Final Report
Chip Seal Performance Review
Washington County, Oregon

January 6, 2012
# Project Information and Acknowledgements

| Project Title | Chip Seal Performance Review  
|---------------|-------------------------------|
| Client        | Washington County Operations Engineer  
|               | 1400 SW Walnut Street  
|               | Hillsboro, OR 97123-5625  
| Project No    | 2011.024  
| Report Date   | January 6, 2012  

This report was prepared by Duval Engineering LLC in support of Washington County Operations. We would like to acknowledge the following Washington County and Duval Engineering personnel and we are grateful for their contributions to this document:

- Mr. Greg Clemmons, P.E., Washington County
- Mr. Keith Lewis, Washington County
- Mr. Ted Voelker, Washington County
- Mr. Aaron Clodfelter, Washington County
- Mr. Ed Meeuwsen, Washington County
- Ms. Lindsi Hammond, E.I., Duval Engineering LLC

Please direct any questions or comments about this report to the undersigned.

John I. Duval, P.E., G.E.  
Principal Engineer  
DUVAL ENGINEERING LLC  
3939 N.E. Hancock Street  
Portland, OR 97212  
(503) 473-8240  
www.duvalengineering.com
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INTRODUCTION

**Washington County** retained the services of Duval Engineering LLC to conduct an independent investigation of gravel roadways upgraded with chip seal surfaces. Washington County operates and maintains approximately 1,300 centerline miles of roads. The Washington County Operations Division is responsible for road maintenance and repair and has been in the process of upgrading gravel roads to all-weather roads for many years. These “Gravel Road Upgrade” projects are typically constructed of a triple-shot bituminous surface treatment (BST), or chip seal, by Washington County forces.

The scope of the investigation of eight separate roadways included a review of construction procedures, planning and direction of core sampling, dynamic cone penetrometer testing, and laboratory testing, and development of this report which contains data analysis, conclusions, and recommendations.

Mr. Greg Clemmons, P.E., Washington County Operations Engineer, served as the primary project manager for Washington County. Mr. John Duval, P.E., Duval Engineering Principal Engineer served as the principal investigator and primary author of this report. This effort was conducted in full cooperation with the Washington County Operations team.

This report provides background information, describes core sampling and laboratory testing plans, summarizes test results, describes our analysis methods, and documents our findings and conclusions related to this matter. The reader is referred to the appendices to assist in describing the observed pavement distresses and core locations. Appendix A contains site maps of the eight roadways and 34 corresponding core sample locations. Appendix B contains the general data tables for each roadway, Appendix C contains the photo log of field work, Appendix D contains the dynamic cone penetrometer test results, Appendix E contains the BST aggregate sieve analysis and asphalt content, Appendix G contains the base aggregate sieve analysis, Appendix H contains the subgrade soil classifications, and Appendix I contains the Carlson Testing Inc. raw laboratory testing data.

BACKGROUND INFORMATION

The triple-shot BST roads have generally performed well over the years, but County Operations personnel have observed that certain roads constructed since approximately 2008 are demonstrating early pavement distresses and localized failures including cracking and loss of fines. Operations personnel have responded by conducting localized repairs of affected areas using hot-mix asphalt (HMA).

The number, location, and circumstances of the chip seal failures are relatively complex and
potentially involve a number of variables. This performance review analyzed three areas related to chip seal performance:

1. Material Characteristics
2. Structural Capacity
3. Construction Practices

Material Characteristics: In approximately 2008, the County switched grades of asphalt binder used for the prime coat, or first shot, in the triple-shot chip seal construction. For many years, a MC-250 cutback was used for the prime coat applied at a rate of 0.50 to 0.55 gal/sy. In 2008, the County transitioned to the use of HFMS-2SP asphalt emulsion applied at the same approximate rate as the prime coat. Washington County practice is to use HFE-90-1S as the binder for the second and third shots. The HFE-90-1S has been used consistently for many years and its performance is generally not in question. This review will examine whether the use of HFMS-2SP asphalt emulsion may be a contributing factor to the observed chip seal failures.

In addition to the grade of asphalt binder, there are questions about the gradation of the aggregate used for chip seal projects. The gradation of chip seal aggregate used for the first two shots is 1/2-1/4”. Many agencies that apply double or triple shot chip seals reduce the size of successive chip applications to allow them to fit within the void space of the underlying chip layer. This review will consider this practice and its effect on performance.

The third and final shot of the triple shot BST process uses a 3/8”-#10 size chip. Observations of the wearing surface of GRU roads by Washington County personnel indicate that certain roads may be losing fines and developing a coarse, rough texture.

County personnel report that the amount of fines in the base aggregate has been variable. In some cases, when the amount of fines in the base aggregate was considered low, County personnel would mix additional fine material into the road base. Base aggregate gradation will be reviewed and its impact on performance will be considered. As part of this review, we will compare the aggregate gradation as reported by the material supplier to that of the in-place based aggregate.

Other material considerations are the compatibility between the asphalt binder and the base aggregate and the chip aggregates. Compatibility refers to the ability of the system to develop adequate bond between the asphalt binder and the chip and base aggregates.

Structural Capacity: In addition to material properties, this review will include an analysis of structural design practices. Washington County personnel report that no formal structural design is conducted for the gravel road upgrade projects. Rather, normal county practice is to conduct augering in several locations along the road to ensure a minimum depth of
existing gravel road base of six inches, a practice that has provided adequate structural capacity for most projects.

There is no current practice of assessing subgrade strength, base thickness requirements, or the impact of traffic type and volume as part of a pavement design process. Additionally, there is no regular use of field tests to assess the structural suitability of the existing base prior to application of the gravel road upgrade. Potential field tests and laboratory tests will be reviewed for application as part a structural assessment process.

Construction Practices: Washington County Operations crews have a long record of successful construction of gravel road upgrades. This review provides the opportunity to assess whether the current construction practices are in need of adjustment or improvement.

**CURRENT GRAVEL ROAD UPGRADE CONSTRUCTION PRACTICES**

Washington County Operations crews have a long record of successful construction of gravel road upgrades. The general process involves preparing the existing gravel road by shaping and compacting the base aggregate followed by a triple shot BST.

Existing gravel roads within the County system are identified as candidates for a gravel road upgrade (GRU) by the Operations Engineer. The Rural Roads Operations and Maintenance Advisory Committee (RROMAC) prioritizes and selects roads for the GRU program. Washington County estimates that a typical GRU project costs approximately $110,000 per mile compared to $176,000 per mile for a HMA wearing surface.

