should not be considered as a solution for structurally unsound pavements. The existing pavement must be properly prepared prior to placing OGFC layers.
- <u>Snowy or Icy Areas</u>: OGFCs should not be placed in snowy or icy areas where tire chains, studded tires or snowplows are commonly used. The guidelines do state, however, that the use of modified asphalt binders may assist in resisting the abrasive forces of tire chains, studded tires and snowplows.
- <u>Areas with Severe Turning Movements</u>: OGFCs should not be used in parking areas, intersections, ramp terminals, truck stops, or weigh stations.
- <u>Muddy and Sandy Areas</u>: OGFCs should not be used in areas where dirt, debris, mud, or sands can be tracked onto the surface. These materials will clog the OGFC void structure. Areas that may be prone to dirt, debris, mud or sand include agricultural areas, areas near beaches or areas near sand dunes.
- <u>Areas Prone to Oil and Fuel Drippings</u>: OGFC should not be used in areas with a high potential for oil or fuel drippings. These drippings can soften the asphalt binder within the OGFC and lead to rapid deterioration. Intersections are an area prone to oil and fuel drippings.
- <u>Bridge Decks</u>: OGFC should not be used on bridge decks without special approval.
- <u>Digouts and Localized Areas to be Removed and Replaced</u>: OGFC should not be placed in areas where a bath tub effect can be created.
- <u>Cold Climate Areas</u>: OGFC should not be placed in cold weather (< 45°F).

1.76 Alvarez, A.E., A. Epps Martin, C.K. Estakhri, J.W. Button, G.J. Glover and S.H. Jung. "Synthesis of Current Practice on the Design, Construction, and Maintenance of Porous Friction Courses." FHWA TX-06/0-5262-1. Texas Transportation Institute. College Station, Texas. July 2006.

1.76.1 General

This synthesis provides an excellent overview on the use of OGFC worldwide. Chapters within the report cover advantages/disadvantages, mix design methods, construction and maintenance practices, and performance.

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1.76.2 Benefits of Permeable Asphalt Mixtures

Benefits listed by the authors were divided into three categories: safety improvements, economic benefits and environmental benefits. Benefits related to safety were predominantly related to wet weather conditions. For this reason, the characteristic permeability of OGFC mixes was highlighted as the most important property related to the safety benefits of OGFC mixes. Because of the ability to drain water from the pavement surface, hydroplaning is basically eliminated.

The relatively high permeability of OGFC mixes also greatly reduces splash and spray. Since water drains into the pavement layer, it is not available to create splash and spray. Splash and spray are generally described together; however, the two terms refer to different phenomena. Spray is very fine water particles that are created by rolling tires advancing on wet pavements. Splash is coarser water particles created when tires pass over pools of water on the pavement surface. Both splash and spray contribute to reduced visibility. The reduction in visibility due to splash and spray can be more pronounced than fog because the water droplets in the combined effect of splash and spray have a higher density and are larger in size.

OGFCs also reduce glare, especially at night. The high macrotexture of OGFC layers helps to diffuse the reflections of light during both darkness and daylight. This characteristic improves the visibility of pavement markings. This improvement is also obtained during wet weather since less water is available on the pavement surface.

Wet frictional properties are also improved when compared to dense-graded HMA. This benefit is importantly achieved at higher speeds on porous layers. However, at lower speeds OGFCs and dense-graded layers perform similarly during wet weather.

The authors state that the use of porous mixtures results in reduced fuel consumption due to improved smoothness generally achieved by OGFC. This benefit was the only one identified as related to economics. Additionally, it has been suggested that the rate of tire wear is decreased when driving on OGFC layers.

The primary environmental benefit identified by the authors was the reduction in tire/pavement noise levels. An additional benefit mentioned was that PFC mixtures produce a cleaner runoff than dense-graded HMA layers. This is based upon work that showed lower total suspended solids, total metals and chemical oxygen demand measured on runoff from PFC layers.

The authors state that the noise reduction capacity of OGFC layers is an important benefit to reduce or control highway noise levels. The noise reducing capabilities of OGFC mixes have motivated the use of porous mixtures in Europe while the safety benefits have been the primary motivation for using these mixes in the US. A number of references were cited that provided various decreases in noise levels when OGFC mixes are utilized. In general, OGFC mixes reduce noise levels by 3 to 6 dB(A) when compared to dense-graded surfaces. Because of the reduced noise, drivers have a higher comfort level when driving.

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When comparing the noise reduction of porous mixtures to noise barriers on a unit cost basis, porous mixtures are 2.5 to 4.5 times more efficient. Further, OGFC is more advantageous because the noise is reduced at the source (tire-pavement interface).

1.76.3 Materials and Mix Design

The authors provide an excellent overview of numerous methods for designing OGFC mixes. Following are brief summaries of each mix design method discussed by the authors.

Federal Highway Administration (FHWA) Method

In December of 1990, the FHWA provided technical guidance on the use of OGFC. Included within this guidance was a method for designing OGFC. This method is based on the evaluation of surface capacity of the “predominant aggregate fraction” by immersing and draining the aggregate in S.A.E. No. 10 lubricating oil. Aggregate passing the 9.5 mm (3/8 in.) sieve and retained on the 4.75 mm (No. 4) sieve is considered the predominant aggregate fraction. Using the surface constant value (K_c) determined from the percent retained lubricating oil and the apparent specific gravity of t he aggregate fraction, the asphalt binder content is calculated. The next step entails taking into account the volume of asphalt binder and the target air void content (15 percent) so that the percent of fine aggregate can be calculated. If the air void content at the selected asphalt binder content is not 15 percent or higher, then the gradation of the predominant aggregate fraction should be modified. Two additional tests are also required within the FHWA method once the mixture has been designed. First a test to evaluate draindown potential is run in order to establish optimum mixing temperature. Finally, a test for evaluating the mixture’s resistance to moisture susceptibility is conducted.

