The Pennsylvania State University
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College of Engineering

IMPROVING UNBOUND AGGREGATE ROADS THROUGH A COMPARISON OF
THE PERFORMANCE OF MATERIALS

A Thesis in
Civil Engineering

by

Danielle M. Lombardi

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The thesis of Danielle Lombardi was reviewed and approved* by the following:

Barry Scheetz  
Professor of Civil Engineering  
Thesis Advisor

Shelley Stoffels  
Associate Professor of Civil Engineering

Angelica Palomino  
Assistant Professor of Civil Engineering

Farshad Rajabipour  
Assistant Professor of Civil Engineering

Mansour Solaimanian  
Senior Research Associate, Geotechnical and Materials Engineering

Peggy Johnson  
Professor of Civil Engineering  
Head of the Department of Civil and Environmental Engineering

*Signatures are on file in the Graduate School
Unbound aggregate roads, or unpaved roads, generally do not hold up to the expectations of the general public. This forces agencies to often pave roads unnecessarily. Wearing course materials used for unpaved roads and the subgrade they are placed on need to be improved in order to see an improvement in these roads. Research was conducted to prove if there is a benefit to using Driving Surface Aggregate (DSA), a material developed specifically for use as a wearing course, over another commonly used aggregate gradation, PennDOT 2A, designed for use as a base course. The performance of DSA and PennDOT 2A under scaled loading with the one-third Model Mobile Load Simulator (MMLS3) was compared in terms of rutting. The materials were evaluated in the laboratory at a lightweight traffic setting (simulating passenger vehicles, pick-up trucks and SUVs) and at a heavy traffic setting (simulating tri-axial and semi-trailers). The MMLS3 was also used in the field on the lightweight traffic setting to evaluate the materials in an as-placed setting. DSA consistently performed better than PennDOT 2A with lower average rut depths, although the predictability of the rut depth formation should be studied in more detail.

In considering subgrade weakness, a rehabilitation practice called full-depth reclamation (FDR) has been used to increase roadbed stiffness in paved roads. The impact of using this method for unpaved roads has been evaluated. In situ stiffness testing using the lightweight deflectometer (LWD) and condition assessments of four roads in Erie County, Pennsylvania were compared to evaluate the potential of FDR as a practice to improve the subgrade for unpaved roads. A road consisting of FDR with a tar and chip surface treatment had the highest overall stiffness when compared to a FDR road without a surface treatment, an aggregate-surfaced road and an unsurfaced road. Both FDR roads showed higher stiffness over the spring-thaw period proving potential benefit to adopting this method as an unpaved road practice; however, future research is needed to evaluate the performance of a FDR road with an aggregate surface. All four
roads showed decreasing condition ratings over the spring-thaw period, but the FDR road with a tar and chip surface treatment required maintenance over the summer months, highlighting one of the major financial downfalls of paving a road.
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Chapter 1

Introduction

Improving the integrity of an unpaved road would require evaluation of both the subgrade and the wearing course, since both have an effect on the performance and overall rideability of a road (Huang, 2004). The Center for Dirt and Gravel Road Studies at The Pennsylvania State University developed a densely-graded aggregate called Driving Surface Aggregate (DSA) as an attempt to improve the wearing course on unbound aggregate roads (Bloser, 2007). To improve subgrade performance, full-depth reclamation has previously been used to rehabilitate paved roads. In Erie County Pennsylvania, an area with subgrade soils subjected to moisture and harsh winters, FDR with cement stabilization was used to improve the subgrade performance on several roads in the area. Most of these roads remained unpaved. The performance of DSA compared to PennDOT 2A as a wearing course and FDR as a means for improving subgrade soils for unpaved roads was evaluated.

Problem Statement

Unbound aggregate roads, commonly referred to as unpaved roads, do not hold up to the expectations of the general public. With this, many agencies are forced to spend money to pave roads unnecessarily. Many of the specifications for aggregate-surfaced roads were developed over time according to an empirical approach of “what has worked” and “what hasn’t worked.” They were also not developed for driving surface purposes. With little money available to maintain roads that do not hold up to expectations, a definitive solution is needed to determine the best practice and designs to use when surfacing unpaved roads with aggregate.
Driving Surface Aggregate (DSA) is an aggregate gradation that was developed relatively recently specifically for use as a surface for unpaved roads. This material is not as well-known and has more strict construction specifications than another commonly-used aggregate gradation, PennDOT 2A, possibly making it less desirable to use. Further research is needed to prove if there is a benefit to using DSA over PennDOT 2A. A rehabilitation practice, full-depth reclamation (FDR), has previously been used to increase roadbed stiffness in paved roads. The impact of using this method for the design of unpaved and aggregate surfaced roads needs to be evaluated.