Other than relying on past practices and identifying known deficiencies, there is no structural design accomplished as part of the GRU program. (A structural design would take into account the expected traffic, existing subgrade strength, existing base layers and quality, and climate to compute the required base thickness to support the BST surface and protect the road from rutting and cracking). Rather, the County ensures that a minimum of six inches of aggregate exists on the road prior to placing the BST. County personnel inspect the GRU candidate and measure the existing depth of aggregate using an auger. This process is done at 500-foot intervals along the length of the road. Areas where the depth of aggregate is less than six inches are identified and additional aggregate is added. Once a minimum thickness of aggregate is achieved, the road surface is shaped and compacted to the desired profile and grade and is ready to receive the BST surface.

BSTs are one of the oldest and most widely used processes in road construction and maintenance and consist of the following two-part process: 1) a spray application of a hot asphalt cutback or emulsion followed immediately by 2) a single layer of stones (chips) that are then rolled into the hot asphalt. When only a single layer of asphalt and chips are used,
the process is known as a single shot BST, or chip seal. Washington County applies single-shot chip seals to preserve many of its existing HMA roads and streets. This process can be repeated a second time to add a second layer of asphalt and chips to form a “double shot” BST. When a third layer of asphalt and chips is added, this process is known as a “triple shot” BST. The triple-shot BST is used as part of the GRU program.

The first layer of the triple-shot BST, also referred to as a prime coat, starts with the application of an asphalt cutback or emulsion. Prior to 2009, the County used an asphalt cutback product known as MC-250. MC-250 and other cutback products consist of a mixture of paving asphalt and a considerable amount of solvent, such as diesel or kerosene. A typical MC-250 product supplied in the Portland Metropolitan area contains approximately 24 percent solvent. The solvent reduces the viscosity of the asphalt and allows it to be evenly sprayed on the road surface at the desired shot rate. Another desirable quality of the asphalt cutback, MC-250, was that the solvent also allowed a certain amount of the asphalt to penetrate into the underlying gravel mat, creating a strong bond between the BST and the aggregate base layer. Since the aggregate base forms the primary structural component of the road and the BST provides a strong all-weather surface, it is desirable that these two components of the system be well bonded together.

Prior to 2009, the County used MC-250 cutback as the prime shot in a GRU project. In 2009, based on the difficulty in obtaining the MC-250 product from local suppliers, the County switched to using an asphalt emulsion product known as HFMS-2SP. The primary difference between the MC-250 and the HFMS-2SP is that HFMS-2SP uses less solvent and creates an emulsion by mixing asphalt with water using an emulsifying agent, or emulsifier. The emulsion suspends the asphalt in solution and allows the product to be sprayed evenly on the gravel mat. Since there is less solvent in the HFMS-2SP product, it is some question about the ability of the emulsion to penetrate into the gravel mat.
Both the MC-250 cutback (used up to 2008) and the HFMS-2SP (used from 2009 on) are applied from a distributor truck at a shot rate of approximately 0.50 gallons-per-square-yard (see Figure 1). Following the application of the cutback or emulsion, County crews applied 1/2”-1/4” chips using a chip spreader at a spread rate of 35 pounds-per-square-yard (see Figure 2). A steel-wheel roller (static) is used to roll the first layer of stones. The purpose of the roller is to “knock down” the aggregate particles so that they find a stable resting position.

The second and third shots of the triple-shot BST use HFE-90-1S, which is water based emulsified asphalt. The second shot (HFE-90-1S) is applied at a rate of 0.55 gallons-per-square-yard followed by 1/2”-1/4” size chips. The third shot (HFE-90-1S) is applied at a rate of 0.45 gallons-per-square-yard followed by 3/8”-#10 size chips. The second and third shots are rolled using a pneumatic-tire (rubber-tire) roller.

Once the final layer has been applied and rolled, sand is occasionally added to absorb any excess asphalt. This helps to prevent automobile and truck tires from picking rock chips out of the mat once traffic is returned to the road.

Aggregate used to build up existing gravel roads is supplied to the County by Westside Rock in Cornelius. Chip seal aggregates have been supplied by Baker Rock and Knife River over the years. Baker Rock supplies the County from its mining and crushing operations in
Scappoose and Beaverton. Knife River has supplied chip seal aggregates to Washington County from its Deer Island site along the Columbia River in Columbia County. Additionally, West Side Rock began supplying chip seal aggregate for some locations in 2011. Cutback and emulsified asphalt products, including MC-250, HFMS-2SP, and HFE-90-1S are supplied by Albina Asphalt from its Vancouver, Washington terminal. Depending on the project, Albina Asphalt also supplies and operates distributor trucks to apply the asphalt material at the specified shot rate.

**OBJECTIVES**

Based on the initial site visit and conversations with the Washington County team, it was concluded that the primary objective of this performance review is to understand the cause of failure of the gravel road upgrade process on eight specific roads. Secondary objectives include providing recommendations for structural design/analysis, material adjustments, and modification of construction practices. Stated in numerical order, the objectives of this study are as follows:

1. Identify Cause of Failure
2a. Develop Recommendations for Structural Design/Assessment
2b. Develop Recommendations for Improvement of Chip Seal Materials
2c. Develop Recommendations for Modification of Construction Practices

**SITE VISITS**

Three site visits were performed during the course of this project—on August 5, 2011, October 6-7, 2011, and on November 17, 2011.

On August 5, 2011, an initial site visit was conducted to gather information on the extent and the nature of this project. John Duval joined a team of Washington County engineers and maintenance personnel to discuss the observed performance problems (cracking and loss of cover aggregate) with the following eight GRU projects constructed by Washington County, which are shown on the Site Plans in Appendix A:

- Moreland Road
- Solberger Road
- Dorland Road
- Wilkesboro Road
- Riedweg Road
- Chalmers Lane
- Dober Road
- Reiling Road
Based on group observations during the initial site visit, a field and laboratory sampling and testing plan was developed to allow further investigation and analysis.

The second site visit was conducted during the period October 6-7, 2011. During this time, Duval Engineering executed a field sampling plan to extract 34 pavement core samples and to auger and collect samples of base aggregate and subgrade soil. In addition to the field sampling, dynamic cone penetrometer testing was conducted at each of the 34 locations and visual observations were made to document cracking, rutting, loss of cover aggregate and other pavement distresses. Photos were taken to document the testing procedures used and overall pavement performance during the second site visit.