National Center for Asphalt Technology (NCAT) Mixture Design Method

NCAT recommended a mix design method for a new-generation OGFC based upon experiences of different states in the U.S., progress in Europe and internal research. This method includes four primary steps: 1) materials selection; 2) selection of design gradation; 3) determination of optimum asphalt binder content; and 4) evaluation of moisture susceptibility. Table 133 provides the recommended requirements for aggregates. NCAT recommended that polymer modified asphalt binders and fibers be used for medium to high traffic volumes and that either polymer modified asphalt binders or fibers be used for low traffic roadways. A single gradation band was recommended form which the design gradation should be developed (Table 134). The design gradation should ensure a high air void content and provide stone-on-stone contact within the coarse aggregate fraction (fraction retained on the 4.75 mm sieve.

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Table 133: NCAT Recommended Aggregate Properties

Parameter	Specified Value
Los Angeles Abrasion, %	30 max.
Fractured Faces, %	90 max with two or more faces 100 max with one face
Flat and Elongated Particles, %	5 max at 5:1 20 max at 3:1
Fine Aggregate Angularity	45 min

Table 134: NCAT's recommended Gradation Requirements

Sieve Size, mm	Percent Passing
19.0	100
12.5	80-100
9.5	35-65
4.75	10-25
2.36	5-10
0.075	2-4

Stone-on-stone contact is evaluated by comparing the voids in coarse aggregate of the compacted mixture (VCA_{MIX}) to the voids in coarse aggregate of the coarse aggregate fraction in a dry-rodded condition (VCA_{DRC}). The VCA_{DRC} is determined in accordance with AASHTO T19, Bulk Density and Voids in Aggregate. When the VCA_{MIX} is less than the VCA_{DRC}, the mixture is considered to have stone-on-stone contact. In order to select the design gradation, three trial blends are developed. Each trial blend is then mixed with 6.0 to 6.5 percent asphalt binder and compacted with 50 gyrations of the Superpave gyratory compactor. The design gradation is selected based upon the existence of stone-on-stone contact and the desired air void content (18 to 22 percent).

Optimum asphalt binder content (step 3) is selected based upon the results of laboratory tests conducted on mixtures comprised of the desired gradation and various asphalt binder contents. Table 135 summarizes the laboratory tests and specification limits recommended by NCAT. Once a mixture has been successfully designed, it is then subjected to moisture susceptibility testing. A modified Lottman method is recommended that includes five freeze/thaw cycles. A minimum tensile strength ratio of 80 percent is required.

Table 135: Specifications for Selecting Optimum Asphalt Binder Content

Parameter	Specification
Draindown Test, %	0.3 max
Air voids, %	18 min
Cantabro Abrasion on Unaged Samples, %	20 max
Cantabro Abrasion on Aged Samples, %	30 max

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The authors also reported on additional research conducted by NCAT to refine the new-generation OGFC mix design method. Some of this recent research confirmed that the design gyration level, 50 gyrations, is appropriate for the design and control of OGFC. Historically, the Cantabro Abrasion test had been conducted on specimens compacted with the Marshall hammer that are both unaged and aged for 120 hours at a temperature of 85°C (185°F). Work by NCAT confirmed that specimens compacted with the Superpave gyratory compactor could be used for the Cantabro Abrasion test. However, NCAT found the testing of aged specimens not necessary since there were only slight differences between results from unaged and aged samples. Requirements for the Cantabro Abrasion were a maximum of 20 percent loss on unaged samples.

Within the original research project, NCAT recommended determining the bulk specific gravity of OGFC samples using volumetric measurements of compacted specimens. Subsequent research recommended using a vacuum sealing method for determining bulk specific gravity. When this method is used, a double bag set up was recommended. Lower air void contents should be expected when using the vacuum sealing method when compared to the volumetric calculation. Therefore, when using the vacuum sealing method, the minimum air void content during design should be 16 percent instead of 18 percent. The final refinement was to reduce the number of freeze/thaw cycles during moisture susceptibility testing from five to one.

Texas Department of Transportation Permeable Friction Course Design

The Texas Department of Transportation (TxDOT) specifies two types of PFC depending upon the type asphalt binder used in the mix. The two binder types include a polymer modified asphalt binder with a minimum high temperature grade of PG 76-XX or a TxDOT Grade C or B crumb-rubber modified asphalt binder. When using crumb-rubber, a minimum of 15 percent, by mass of asphalt binder, is required. PFC mixes utilizing polymer modified binders include 1 to 2 percent lime and 0.2 to 0.5 percent fibers.

Gradation requirements are based upon the type asphalt binder used in the mix. Table 136 provides TxDOT's gradation requirements and asphalt binder requirements. Aggregates must meet coarse aggregate angularity, deleterious materials, soundness, Los Angeles Abrasion, Micro-Deval and flat and elongated requirements.

Table 136: TxDOT Master Gradation Band and Binder Content

Sieve Size, mm	PG 76-XX Mixtures	Crumb-Rubber Mixtures
19.0	100	100
12.5	80-100	95-100
9.5	35-60	50-80
4.75	1-20	0-8
2.36	1-10	0-4
0.075	1-4	0-4
Asphalt Binder Content, %	6.0-7.0	8.0-10.0

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Samples of the PFC are compacted in the Superpave gyratory compactor using a design number of gyrations of 50. Initially, two replicates for each of three binder contents are mixes and compacted. Optimum asphalt binder content is selected based upon the target air voids of 18 to 22 percent. TxDOT utilizes a minimum asphalt binder content requirement of 6 percent.