**Objectives**

The objectives of the research were to determine the following:

1. How does Driving Surface Aggregate (DSA) perform compared to PennDOT 2A as a result of accelerated pavement testing?

2. How much do the benefits of a cement stabilized base provide against frost and moisture? Is it enough to decrease seasonal parameters in unpaved road design?

3. Does lack of a surface overlay on a cement-stabilized full-depth reclamation base affect the performance of the road?

**Scope of Research**

The research focused on collecting three types of data consisting of rut depths, stiffness measurements, and condition assessments. Rut depths as a result of accelerated pavement testing
with the one-third scale Model Mobile Load Simulator (MMLS3) were measured in order to compare PennDOT 2A aggregate with DSA. Both stiffness and condition of full-depth reclamation roads were measured and assessed six times in a one-year span; once a month from February to May, once in the summer (August) and once in the fall (October). This was collected to assess how well the cement-stabilized FDR base held up over the spring-thaw period and in order to determine if an overlay is needed on an FDR road.
Chapter 2

Literature Review

Unbound aggregate roads are designed similarly to asphalt concrete or portland cement concrete roads. The material properties, the underlying subgrade material, the applied loads, and the climate are taken into consideration. Analyzing road performance can be done in numerous ways including full-scale, field observations of deterioration as well as with accelerated pavement testing. Engineering properties can also be evaluated in an attempt to predict the deterioration or to design a better structure. Since the environment plays a large role in the performance of unpaved roads, consideration of how the materials perform seasonally is necessary for proper road design. In order to improve the performance of unbound aggregate roads, the full road system needs to be evaluated. This includes reviewing the current design approach, materials being used, and the methods used for analyzing the system.

Pavement Design

Roadways usually consist of several layers of varying materials providing various stiffness and strength to support the loads they receive. The number of layers, the thickness of layers, and the type of layers all depend on the expected traffic loads. In general, the cross section of a road includes a wearing course, a base course, a subbase, and the subgrade. Depending on the traffic loads for a particular road and the quality of materials used, layers may be eliminated if deemed structurally unnecessary or added if more are needed.

The wearing course is the topmost pavement layer and receives the most direct loading from traffic. It is the most obvious layer to the user. On paved or bound roads, the wearing
course is typically asphalt concrete (AC) or portland cement concrete (PCC). The base and subbase can vary extensively in the types of material used including both untreated granular materials (aggregate) and treated aggregate bases. If added stiffness is needed, bases are most commonly cement or asphalt-treated. It is common on lower volume roads for the base and subbase to consist of an untreated aggregate. Depending on the stiffness required in design the use of a subbase can be eliminated; however, the base and the subbase are also useful at providing drainage. (Huang, 2004; Yoder and Witczak, 1975)

In addition to providing drainage, the layers above the subgrade serve to add structural stability and provide load distribution. This concept is considered by the American Association of State Highway and Transportation Officials (AASHTO) for road design. They developed empirically-based equations for designing a flexible pavement based on material properties, layer thicknesses, and drainage. This method assigns a structure number (SN) to a road in which each of the layers above the subgrade are matched with appropriate structural coefficients dependent on the material properties. The coefficient for each layer is multiplied by the thickness and a drainage coefficient of that layer, and then each layer is added together. (Huang, 2004; AASHTO, 1993) The equation for structure number as outlined in the AASHTO Design of Pavement Structures (1993) is:

\[
SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3
\]

Where \(a_1\), \(a_2\), \(a_3\) are the structural coefficients for each layer. \(D_1\), \(D_2\), and \(D_3\) are the thicknesses of the wearing course, base course, and the subbase, respectively. The \(m_2\) and \(m_3\) factors are for considering drainage of the base and subbase materials.