The third site visit occurred on November 17th, 2011 when engineers from the Federal Highways Administration Western Federal Lands, U.S. Forest Service Region 6, Duval Engineering, and Washington County Operations met to discuss the possible causes of preliminary cracking and loss of cover aggregate on gravel road upgrade projects. The meeting began with Mr. Greg Clemmons presenting on the background and potential causes of the premature failures. After the presentation the group discussed other causes and further reasons why the premature cracking and loss of cover aggregate is occurring.

After the discussion the group observed the equipment used to distribute the asphalt and spread the chip rock. Next, the group departed by van for a site visit to Moreland Road, Dorland Road, Solberger Road, Wilkesboro Road, Reiling Road, and Chalmers Lane. During the site visit to these roads, the engineers had the opportunity to observe the distresses first hand. Once the site visits concluded, the group summarized the main points of discussion and documented the primary areas of concern. The benefit of the third site visit was to sharpen the focus on the likely causes of the GRU failures.

**FIELD TESTING**

Duval Engineering developed a test plan to extract 34 core samples from the eight different roads. In addition to pavement coring, the plan called for sampling BST, aggregate, and subgrade materials, sand patch testing of BST texture, measurement of pavement layer thickness, and subgrade strength testing at each core hole.

Duval Engineering and Washington County representatives jointly selected and marked the planned testing locations on the eight roads. Washington County personnel called for utility locates to be conducted to ensure that none of the planned pavement cores were in areas of underground utilities. Utility locates were marked and all test locations were cleared prior to initiation of pavement coring and testing.
On October 6th and 7th, 2011 Duval Engineering and Washington County executed the sampling and testing plan. On October 6, 2011, testing was accomplished on Moreland, Dorland, Solberger, and Wilkesboro Roads. Testing and sampling was completed on Reiling, Chalmers, Riedweg, and Dober Roads on October 7, 2011. Carlson Testing provided a team of two technicians to accomplish coring and material sampling. Washington County provided traffic control. Duval Engineering oversaw the testing and sampling operation and provided an engineer and technician to conduct measurements, perform tests, and record data.

The Carlson Testing crew initiated the exploration at each of the 34 test locations by advancing an 8-inch diameter core barrel as shown in Figure 4. Two pairs of core samples were extracted on each of the eight roads. Core pair locations were selected based on a visual observation of good or poor structural performance or coarse and average surface texture. The exception was Reiling Road where a total of six cores were taken—four in areas of structural deficiency and two in structurally sound areas. After the BST core sample was collected, examined, and its thickness measured, the Carlson technicians augered through the base aggregate until the subgrade soil was reached. During the augering process, a sample of the base aggregate was collected, bagged, and labeled for further laboratory testing.
Duval Engineering conducted thickness measurements of all BST samples and the base aggregate layers as shown in Figure 6. After the base aggregate sample was collected, Duval Engineering conducted subgrade strength testing using a Dynamic Cone Penetrometer (DCP).

The DCP test was executed according to ASTM D 6951. The purpose of the DCP test is to measure the structural capacity of the in-situ subgrade soils. Results of DCP testing have been well correlated to the widely used California Bearing Ratio (CBR) which is a standard index of the strength of soil and unbound aggregate materials. The CBR test is traditionally conducted in the laboratory on a sample of subgrade soil that has been remolded to simulate field conditions (moisture content and state-of-stress). The DCP test, on the other hand, is conducted in the field at the moisture content and stress condition that exists at the time the test is conducted.

DCP tests were performed at each of the 34 test locations identified on the field testing plan. At two test locations, core holes 31 and 32, the DCP met refusal at the top of the subgrade indicating a very stiff subgrade material or perhaps some other obstruction. As a result, no DCP test results were obtained for cores 31 and 32. The results of the other 32 DCP tests can be found in Appendix D.

The DCP tests conducted in support of this investigation were completed in early October 2011, after several months of dry weather. At most test locations the DCP encountered a stiffer layer of subgrade materials that transitioned to very weak slightly plastic fine grained silts and silty clay materials.

After the DCP test was completed a sample of the subgrade was collected and a field classification was accomplished according the Unified Soil Classification System. The sample was sealed in a plastic bag, labeled and stored for further laboratory testing and classification.

In addition to material sampling, Duval Engineering performed a modified sand patch test to assess the average texture depth of the BST surface. This method was selected as a way to
make comparisons between the surface texture of the roads under review.

The sand patch test equipment was modified from that used to conduct a grease smear test on a runway pavement surface as described in Federal Aviation Administration Advisory Circular 150/5320-12C. Duval Engineering adapted the method to use a known quantity of sand, rather than grease, to measure texture depth in these coarse-textured BST surfaces. The procedure involved measuring a known volume of fine sand (1/2 cup) and spreading the sand evenly within a specially configured template of fixed width (4 inches). The length of the resulting rectangular-shaped sand patch was measured with a tape measure allowing the area of the sand patch to be calculated (see Figure 8). Dividing the sand patch area into the known volume allowed the team to compute the average depth of the sand patch, which was taken as the average texture depth (ATD) of the BST surface.

As a way to assess the potential change in surface texture over time, sand patch tests were conducted on two “control” sections which were two recently constructed GRU projects on Jackson School Road and Hahn Road. These two roads were constructed in 2011 and displayed surface texture that was considered about average for a newly constructed BST road using 3/8” - #10 aggregate. Sand patch results for all 34 test locations plus those for the control tests are compiled in Appendix E.

Duval Engineering performed a cursory visual condition assessment of the pavement at each of the 34 test locations in an attempt to identify the type of pavement distresses observed. The cursory survey focused on identifying areas of rutting, cracking, and raveling, which may relate to surface texture. To aid in the evaluation of rutting, field personnel used a six-foot-long straightedge to identify the extent of any surface deformation or rutting. This was done by placing the straightedge on the roadway surface oriented transversely to the direction of traffic so that the straightedge rested perpendicular to the wheel path. Rutting was observed by visual examination and measurement of the vertical distance between the pavement surface and the straightedge using a tape measure.

Figure 8—Sand Patch Test and Equipment

Figure 9—Straightedge Used for Rut Measurement
Digital photographs were captured during the field testing process for the purpose of documenting the testing and sampling process. A Photo Log was prepared and is presented in Appendix C. These photographs were used to validate the visual survey results including the type and extent of cracking and rutting as well as the condition of the roadway surface texture.