Mixtures prepared at the selected optimum asphalt binder content are then prepared to evaluate draindown, moisture susceptibility and durability. A maximum draindown of 0.2 percent is specified using a draindown basket. Moisture susceptibility is evaluated by boiling loose mixture in water for 10 minutes and visually evaluating the percentage of stripping immediately after boiling and again after 24 hours. Durability is evaluated using the Cantabro Abrasion test with a requirement of 20 percent loss maximum.

Danish Mixture Design Procedure

The Danish method for designing porous asphalt utilizes the Marshall hammer for compacting samples. A target air void content of 26 percent is used during design. In order to limit excessive asphalt binder contents, the Danish use a draindown test. For rutting, the Hamburg Wheel Tracking test is conducted while for durability the Rotating Surface Abrasion test is used. The premise of the mix design method is to maximize air void contents to provide improved functional properties while also maximizing asphalt binder content to provide durable wearing layers. To assist in these objectives, the Danish generally utilize SBS modified asphalt binders, cellulose fibers, hydrated lime and limestone filler.

The Netherlands Mixture Design Method

The design of porous asphalt in the Netherlands is based upon the compaction of samples using 50 blows per face with the Marshall hammer. A minimum air void content of 20 percent is specified for the Marshall compacted specimens. Porous asphalt gradations have maximum aggregate sizes of 11 and 16 mm with a requirement for crushed aggregates. Penetration-graded asphalt binders are used; however, polymer modification is not generally used.

Australian Mixture Design Method

The design of porous asphalt in Australia includes 80 gyrations of the Australian gyratory compactor. Three maximum aggregate size gradations are used in Australia, 10, 14, and 20 mm. Two classes of porous asphalt are specified in Australia depending upon the expected traffic volume. Type I porous asphalt is suggested for roads with lower traffic volumes while Type II is suggested for higher traffic volumes. For the Type II porous asphalt mixtures, between 0.3 and 0.5 percent fibers are identified as a method for preventing draindown. Modified asphalt binders and hydrated lime are also suggested for the Type II porous asphalt.

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Minimum asphalt binder contents are established using the Cantabro Abrasion test while air voids and draindown are used to establish a maximum asphalt binder content. Air voids are determined after compaction using 80 gyrations of the Australian gyratory compactor. Table 137 summarizes the requirements for designing porous asphalt in Australia.

Table 137: Summary of Design Requirements in Australia

Design Property	Type I Mixture	Type II Mixture
Cantabro Abrasion, Unconditioned %	25 max	20 max
Cantabro Abrasion, Moisture Conditioned %	35 max	
Air Voids, %	20 min	20-25
Draindown, %	0.3 max	0.3 max

<u>Belgian Mix Design Method</u>

In Belgium, aggregate gradations are optimized using a software program entitled PradoWin (Programs for Road Asphalt Design Optimization). Gradations are required to have between 81 and 85 percent "stone fraction" (material larger than 2 mm). The "sand fraction" (material retained on 0.063 mm and passing 2mm) must be between 11 and 13 percent of the aggregates and the filler fraction (passing 0.063) must be between 4 and 6 percent. Maximum and minimum asphalt binder contents are selected based upon volumetric properties and the Cantabro Abrasion loss test.

In order to select a maximum asphalt binder content, porous asphalt samples are compacted with the Marshall hammer at varying asphalt binder contents. A minimum of 21 percent air voids is specified. To select a minimum asphalt binder content, the Cantabro Abrasion loss test is conducted. A maximum of 20 percent loss is specified when testing is conducted at 18°C.

1.76.4 Construction Practices

During the production of OGFC mixes, special attention has to be paid to the moisture within stockpiled aggregates. Control of the moisture enhances the ability to properly control the temperature and homogeneity of the produced mix. Some agencies have a requirement to maintain the stockpiled aggregates at a moisture content near the saturated –surface dry condition. Additionally, some agencies require that at least two days of reserve aggregates be maintained at the production facility.

The authors state that conventional asphalt production plants can be used to produce OGFC. The primary modifications required are a method of introducing fibers and the use of polymer-modified asphalt binders. A fiber feed device is the most common method of introducing fibers. The authors state that pelletized fibers generally include some amount of asphalt binder within each fiber in order to bind the fibers together. When placed in contact with the heated aggregates, the asphalt binder within the pellets

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melts and allows the fibers to be distributed within the mixture. The authors indicate that the asphalt binder included within the fibers should be considered as part of the asphalt binder in the mixture.

When producing with a batch plant with fibers, both the dry and wet mixing times should be increased. This enhances the distribution of the fiber within the mixture. Because almost a single size of aggregate is used to comprise an OGFC's gradation, all screen decks should be inspected prior to beginning production to prevent hot bins from being overridden.

Because of the potential for draindown within OGFCs, close control of production temperatures is needed. Some agencies have limited the mixing temperature to minimize the potential for draindown. Typical maximum temperatures referenced by the author included values from 320 (160) to 347°F (175°C). Spanish standards establish a maximum mixing temperature based upon the type plant used to produce the mixture. The maximum temperature for OGFC produced by drum plants is 311°F (155°C) while for batch plants the maximum temperature is 338°F (170°C). However, some agencies simply recommend a target viscosity for the asphalt binder to be maintained during mixing. The FHWA recommends a target viscosity of 700 to 900 centistokes and within the Design Manual for Roads and Bridges in Britain recommends 0.5 Pa-s.

Also because of potential draindown problems, some agencies limit the amount of storage and transportation times. Typical time limits included for storage are between 1 and 12 hours. The FHWA suggested both a time and distance requirement that for combined handling and hauling of OGFC mixtures should not be more than 40 miles or 1 hour. In Britain, a maximum time limit of 3 hours is specified for mixing, placement and compaction.

In order to minimize the amount of cooling that takes place during transportation, it is common for agencies to require the use of tarps. Maintaining the heat during transportation will help prevent crusting of the OGFC. In Britain, insulated trucks are also required during transportation. Acceptable release agents are also recommended for truck beds. This is especially true when polymer-modified binders are utilized in the OGFC.

OGFC layers should not be considered for correcting surface defects. Prior to placing an OGFC layer, any pavement deficiencies should be corrected. Additionally, areas prone to holding water should be corrected. Areas of the existing pavement that hold water can increase the potential for moisture damage in underlying layers. All OGFC layers should be placed over an impermeable layer by applying an appropriate tack coat and sufficient cross-slope. In Britain, a minimum cross-slope of 2.5 percent is specified. The FHWA suggested a tack coat application of 0.05 to 0.1 gallons per square yard to seal the underlying layer.

As with the construction of any HMA layer, the paver should advance continuously with a minimum number of stoppages. Continuous stoppages will result in an OGFC layer

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that is not constructed smooth. The authors state that the use of a remixing materials transfer device should be considered when constructing OGFC. When a materials transfer device is not used, it is important to minimize the number of cold lumps and crusting that occurs during transportation.

When pavers with extendible screeds are used, auger extensions should be used to avoid irregular distribution of the mixture from the center of the paver to the ends of the screed. Prior to beginning paving, the screed on the paver should be heated. Cold screeds will cause excessive pulling of the placed mat. Handwork should be minimized for OGFC.

A typical manner of defining acceptable paving conditions is to define a minimum ambient temperature. Within the US, an air temperature of 60°F (15°C) is common. The British Manual of Contract Documents for Highway Works also specifies a maximum wind speed as part of acceptable paving conditions.

When compacting OGFC mixtures on the roadway, static steel-wheel rollers are the most common type of roller used. Typically, two to four passes of an 8 to 9 ton roller is appropriate for compacting OGFC. In Britain, five passes is recommended; however, lift thicknesses in Britain are generally 2 inches. Heavy rollers should be avoided as they can lead to excessive breakdown of the aggregates. Pneumatic rollers should not be used because the kneading action of these rollers reduces the drainage capacity by closing the surface voids of the mixture. Vibratory rollers should not be used. The compactive energy created by vibratory rollers can break down the aggregates within the layer.

Longitudinal and transverse joints require special attention. Transverse joints should be minimized if at all possible. Where transverse joints are required, they can be formed by using lumber attached to the underlying layer to create a vertical face. When this method is used, no tacking of the vertical face is needed. However, when a saw cut technique is employed, a small amount of asphalt binder is needed to enhance adhesion. The transverse joint can be formed by placing the screed flat on the existing OGFC approximately 1 ft (0.3 m) before the joint. The new mixture should be allowed to advance in the paver until it reaches the front of the screed. Transverse joints should be cross-rolled. A vibratory roller can be used when constructing transverse joints.

The preferred method of constructing longitudinal joints is echelon paving. When a cold longitudinal joint is required, the joints should not be located within wheel paths or next to pavement lane markings. Longitudinal joints are constructed by placing mixture approximately 0.06 in (1.5 mm) above the existing mat and compacting the mat. The authors state that longitudinal should be sawed prior to placement of the hot side and that a small amount of asphalt binder should be placed on the cut face to promote adhesion. The sawn face should not be completely covered with asphalt binder as that would block the lateral flow of water.

Acceptance of produced and placed OGFC generally entails asphalt binder content and gradation. Only qualitative measures of compaction processes are generally conducted. These qualitative evaluations are generally to assess density material variability and

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segregation. Essentially all agencies specify a minimum value of smoothness. In Britain, a specified permeability of the OGFC layer is required which is tested before traffic is allowed to pass on the pavement. In Spain, acceptance is based upon air voids within the mixture compared to a reference air void content.

1.76.5 Maintenance Practices

Within this report, maintenance was divided into two categories: winter maintenance and surface (general) maintenance. The authors state that maintenance is a fundamental aspect that must be considered for OGFC because maintenance activities on these pavement types are different from for conventional HMA pavements.

Open-graded mixtures have a different thermal conductivity than conventional HMA layers. Because of the relatively high air void contents contained in OGFC layers, the flow rate of heat through the mixture is reduced. The authors cite a source that says an OGFC may have 40 to 70 percent of the thermal conductivity of dense-graded HMA layers.

Because of the differences in thermal properties, the surface of an OGFC layer during cold weather can be between 1.8 and 3.6°F (1 and 2°C) lower than the surface temperature of a nearby dense-graded layers. This results in earlier and more frequent frost and ice formation on OGFC layers than conventional HMA layers. Therefore, it is expected that winter conditions (frost, ice, snow, etc.) will stay on OGFC for longer periods of time. Formation of hazards like black ice and extended frozen periods are considered the main problem with OGFC layer maintenance within the US.

Because of the issues highlighted above, OGFC layers require different winter maintenance practices than dense-graded HMA layers. Some agencies employ the use of pavement condition sensors, meteorological instrumentation, and connecting hardware and software to assist in the decision process for winter maintenance of OGFC.