At the completion of the field testing stage of this investigation, all material samples were sent to Carlson Testing Inc. for further laboratory testing as described below.

**LABORATORY TESTING**

Laboratory testing was accomplished by Carlson Testing in their Tigard, Oregon laboratory facilities to assess the material characteristics of BST, aggregate, and subgrade soil.

**BST Materials:** For each roadway evaluated, the BST samples (typically four) were grouped to create a single sample for further analysis of asphalt and aggregate materials. Asphalt materials were extracted from the combined BST core sample using AASHTO T164, Test Method for Quantitative Extraction of Bitumen from Paving Mixtures. The extracted asphalt was weighed. Aggregate gradation was measured according to AASHTO T30 Mechanical Analysis of Extracted Aggregate, resulting in a sieve analysis of the BST aggregates. The results of laboratory testing on BST materials are presented in Appendix F.

**Base Aggregates:** Unbound base aggregates were sampled at each of the 34 core holes, four samples on each of the seven roads and six samples from Reiling Road. For assessment of aggregate gradation two samples were selected from each road and subjected to wet and dry sieve analysis according to AASHTO T27 Sieve Analysis of Fine and Coarse Aggregate and AASHTO T11 Materials Finer than No. 200 (75 μm) Sieve in Mineral Aggregates by Washing. The results of this testing are the percent of the aggregate sample passing each of the standard sieves (by weight). The wet sieve analysis when completed in conjunction with the dry sieve analysis permits a much better measurement of the amount of dust in the aggregate sample. The amount of dust is an important quality indicator of unbound aggregate materials.

The results of laboratory testing on base aggregates are presented in Appendix G.

The results of laboratory testing on subgrade soils are presented in Appendix H.

**METHODOLOGY**

**Material Characteristics**

In order to assess the difference in material characteristics among the eight roads, we gathered data from a number of sources including the following:

- Interviews with Washington County Operations staff
- Laboratory test reports
- Supplier quality control reports

Duval Engineering compiled this data which is presented in Appendix B. Comparison and analysis of this data will be discussed in the following section.

**Structural Analysis and Design**

Pavement design was accomplished according to the Asphalt Institute (AI) pavement design methodology using the SW-1 Asphalt Thickness Design Software for Highways, Airports, Heavy Wheel Loads and Other Applications. This design method is a layered-elastic methodology that uses the following design inputs to analyze and design pavements for a variety of pavements. The primary inputs for the AASHTO design method are monthly climate inputs, monthly traffic data, elastic layer properties for each pavement, base, and subgrade layer, analyzed for fatigue and rutting failure over a twenty year period.

For the analysis of Washington County GRU roads, Duval Engineering utilized the Advanced Structural Analysis Module within the SW-1 software package, known as DAMA. SW-1-DAMA is a very powerful analysis tool that allows layered elastic analysis using monthly climate, traffic, and material characteristics inputs.

**Climate**

Environmental effects were characterized by mean monthly air temperatures (MMAT) that influenced the monthly material moduli. The capability to account for monthly variations in
unbound material properties allows one to assess the effects of freeze-thaw or variable moisture conditions throughout a pavement life analysis having 12 distinct periods (months) each year. The following chart shows the mean air temperatures for measurements taken at the Hillsboro Airport taken over past 30 years.

![Mean Monthly Air Temperatures at Hillsboro, Oregon (WRCC 2011)](image)

**Figure 11**— Mean Monthly Air Temperatures at Hillsboro, Oregon (WRCC 2011)

**Traffic**

Washington County supplied traffic counts for the eight roads being investigated. Traffic count data was supplied as a total Average Daily Traffic (ADT), the total number of vehicles that are expected to use the road daily (on average). The traffic counts showed the total vehicles using each roadway in each of thirteen Federal Highway Administration (FHWA) classes of vehicles.

Duval Engineering applied truck factors to each of the FHWA classes of vehicles to compute the number of Equivalent Single Axle Loads (ESALs) over a 20-year design period for each of the FHWA classes of vehicles.

On Moreland and Solberger Roads, we added 2,000 ESALs to each road’s 20-year total of Class 9 vehicles (standard 18-wheel tractor semi-trailer combination). This accounts for the possibility of timber sales in the vicinity of each of these roads that would apply additional vehicles to the roadway.

On Chalmers and Dober Roads, we increased the number of Class 9 vehicles by 20 percent to account for seasonal truck traffic around the farms that front these roads. For Wilkesboro, Reiling and Riedweg Roads, a similar increase of 20 percent was applied to the
Class 8 vehicles to account for the types of trucks that are used by the nurseries and farms that front these roads.

A summation of all the 20-year ESALs for each vehicle class is presented in Table 1 below. For each road, the sum of the 20-year ESALs is shown in the table. For the purpose of computing pavement strength requirements for each road in SW-1-DAMA, a monthly number of ESALs is computed and presented in the table.

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<th>Dorland</th>
<th>Solberger</th>
<th>Wilkesboro</th>
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<td>241</td>
<td>294</td>
<td>193</td>
<td>193</td>
<td>473</td>
<td>131</td>
<td>415</td>
</tr>
</tbody>
</table>

The final traffic input in SW-1-DAMA was the number of monthly ESALs as shown in the bottom row of Table 1. This equates to the number of monthly repetitions of an 18,000 lb single axle load on each of the roadway sections.

Pavement Materials

In SW-1-DAMA, the pavement cross-section was entered using known thickness and elastic properties. For pavement and aggregate layers, we used a Poisson’s Ratio of 0.35. For subgrade soils, Poisson’s Ratio was selected as 0.45. Layer stiffness was selected as 100,000 psi for BST layers and 75,000 psi for aggregate base layers.

Subgrade

Duval Engineering conducted DCP testing at 32 locations on Washington County Roads to establish subgrade strength. The DCP data is used for two purposes. First, the DCP data will be used to analyze the existing strength of each roadway in terms of number of years to failure. (Failure is defined as fatigue cracking or subgrade/base rutting).
Figure 12—Typical Subgrade Strength Model—Dorland Road Shown (Shape Consistent—Values May Differ)

Another use of the DCP data will be to estimate the thickness of the pavement section to support 20-years of traffic on each of the eight roads. The so-called “design strength” will be used as a comparison tool to allow a normalized comparison of the actual strength and the design strength of each of the eight roads.