OGFC layers generally require more deicing agents as well as more frequent applications. In Texas, deicing agents are currently considered the most effective winter maintenance treatment followed by liquid deicers and sand. However, the use of sand or other abrasive substances to improve friction contributes to the clogging of OGFC layers. One problem with many deicers is that the materials can flow into the void structure of the OGFC. The authors cite work in Oregon which states they have conducted research that states organic deicers with higher viscosity and electrostatic charges (similar to emulsions) may improve the bonding of the deicing materials to the OGFC surface.

In Belgium, “intensive” application of deicing materials to OGFC layers is needed to provide similar conditions between dense-graded and HMA surface layers subjected to snowy weather. Also, more frequent applications and about 25 percent deicing material per application are reported in the Netherlands. The use of liquid chloride solutions has been reported as more effective than salt in the Alpine regions of Italy, Austria, and Switzerland. In Britain, the practice is to apply preventative salting just before a snowfall with more frequent applications of salt compared to dense-graded HMA. They also

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recommend additional salt near transitions between OGFC and dense-graded surfaces because there is a reduction in the transfer of salt from the OGFC and dense-graded HMA. When snow plowing is utilized (which is promptly after a snow), they utilize plows fitted with rubber edges on the blades to prevent the blades from damaging the OGFC. However, a Japanese study concluded that the performance of OGFC surfaces and dense-graded surfaces were similar during the winter; therefore, winter maintenance practices were similar.

Based upon cited work, the authors state that within the US there is no major general maintenance programs employed for OGFC surfaces. About the only maintenance activity that is utilized is the use of fog seals and these are only used by a very few US agencies. The authors also state that there is no quantitative evidence that fog seals are effective preventative maintenance techniques. Some research in Oregon has shown reductions in permeability and changes in pavement friction when fog seals are applied; however, it was concluded that the mixtures maintained some level of permeability and macrotexture which still preserved the ability to reduce draindown potential. The authors state that it is expected that fog seals would extend the life of OGFCs since that provide a small film of unaged asphalt binder at the pavement surface.

The authors state that cleaning of OGFC layers to improve permeability is not common practice in the US. The authors state that this indicates that local agencies accept that OGFC functionality is maintained due to the ability to self-clean. Current general maintenance activities in Denmark include cleaning OGFC layers by high pressure water and air suction twice a year.

1.76.6 Rehabilitation Practices

Rehabilitation practices within the report can be divided into minor or major rehabilitation activities. Minor rehabilitation practices include crack sealing, pothole repair, and patching of delaminated areas. When repairing potholes or localized areas of distress, the FHWA advises that one must consider the area and drainage continuity when selecting the rehabilitation technique. In Britain, dense-graded patches are limited in size to 1.64 ft by 1.64 ft maximum. Small areas that are in need of patching can be patched with dense-graded materials as the flow can be maintained around small areas. Large areas should be patched with OGFC mixes or other types of mix that allow water to drain through the pavement. Dense-graded HMA placed in large areas will prevent water from draining.

When dense-graded HMA is used to patch OGFC layers, the patch should be such that it forms a diamond shape. The diamond should be rotated 45 degrees such that water will flow around the patch.

Work in Oregon indicated that machine patches, blade patches, or screed patches with OGFC may be used. This is especially true if some OGFC material still remains in the repair area.

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Major rehabilitation practices mentioned included mill and inlays. This technique was recommended by Oregon when quantities of patching materials justified this technique. Some DOTs state that mill, recycle and inlay is the preferred major rehabilitation technique. This technique is also used in the Netherlands.

In most cases, the milled OGFC is replaced with new OGFC. The practice of placing dense-graded HMA directly over OGFC as a rehabilitation technique is not recommended.

Reports from the Netherlands indicate that recycled OGFC kept approximately the same level of permeability and that durability is about the same as new OGFC layers.

1.76.7 Performance

The authors divide performance into one of two categories: durability and functionality. Durability includes issues such as moisture sensitivity and aging potential. Functionality considers noise reduction and permeability.

The most prevalent distress reported for OGFC layers is raveling. This distress is related to durability. Raveling in OGFC layers is often characterized by its rapid progression. When raveling occurs rapidly, the layer can disintegrate within a few weeks or few months. Raveling can be associated with the aging of the asphalt binder coating the aggregates or from softening of the binder film due to oil and fuel drippings.

As the asphalt binder ages (oxidizes and age-hardening), the asphalt binder film coating the aggregates becomes brittle. When this occurs, the binder cannot accommodate the strain from traffic loadings and results in brittle failure. The authors cite work that suggests a critical asphalt binder penetration of approximately 15 (1/10 mm) and softening point of 158°F (70°C) where OGFCs will fail at low temperatures. The same reference states that the addition of hydrated lime and the use of higher asphalt binder contents will result in lower rates of age hardening and prolong the durability of OGFCs.

The authors state that understanding the impact of field aging is an important factor in the design of OGFC mixes. They provided several areas where the aging of OGFC mixes is different from typical dense-graded HMA layers:

- OGFC layers are placed on pavement surfaces where oxidation rates are higher.
- OGFC mixtures, because of their high permeability, will provide better access of oxygen to the asphalt binder film, tending to increase oxidation rates.
- Thicker asphalt binder films will serve to reduce the oxygen transport rates into the asphalt binder, thereby, slowing oxidation.
- The thicker asphalt binder films in OGFC mixtures will favorably affect the impact of aging on durability compared to dense-graded HMA which have thinner films of asphalt binder.
- Fibers used in some OGFCs may act to reinforce the asphalt binder film and minimize the effects of age hardening.
- The presence of hydrated lime may retard the effects of binder aging.

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- Use of polymer-modified asphalt binders may have a beneficial impact on age-related durability.

As shown in Table 138, the reported service lives of OGFC mixes is very variable and has a range from 6 to 15 years. One of the factors that most influences the durability of OGFC mixtures is asphalt binder type. The majority of agencies that have reported successful use of OGFC are utilizing modified asphalt binders.

Table 138: Typical Reported OGFC Mixture Service Life

Typical Mixture Service Life, yrs.	Type of Mixture	Country
8 or more	OGFC	United States
13	Rubber-modified OGFC (Arizona)	United States
15	OGFC (Wyoming)	United States
6 to 8	OGFC (TxDOT Project)	United States
7 to 10	Porous Asphalt	United Kingdom
7	Porous Asphalt	Denmark
8 to 12	Porous Asphalt	France

The primary performance characteristic related to functionality of OGFC is the high air void content and, thus, permeability. OGFCs will become clogged over time. In an effort to prolong the effect of clogging, some agencies tried to utilize larger aggregate size gradations. Larger aggregate size gradations would result in larger voids. However, major changes in the ability to maintain permeability were not observed. Another method evaluated for maintaining permeability was to design OGFC to higher air void contents. During the 1980's in Spain, OGFC mixes were design with air void contents between 15 and 18 percent. However, these mixes became clogged after a relatively short time. After 1986, they began designing mixes with air void contents above 20 percent and showed an increased functional life. Permeability was maintained for a longer period of time.

Clogging is delayed when suction forces produced by high speed traffic flushes debris from the void structure. Some have suggested that speeds should be higher than 44 mph (70 km/hr) to minimize clogging in OGFC. A minimum speed of 31 mph (50 km/hr) has been reported by others.

Determination of permeability in the laboratory and field is an important part of designing mixtures that will maintain permeability. However, the authors state that the measurement of permeability is not widespread. Several mix design methods do include permeability measurements in the laboratory. Different equipment is needed, though, to measure permeability in the field. The common approach to measuring permeability in the field is to measure a discharge rate for a specific volume of water. Several equipment were mentioned within the report including the IVT Swiss Federal Research Institute device, LCS drainometer used in Spain and a device being used by the Belgium Road

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Research Center. A unique modification to the above described approach of specifying a discharge rate for a specific volume of water is the Zarauz permeameter. With this device, the water falls from a certain height onto the OGFC surface and flows freely into the pavement. Two parameters are mentioned with this test: the maximum radial distance advanced by the water before it penetrates into the OGFC and the total time required for the water to disappear from the pavement surface.

1.76.8 Structural Design

Application of OGFC layers within structural design varies widely throughout the world. In Spain and Britain, OGFC and dense-graded layers are considered to provide similar mechanical responses. Similar conclusions were found by Oregon based upon deflection measurements. However, others have stated that the structural layer coefficients for OGFC should be 60 to 70 percent of the magnitude for dense-graded HMA. Based upon laboratory testing, researchers in Argentina indicated that OGFC had about 50 percent of the structural capacity of typical dense-graded layers.

Most laboratory research studies that have compared the modulus of OGFC mixes to dense-graded HMA have shown that OGFC is not as stiff as dense-graded HMA. However, many references indicate that rutting is not a problem within OGFC layers.

1.76.9 Limitations

The authors highlighted a number of disadvantages related to the use of OGFCs which included reduced performance, high construction costs, winter maintenance, and minimal contribution to pavement structural capacity. As stated above, performance can be categorized as durability and functionality. The primary durability issues are associated with raveling. According to a 1998 survey cited by the authors, service lives of 8 years or more were reported for OGFC and positive experiences were indicated by half of the survey respondents.

For functionality, accelerated loss of permeability and noise reducing capability are the primary concern. Based upon work in Spain, OGFC layers with air void contents near 20 percent retained their permeability for 9 years when subjected to medium traffic, whereas, clogging was reported after 2 years in mixes subjected to heavy traffic.

Construction of OGFC layers is more costly compared to dense-graded HMA layers. On a per ton basis, OGFC in the US will cost between 10 and 80 percent more than dense-graded HMA. Also, the life span of OGFC will be 50 to 100 percent that of dense-graded layers.

The authors state that winter maintenance is considered a significant disadvantage of OGFC layers. Because of the different thermal properties of OGFC layers, larger amounts and more frequent application of winter maintenance activities is required. These activities result in higher maintenance costs for OGFC.

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In many cases, OGFC layers are not considered to provide structural capacity. Therefore, the underlying pavement must be constructed to withstand the effects of traffic and the OGFC is solely a sacrificial layer.

1.77 Poulikakos, L.D., S. Takahashi and M.N. Partl. "Evaluation of Improved Asphalt by Various Test Methods." Report Nr. 113/13 (EMPA No. FE 860076). EMPA. October 2006.

1.77.1 General

This report documents a joint Swiss-Japanese research study focusing on the optimization of porous asphalt gradations. Porous asphalts consist mainly of coarse aggregates with small amounts of fine aggregates and fillers resulting in a mix with an open texture and permeable structure. Due to the open texture and permeable structure, porous asphalt improves frictional properties and provides good visibility by reducing splash and spray on wet surfaces. However, porous asphalt mixes can suffer from problems. Because of the permeable structure, asphalt binder films coating aggregates within porous asphalt are exposed to air and water, increasing the risk of binder aging. This report documents a study to evaluate different tests for characterizing porous asphalt for optimizing gradations and improving mix design methods.