For analysis purposes, the subgrade layer strength was allowed to vary from a low (weak) point that occurred throughout the winter months then gradually increased to a high point in the September timeframe when the subgrade soils have typically significantly dried. Weakening occurred again beginning in November as the rains softened the moisture-susceptible subgrade soils. The subgrade strength model is presented in Figure 12.

Since the selection of the 34 DCP locations was made based on whether the road was performing well structurally, the Duval team decided to establish three groups of roads for the purpose of establishing design subgrade strength. The first group includes Moreland, Dorland and Solberger Roads. The second group includes Wilkesboro, Reiling, and Chalmers Roads. The third group includes Riedweg and Dober Roads. In each of the three areas, the DCP data was grouped to allow calculation of an average subgrade CBR value. (The DCP data was correlated to CBR values in all cases for computation of pavement strength.)

Structural analysis was conducted in two steps. First, with the input values described above, we utilized SW-1-DAMA to calculate the required pavement thickness for a 20-year design on each of the eight roads using the design strength for the subgrade soils in the three groups of roads. Next, we calculated the design thickness of the existing roads using
the specific subgrade strength measured at each road. To analyze the structural capacity of the eight roads, we compared the thickness of the analyzed section to the design thickness to compute a “percent design thickness,” which is presented in the following section.

Construction Practices

In addition to the material characteristics and the structural capacity of the existing roads, Duval Engineering reviewed the construction practices used by the Washington County Operations team. In order to analyze these construction practices, we interviewed Operations personnel and observed construction equipment used for GRU projects. Further analysis of this information is presented in the next section.

DATA SUMMARY AND ANALYSIS

Material Characteristics

We reviewed the material characteristics are the BST, base aggregate and subgrade soils and present the following data summary and analysis.

Bituminous Surface Treatment

We analyzed the gradation of the BST aggregates used on the eight roads. In particular, we examined the percent passing the No. 200 sieve (P200). Increasing amount of dust in the BST aggregates interferes with the ability of the aggregates to form a bond on the underlying layer. Dust in BST aggregates is typically limited by specification to 1-2 percent of the total weight of the aggregate. Dust content greater than these specification limits can be a cause for rejection of the BST aggregate during construction.

In Figure 13 we show the percent, by weight, of dust in the BST aggregate. The chart shows the 34 core samples on the horizontal axis. There are only 11 data points because the BST samples from the multiple core hole locations at each roadway were combined and tested as a composite sample. Detailed raw data can be found in Appendix I. The vertical axis shows the percent of dust in the BST aggregate. The point for each core hole is colored green, yellow, or red depending on the performance of the road. Well-performing roads are green colored circles—these are roads that exhibited no cracking or rutting. Poorly performing roads are yellow colored squares or red colored triangles, with those roads showing some cracking colored yellow and those roads showing cracking and rutting colored red. Note, all following graphs that use the same performance legend will use the same color and shape arrangement.
Based on the review of this data, we find that in all cases, the percent of fines in the BST aggregate exceeds the two percent specification limit. This is not considered problematic for several reasons, including the following:

1. The BST samples were existing pavement surfaces that were several years old. The amount of dust was likely influenced by this fact.
2. BST samples were composite samples of the entire triple-shot system. The percent fines measured is not specific to any single layer.
3. The BST samples also included base aggregate material that was bound to the bottom of the prime shot layer. This more than likely influenced the results of our testing.

Washington County acquires BST aggregates from several sources, depending on the year. Baker Rock supplies BST aggregates from its Dayton and Farmington locations. Knife River (formerly Morse Bros.) supplies BST aggregates from its Deer Island location. In support of our BST P200 lab analysis, quality control data was collected and analyzed from each of the BST chip suppliers and is presented in Figure 14. From the gradation curves, it is clear that while both Baker Rock and Knife River supply products classified as 3/8”–#10 chips, the chips supplied by Baker Rock are generally finer than those supplied by Knife River. The shape of all three gradation curves shows that all three sources supply chips that meet the objective of a “single-size” stone. The primary difference between the Knife River and Baker Rock sources is that the Baker Rock chips are slightly coarser, with approximately 85 percent passing the 3/8” sieve and approximately 85 percent retained on the #4 sieve. The Knife River Deer Island source has approximately 85 percent passing the 1/4” sieve and...
approximately 95 percent retained on the #8 sieve. This difference in gradation may explain the difference in surface texture observed on Washington County GRU roads.

A review of the BST P200 data shows that the percent fines in the BST aggregates is not related to performance. In other words, there does not appear to be any relationship between the performance of the roads and the number of fines in the BST aggregates.

We also measured the average texture depth (ATD) of the BST surface, which are plotted on the chart shown in Figure 15. In this chart we show the measured ATD based on the sand patch test that was conducted at each of the 34 test locations. Higher ATD indicates a more coarse texture. Lower ATD indicates a finer texture. Each test point shown in the figure is either a green circle or red a triangle based on our observation of surface texture. Where Duval Engineering representatives visually observed the BST surface as being coarse, the data point is a red colored triangle. Where a finer surface texture was observed, the data point is a green colored circle. Lastly, ATD measurements were taken on two control road surfaces—Hahn Road and Jackson School Road. These two roads were identified by Washington County as having been constructed in 2011 and of desirable surface texture. The mean ATD for the control roads was 0.105 inches, which is plotted as a horizontal line on the chart for comparison.
Figure 15—Average Texture Depth of BST Surface

Based on our review ATD measurements at the 34 test locations in comparison with the control surfaces, it is apparent that there is a good relationship between the measured ATD and the visual observation of surface texture. Where measured ATD is greater than about 0.100 inches, the BST exhibited a coarse surface texture while on roads where the measured ATD was less than about 0.100 inches; the BST exhibited a finer, more desirable surface texture.

Figure 16—Average Texture Depth of BST Surface and BST Chip Rock Source
When we analyzed the BST gradations and compared them to the source of the material, in all locations where the ATD exceeded 0.100 inches, the source of the BST aggregate was Baker Rock Dayton. This can be viewed graphically in Figure 16, where Baker Rock Dayton is represented as purple diamonds, Morse Bros Deer Island is shown as pink Xs, and Baker Rock Farmington is shown by the blue asterisks. Detailed ATD information is included in the data tables contained in Appendix B.

Base Aggregate

Samples of base aggregate were collected and shipped to Carlson Testing for laboratory analysis. Both wet and dry sieve analyses were performed on the aggregates to evaluate the gradation of the material. Of particular interest is the amount of fine material or percent passing the No. 200 sieve (P200), in the base aggregate, which is presented in Figure 17. There are 17 data point and each one represents the core hole pair where good or poor performing pavement was observed or coarse or normal surface texture was present. P200 is an indicator of the cleanliness of the base rock. For gravel road applications, it is recommended to have up to 15 percent P200 in the surface aggregate. For GRU projects, the existing gravel road must be re-shaped, new aggregate added in some cases, moisture-conditioned, and compacted prior to receiving the initial BST shot.

![Figure 17—Percent Passing No. 200 Sieve in Base Aggregate (AASHTO T11—Wet Sieve)](image-url)

When new aggregate material is added to the road, Washington County reports that the material is acquired from the Westside Rock quarry. Duval Engineering contacted Westside Rock and requested the mean aggregate gradations from the stockpiles at the quarry. (These quality control gradations are required by the Oregon DOT as part of Westside Rock being listed on ODOT’s Qualified Products Listing (QPL) for aggregate base and other...
Westside Rock supplied the QPL aggregate gradation reports which show that their material is very clean, with typically 1.76 percent P200 in its aggregates (per AASHTO T27—dry sieve analysis). As shown in Figure 17, the percent P200 of the in-place base aggregate is reported using a wet sieve analysis (AASHTO T11) and the results indicate that the P200 is much higher than that of the virgin aggregate source at Westside Rock. As shown in Figure 17, P200 values of the base aggregate ranged from 8 to 25 percent. The difference in P200 between Westside Rock and the in-place base aggregate can be attributed to several factors including: 1) the inability of the dry sieve analysis to accurately measure P200 that “clings” to larger aggregate particles, and 2) the likely high P200 of the existing gravel road prior to adding any additional new rock.

As with previous data, Duval Engineering attempted to correlate the P200 in the base aggregate to performance indicators such as rutting and cracking. In general terms, it appears that those roads with high levels of P200 in the base aggregate appear to have more observed cracking and rutting. Said another way, seven of ten (70%) of the well performing roads had base aggregate P200 values less than or equal to the specified 15 percent maximum. Four of seven (57%) of the roads with structural distresses such as rutting and cracking had P200 values above the specified 15 percent maximum. Six of seven (86%) of the same roads had P200 values near or above the 15% maximum limit.

It is important to have adequate P200 when constructing a gravel road, but having too many fines in the base layer can reduce the structural integrity. Additional fines above and beyond the 15 percent specification limit can reduce the shear strength of the base aggregates, which limits the ability of even thick layers to resist stresses from traffic loading. This is important since the load carrying layer is not the BST surface but the underlying sub layers. These circumstances are explained in the next section.

**Structural Capacity**

We analyzed the raw test data as well as the structural computations reported in the previous section to better understand the relationship between the performance of Washington County roads and various aspects of the structural capacity of the pavements.

Flexible pavement systems, including gravel roads, BST roads, and HMA roads, derive structural capacity from the strength of the underlying subgrade soils and the thickness and quality of the pavement layers constructed above. We know that climate affects the structural capacity of roads, but since the eight roads are all in Washington County, climate conditions are expected to be fairly consistent for all eight roads. Consequently, the structural capacity of the eight GRU projects is influenced by the subgrade strength, base thickness, base quality, and traffic.
Subgrade Strength:
The geologic setting throughout much of Washington County includes near-surface weak clays and low-plasticity silts are relatively weak. As shown in the previous section, DCP testing by Duval Engineering confirmed that in-place CBR values of 1-5 are relatively common for subgrade soils within this group of eight roads. These values are consistent with our previous knowledge of near-surface subgrade soils in the County.

In our subgrade strength model established in the previous section, we show that subgrade CBR is likely to vary throughout the year based on the moisture conditions at depth. The existing clays and silty clays are moisture-sensitive and are prone to become weak when saturated.

The measured CBRs at each test location, under what are believed to be saturated conditions, are plotted in Figure 18 in for each point. There are sixteen data points for the analysis of the soil subgrade because for each road the areas of good performance and poor performance or good surface texture and poor surface texture were combined and analyzed together. At core holes 31 and 32 a subgrade soil sample was unable to be extracted. We know that subgrade CBR is not the only input to the structural capacity of these roads, but we were interested to see if any direct comparison could be made between subgrade CBR and the observed performance of the roads. As in previous charts, test data is plotted for each pair of test locations, indicating that the reported values are averages of the two DCP tests taken at each pair of locations. On the horizontal axis, test locations are numbered. On the vertical axis, CBR at what is believed to be saturated conditions is shown. Test points are green colored circles if no structural distress was observed. Where cracking or rutting was observed, the data point is colored yellow colored squares or red colored triangles, respectively.
All else being equal, it is widely understood that increasing subgrade CBR will result in higher structural capacity—in terms of number of ESALs allowable until failure. A review of the data in Figure 18 shows that other factors are likely influencing the structural capacity in addition to CBR. For example, as shown in the chart at test locations 3-4, the average subgrade CBR was 6 and yet this was an area that exhibited some of the worst cracking and rutting observed on these GRU projects. If only CBR mattered, we would see fewer cracks and less rutting in this area. On the other hand, several test locations showed that even in areas of weak subgrade soils—test locations 9-10 for example—no indication of structural distress was observed.

As gravel roads are built up over weak soils, it is likely that areas with weak subgrade CBR will receive additional thickness of aggregate base. This makes sense because if the pavement system consists of a weak subgrade soil layer and a thin non-structural BST wearing surface, the only layer to provide a significant amount of structural capacity is the aggregate base layer.

Washington County is aware of the importance of having adequate base thickness. They demonstrate this by conducting a quick test ahead of its GRU projects to assess whether at least six inches of gravel is in place to act as a base layer.

Duval Engineering analyzed the thickness of the aggregate base as an indicator of structural capacity on Washington County roads. The following chart, shown in Figure 19, summarizes the measured thickness of base aggregate for each core hole. There are 32 data points in
Figure 19 and 20 due to the inability to completely remove the base aggregate and get a conclusive depth reading at core holes 31 and 32. A similar color scheme was used as before to separate the test locations based on road performance. Green circles indicate good performance—no distresses. Yellow squares are those roads with some cracking. Red triangles indicate test locations where rutting and cracking was observed. We also plotted a line indicating base aggregate thickness of 6 inches. This is the current standard used by Washington County in preparation of existing roads for a GRU project.