1.77.2 Benefits of Permeable Asphalt Mixtures

Open-graded asphalt is widely used in Western Europe and Japan. Following are benefits identified by the authors when using OGFCs:

- Reduced splash and spray levels from vehicle tires in wet conditions
- Prevention of hydroplaning
- Improved skid resistance in wet conditions
- Reduction in vehicle induced traffic noise generated at the tire/pavement interface
- Reduction in headlight glare from oncoming vehicles on wet pavements
- Improved nighttime wet weather visibility

1.77.3 Materials and Mix Design

The main focus area of this report was a research effort to evaluate the Dry Packing Method of evaluating porous asphalt gradations. This methodology is summarized in Figure 27. The Dry Packing Method (DPM) entails designing a porous asphalt to a design porosity (air void content) based upon the packing characteristics of selected aggregates. The process begins by separating the selected aggregates by size. Figure 27 illustrates the aggregates separated into five sizes. Initially, the largest aggregate size is blended with the finer fraction until the combination of aggregates A and B reach the maximum density (AB). Then AB is combined with aggregate C and the maximum density is determined. The process continues until all but the finest fraction have been combined. Finally, the smallest fraction of aggregate is added (blended) until the target air void content is achieved. [This method appears to have some of the same concepts as the Bailey Method currently being used to design some dense-graded mixes in the US.] A variation of the DPM has also been used to design porous asphalt. This method, called the Wet Packing Method (WPM), includes the same methodology; however, the aggregates are coated with asphalt binder prior to compaction.

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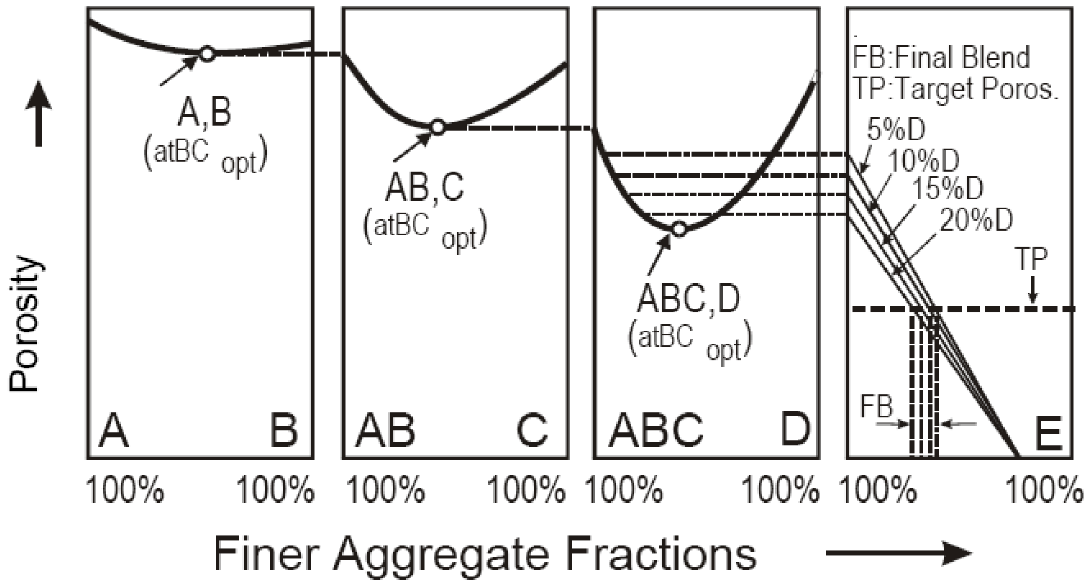

Figure 27: Dry Packing Method for Porous Asphalt

Typical gradation requirements for porous asphalt in Switzerland are provided in Table 139. The wearing courses shown in Table 139 (labeled as DRA) are designed to have 21 to 27 percent air voids. Typical gradation requirements used in Japan are illustrated in Table 140.

Table 139: Gradation Requirements in Switzerland

Sieve Size, mm	Percent Passing			
	DRA 6	DRA 11	DRAT16	DRAT 22
22.4			100	90 – 100
16.0		100	90 – 100	25 – 60
11.2	100	90 – 100	20 – 50	15 – 30
5.6	90 – 100	15 – 40	10 – 25	10 – 20
2.8	15 – 40	8 – 20	7 – 17	---
2.0	10 – 25	---	---	6 – 15
0.5	4 – 10	4 – 10	4 – 10	4 – 10
0.09	3 – 5	3 – 5	3 – 5	3 – 5

DRA – Wearing Courses
DRAT – Base Courses

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Table 140: Gradation Requirements in Japan

Sieve Size, mm	Percent Passing	
	0/20 mm	0/13 mm
19	95 – 100	
13.2	53 – 78	92 – 100
9.5	35 – 62	62 – 81
4.75	10 – 31	10 – 31
2.36	10 – 21	10 – 21
0.60	4 – 17	4 – 17
0.30	3 – 12	3 – 12
0.15	3 – 8	3 – 8
0.075	2 - 7	2 - 7

The authors report conducting a number of different tests on mixes prepared to meet both the Swiss and Japanese gradation requirements. Mixes used during the research were compacted with a Superpave gyratory compactor. Instead of compacting samples to a certain number of gyrations, the samples were compacted to a target air void content of 22 percent.

Tests conducted included permeability testing in accordance with a Japanese Standard laboratory test, an interlayer shear test, indirect tensile test, co-axial shear test, wheel tracking, Cantabro Abrasion loss, and particle loss by shearing.