![Figure 19—Base Aggregate Thickness Related to Cracking and Rutting Performance](image)

From a review of the data presented in Figure 19, there is a noticeable trend. Well performing roads (green circles) generally have thicker aggregate base layers than those pavements exhibiting either cracking (yellow squares) or combined rutting and cracking (red triangles). While there are some test locations that violate this trend, it can be seen that the majority of the green dots indicate that these pavements have aggregate base thickness greater than six inches.

Considering the chart in Figure 19, it can be seen that if the minimum base thickness were established at 12 inches, all the yellow square and red triangle data points would be below the line. At some other minimum threshold, say 8 inches, the majority of the yellow square or red triangle data points would be below the line.

While both rutting and cracking are indicators of structural distress, rutting is more clearly related to subgrade failure (inadequate base thickness). Cracking can be related to the fatigue failure of the bituminous layer. In a BST road, the surface layer is very thin, about one inch thick, and fatigue failure is not as good of an indicator of structural failure.
Rutting, on the other hand, is related to permanent deformation of the subgrade. Moreover, we protect subgrade rutting by designing and constructing adequate base layers to support the intended traffic.

As shown in Figure 20, a similar analysis was conducted showing only the performance of the GRU projects related to rutting. No cracking was considered in this analysis. Only the performance related to rutting was observed. Rutting of less than 1/4” is considered good performance and the data points are green colored circles. Rutting of 1/4” or more is considered marginal or poor performance and those data points are red colored triangles.

![Figure 20—Base Aggregate Thickness Related to Rutting Performance](image)

From the data presented in Figure 20, we can see a similar pattern as was shown in Figure 19, which related cracking and rutting. Figure 20 focuses entirely on rutting performance and it is clear that the majority of the red triangle data points are at or below the line that indicates the 6 inch base thickness standard for GRU projects. What is evident from Figure 20 is that if the standard were to be raised to 8 inches, only one of the red triangle data points would be above the line. This indicates that, for the eight roads evaluated, having a minimum of 8 inches of rock may reduce the number of roads with rutting of 1/4" or more.

Although the picture is becoming clearer with regard to the relationship between deficiencies in base aggregate thickness and structural performance, some roads handle significantly more traffic than other roads in the County. One way to normalize the effect of traffic is to compute the required “design thickness” of aggregate base needed to support the expected traffic during a 20-year period.

Duval Engineering calculated the required design thickness for each road as discussed
previously in this report. In Figure 21 we present the percentage of design base thickness attained at each pair of test locations. There are 16 data points that represent each core hole pair excluding core hole pair 31 and 32. In other words, what percentage of the design thickness is in place? Each pair of test locations is related to a performance indicator. Green circle data points indicate rutting of 1/4” or less. Red triangle data points indicate rutting of 1/4” or more. In addition, we have shaded the area of the chart where the percent attained is 100 percent or less.

![Figure 21—Existing Base Thickness as a Percentage of Design Base Thickness](image)

What is evident from Figure 21 is that rutting occurs predominantly on those sections that have attained 60 to 80 percent of the design thickness. When the pairings that show structural performance deficiencies are compared to the sections on the same road that indicate good performance, it is a clear that the percentage of design base thickness attained is higher. For example, when comparing test locations 1-2 with those from 3-4 on Moreland Road, with same traffic and climate inputs, we can see a clear difference. The well performing section of road, test locations 1-2, was constructed at 140 percent of the design thickness. The poorly performing section of road, on the other hand, was constructed at only 40 percent of the design thickness. We can make the same types of comparisons for each of the test pairs.

At test locations 21-22 and 29-30, rutting of 1/4” or more was observed even though the actual thickness of the section met 100 percent or more of the design thickness. We interpret as a challenge to the design methodology. What this indicates is that even if a full pavement design was accomplished according to the AI method, using the traffic, climate, and subgrade design inputs described above, rutting would have occurred.
Construction Practices

Duval Engineering reviewed the construction practices employed by Washington County in the GRU program. The following are a number of the topics that were reviewed, investigated, and discussed with Washington County personnel.

Time Between Base Construction and BST Application

Gravel road upgrades are initiated by moisture conditioning, shaping, and compacting the existing gravel road to serve as an adequate base for the new BST surface. County crews conduct the grading, moisture conditioning, and compaction of the gravel road surfaces. Duval Engineering interviewed County construction crews and reviewed construction documents to assess the time lapse between aggregate base construction and the performance of the GRU project on a per road basis. The results of this analysis are presented in Figure 22.

![Figure 22—Elapsed Number of Days Between Base Construction and BST Application](image-url)

The amount of time that passes between the gravel base construction and the BST application is significant because the moisture-density condition of the aggregate base will degrade over time when exposed to heat and traffic. In reviewing the data presented in Figure 22, we observed that for roads where elapsed number of days between base construction and BST application exceeded 15-20 days, the pavements exhibited structural performance distress such as rutting and cracking. Where the number of elapsed days was 10 or fewer, there were generally no distresses noted. This applied to most of the road sections, except on Reiling Road. For this and for other reasons, we believe that the performance data should exclude Reiling Road.
Reiling Road proved to be an outlier specifically in the analysis of the base aggregate. Overall it was structurally deficient; Figure 22 displays the elapsed time between base compaction and BST application and Reiling Road had one of the smallest elapsed times which should have aided in its performance but the data collected stated that it showed signs of rutting and cracking. In Figure 21, core holes 21-22 were analyzed and found to be built at 100% the recommended design thickness. Observations showed that although it was built to the suggested structure it did not perform accordingly. Again in Figure 19, Reiling Road has the minimum or greater existing base thickness and in these same areas the pavement structure did not sustain itself with its current use.

In summary, the Duval Engineering team acquired a tremendous amount of data during the course of this investigation that allowed us to analyze and review the performance of these eight selected GRU projects. We present our conclusions and recommendations in the following section.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based on field and laboratory testing, a review of construction-related documents, test reports and discussions with Washington County personnel, we conclude the following:

1. The surface texture of gravel road upgrade (GRU) projects constructed in Washington County is related to BST aggregate source.

   With regard to surface texture, we did not find any relationship between the use of any particular asphalt emulsion grade or other material property. The only relationship that we observed was between the source of the aggregate used for the BST aggregate and the surface texture. In all cases where the surface texture was visually observed to be coarse and the average texture depth (ATD) was measured to be greater than approximately 0.100 inches, the source of the BST aggregates was Baker Rock. BST aggregates from other sources had a finer BST surface texture with ATD less than approximately 0.100 inches.

   Based on our review of field and laboratory test results BST surface texture is not changing with time. Furthermore, BST surface texture is not affected by the grade of asphalt emulsion used as the prime shot, second shot, or third shot.

   While Washington County has expressed concern about the perceived coarse surface texture of GRU roads, it is the opinion of the author that the surface texture of the eight GRU roads reviewed in this study is acceptable. Chip seal wearing surfaces are, by definition, coarse when compared to a typical dense-graded HMA surface. Even chip
seal surfaces constructed with 1/4"-#10 stones, cannot duplicate the smoothness and dense-graded texture of a well constructed HMA mixture. We believe that for future Washington County GRU projects, the 3/8"-#10 chips that have been used from the three primary sources in this study (Baker Rock—Dayton, Baker Rock—Farmington, and Knife River—Deer Island) will all provide a long-lasting wearing surface and acceptable surface texture.

2. The primary cause of structural failure (rutting and cracking) is inadequate aggregate base thickness.

Structural failure of the Washington County GRU projects, identified primarily as rutting greater than or equal to 1/4", is caused by deficient thickness of aggregate base. After analyzing the subgrade CBR values, traffic volumes, and normalizing for our climate, we conclude that achieving approximately 60 percent of the design thickness of aggregate base would protect the majority of roads from structural failure.

3. Washington County Construction Practices are generally acceptable to construct long-lasting GRU projects.

After discussing means and methods with knowledgeable Washington County Operations personnel and reviewing construction documents related to the GRU projects, it is clear that Washington County construction methods are acceptable. The construction quality of the GRU projects could be improved by reducing the number of days that lapse between gravel road construction and application of the BST. In addition, Washington County could benefit from a quality assurance (QA) program to validate and accept source material and to document internal construction procedures.

From our review of construction records associated with GRU projects in Washington County, less cracking and rutting was observed on roads that were constructed with a minimum of time between initial road shaping and profiling to BST application. Specifically, as shown in Figure 22, the best performance was achieved when the number of elapsed days was kept to 10 or fewer.

QA processes are a relatively straightforward to establish and can pay huge dividends. Essentially, QA is the process of confirming or verifying that the product or service that was received meets or exceeds the minimum specifications. For a GRU project, QA processes would include testing at several steps in the construction sequence. Random testing of base thickness, moisture content, and relative compaction at the time of completion of construction preparation would be an important QA activity. Supplied materials should be subjected to QA testing, such as random testing to assure that BST aggregates meet gradation requirements and that asphalt emulsion products meet the specified grade. Final BST application should be verified by QA testing to assure that the
specified amount of aggregate and emulsion were applied as intended. Independent testing by a third-party consultant or testing laboratory would be the best method of instituting QA testing to ensure that the QA test results are unbiased.

Recommendations

BST Aggregate Size
We recommend that future GRU projects be constructed with three distinctly different aggregate sizes. The current ½” - ½” - 3/8” strategy is problematic in that the first two layers are the same size rock. We recommend a larger 5/8” stone in the first layer to allow the second layer to “nest” in the voids that are formed. The resulting strategy would be 5/8” – ½” – 3/8”. Other than an adjustment to the shot rate for the prime coat (5/8”) all other components of the GRU system should be retained.

Application of a Fog Seal
Based on our conclusion that surface texture is related to aggregate source, we do not recommend the use of a fog seal on low volume county roads. The fog seal would add additional expense to the construction of the project and would have limited benefit on GRU roads.

On the other hand, where GRU projects are applied to higher volume roads and the final surface will be striped, we recommend the use of a fog seal for two reasons. Firstly, the fog seal will aid in chip retention and will minimize loss of chips after construction. Finally, the fog seal provides a dark black surface upon which traffic markings can be placed for maximum visibility.

Institute Quality Assurance Practices
For in-house construction such as GRU projects, we recommend that Washington County institute QA practices to ensure that construction materials meet specified quality levels. Specifically, we recommend developing a process to periodically obtain, on a random basis, representative samples of materials being supplied to the County for GRU projects. This includes BST aggregates, base aggregates, and asphalt emulsion products. Samples should be tested by an accredited, independent laboratory for verification that the products meet County specifications.

Structural Evaluation of GRU Candidates
The County has relied on a system that defines a minimum thickness of six inches for GRU bases layers. In most cases, the six inch base thickness is not adequate to support traffic on even these low-volume roads.

We recommend that the County establish a system of evaluating GRU candidates using some method to ascertain the engineering properties of the existing road. Possible
methods would include a Dynamic Cone Penetrometer (DCP) such as was used for this evaluation or falling weight deflectometer (FWD) equipment. The objective of such an evaluation would be to establish the thickness of aggregate base that must be achieved to ensure that the BST road can last at least 10 years or more.

We are aware that a portable lightweight deflectometer (LWD) device has been developed and could be useful for evaluating GRU candidates. The Minnesota Department of Transportation (DOT) has extensively evaluated the LWD and has developed training materials and specifications for use of the LWD as a QA test device. Based on our review of the Minnesota DOT the LWD appears to be an excellent tool to quickly evaluate construction quality in a cost effective manner. Some of the primary benefits of the LWD are its good correlation between construction quality and pavement design, no need for laboratory testing, and quick feedback to construction crews. Further information on the LWD can be obtained from: http://www.dot.state.mn.us/materials/gblwd.html

The structural evaluation should take into account the existing and future traffic demands of the road and the nature of the existing subgrade soils. We recommend a field evaluation program to collect subgrade soil samples, perform DCP or FWD testing, and perform engineering analysis to establish the base thickness to protect against rutting.

Alternatively, the County may consider simply increasing the minimum base thickness to 8 inches.

Closing Comments

By adopting these recommendations, we believe that Washington County Operations will recognize significant savings, both in the short-term and the long-term on these roadways. Washington County already serves as a leader in building and maintaining cost-effective roads serving the motoring public. We believe that the County is poised to improve its system and to continue to set an example for other agencies to follow.

It has been a pleasure to be of service to Washington County Operations on this project. It would be our pleasure to work with the County on future projects. Please contact the author if you have any questions about the content of this report.

♦ ♦ ♦
REFERENCES