The interlayer shear test was used to analyze the interlayer adhesion between porous asphalt and dense-graded HMA. The Layer Parallel Direct Shear test (Figure 28) was conducted on samples prepared in the laboratory. Double layer samples were first made by preparing full height dense-graded samples in the Superpave gyratory compactor. After cooling to room temperature, these samples were cut in half. The half samples were then placed back into the gyratory mold and porous asphalt added and compacted.

A typical indirect tensile strength test was conducted. To evaluate different conditions, samples were tested at two different temperatures. In order to evaluate the ability of the different porous asphalt mixes to resist thermal cracking, tests were conducted at 32°F (0°). Samples were also prepared for moisture susceptibility testing. Conditioned samples were placed in a 140°F (60°C) water bath.

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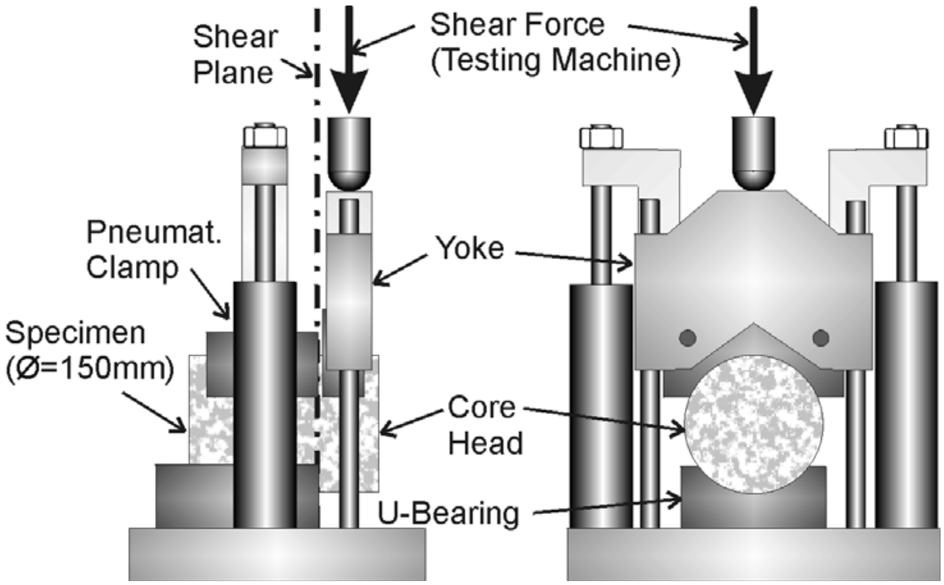

Figure 28: Schematic of Layer parallel Direct Shear Test

The Co-Axial Shear test was conducted to evaluate modulus values for the different porous asphalt mixtures. Tests were conducted at various temperatures and frequencies. To run this test, Superpave gyratory specimens were cored to produce asphalt concrete rings with a 6 in (150 mm) outer diameter and a 55 mm inner diameter. The specimens were then glued to the test molds and the test rod within the center of the sample. The test rod was then torqued to creating the loading. Deformation was measured with LVDTs.

Two wheel tracking tests were conducted. First, the French Wheel tracking test was conducted. Tests were conducted at 140°F (60°C). Secondly, the Japan Highway Public Corporation test was run. Within this test, a steel wheel loaded to 686 N is tracked on the sample for 1 hour. This testing was conducted to evaluate the potential for permanent deformation in the various porous asphalt mixes.

Cantabro abrasion loss test was used to evaluate the resistance to particle loss by abrasion and impact effects. For this study, specimens were conditioned at -4°F (-20°F) prior to testing.

Another test related to particle loss conducted by the authors was the particle loss by shearing force test. Mixture was placed in a mold and exposed to a rotating wheel.

Based upon the laboratory testing, the authors state that mixes designed using the packing theory performed better. The most obvious improvements were observed in the Cantabro Abrasion test and Co-Axial Shear test. Specific conclusions by the authors include:

- Good repeatability was achieved in all the tests used in the study
- Non SBS modified asphalt binders produced specimens with lowest G*.

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- Aging should be used in order to properly identify long term effects of any packing theory
- The Japanese asphalt binder used was stiffer than asphalt binder with NAF used in the Swiss SPA mix
- The Japanese binder yielded a smaller phase angle than the Swiss polymer modified binder (Black diagram). Clear differences in Black diagram between neat asphalt binders and polymer modified binders were observed.
- The Interlayer shear strength of packed mix increased with aging
- Interlayer shear tests indicated no significant differences between dry and wet conditioning
- CAST tests show that in the post-compacted stage the packed mixes are stiffer at high frequencies (unaged and aged)
- In the French wheel tracking tests, the packed and unaged mixes behave quite similar whereas the unpacked mixes differed considerably
- In the aged stage all mixes perform very similar in the French wheel tracking test and the packing theory does not show an improvement.
- The packed theory does show an overall increase in the stiffness of the Japanese and Swiss mixes for aged material as seen in CAST. This increase is especially prevalent at lower temperatures and higher frequencies.

1.77.4 Construction Practices

No specifics were given on construction practices

1.77.6 Rehabilitation Practices

No specifics were given on rehabilitation practices.

1.77.7 Performance

No performance measures were given.

1.77.8 Structural Design

No specifics on inclusion within mix design were given.

1.77.9 Limitations

Typical problems with porous asphalt listed by the authors are as follows:

- Hardening of binder (aging) due to oxidation effects of air reduces durability
- Draindown in the pavement
- Stripping of the binder from the aggregate during service due to high exposure of binder to water
- Densification under traffic, reducing permeability to water

Loss of permeability to water due to clogging by road debris and detritus

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