The empirically-based equation relating SN to traffic loads is:

\[
\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN + 1) - 0.20 + \frac{\log_{10}\left(\frac{APSI}{Z_R}ight)}{0.40+\frac{\log_{10}(SN+1)}{0.35}} + 2.32 \times \log_{10}(M_R) - 8.07
\]
Equation 2 predicts an estimate of traffic for which the subgrade is protected. The use of SN is suggested for flexible pavement design, but not suggested for aggregate-surfaced roads according to the AASHTO design procedure.

Figure 2-1 provides a schematic of pavement cross sections. For a low volume road, the subbase and the subgrade can both be a combination of fine grained and coarse grained soils and aggregates, where the subbase will be a higher quality soil to add stiffness. The base or subbase can also be stabilized with cement additives or other stabilizing agents, or it may be eliminated if additional stiffness is not needed (Huang 2004, Yoder and Witczak 1975). The schematic shows two scenarios of pavement cross sections and does not represent all pavement cross sections used in practice.

For unpaved roads, road design is similar in construction to paved roads; however, the AC or PCC layer is eliminated creating additional needs for drainage and load distribution. With the removal of the AC or PCC layer, the aggregate base course becomes the wearing course.

AASHTO’s Guide for the Design of Pavement Structures (1993) outlines steps to take when designing an aggregate-surfaced road. Design requirements for an aggregate-surfaced road include knowledge of the traffic, season length, resilient moduli of the subgrade, elastic moduli of the aggregates, allowable rut depth for the road, and the amount of aggregate loss. This data can then be used in a computational chart to compute the pavement damage. Design nomographs are
also used to account for serviceability loss and rutting. This graphical procedure must be reiterated several times to determine an adequate thickness of gravel base layer from the known engineering properties. Figure 2-2 is an example of the chart used in designing aggregate-surfaced roads.

The guide places emphasis on not using the same effective modulus values used in flexible pavement design. Season lengths for specific climate regions are important in the design of aggregate-surfaced roads in order to determine the amount of traffic on the road during a particular season. Once the seasonal damage is determined, the thickness of the aggregate layer can be determined. Aggregate loss and conversion of aggregate base to the subgrade can be considered if the effects are significant for the road. The above outlines the use of the AASHTO flexible-pavement design method. Several other design methods exist for both flexible and rigid pavements (Huang, 2004; Yoder and Witczak, 1975); however, AASHTO applies the best to unpaved roads.

<table>
<thead>
<tr>
<th>TRIAL BASE THICKNESS, $D_{bs} = ___$ inches</th>
<th>Serviceability Criteria, $\Delta$ PSI = ___</th>
<th>Rutting Criteria, $RD = ___$ inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Seasonal Roadbed Condition</td>
<td>(2) Roadbed Resilient Modulus, $M_r$(psi)</td>
<td>(3) Base E-Modulus, $E_{BS}$</td>
</tr>
<tr>
<td>Winter (Frozen)</td>
<td>(4) Projected Traffic, $w_{18}$</td>
<td>(5) Allowable Traffic, $(W_{18})_{PSI}$</td>
</tr>
<tr>
<td>Spring/Thaw (Saturated)</td>
<td></td>
<td>(6) Seasonal Damage, $w_{18}/(W_{18})_{PSI}$</td>
</tr>
<tr>
<td>Spring/Fall (Wet)</td>
<td></td>
<td>(7) Allowable Traffic, $(W_{18})_{RUT}$</td>
</tr>
<tr>
<td>Summer (Dry)</td>
<td></td>
<td>(8) Seasonal Damage, $w_{18}/(W_{18})_{RUT}$</td>
</tr>
</tbody>
</table>

Total Traffic = Total Damage = Total Damage =

Figure 2-2: Sample computing chart for determining pavement damage in aggregate-surfaced road design, from the AASHTO Design of Pavement Structures (1993).
Traffic Loading

Loads applied to the pavement by the vehicles add to its degradation and affect the lifespan of a pavement. Traffic loads are generally converted into an equivalent 18-kip single axle load (ESAL) for pavement design. To calculate the number of ESALs a vehicle exerts on a pavement, the number of axles, space between axles, size of the tires, number of tires, and the typical gross weight of a vehicle, and the pavement type is considered (Huang, 2004). Passenger cars, lightweight trucks and SUVs amount to a very small fraction of an ESAL while busses and heavy trucks can amount to many ESALs.

The size and number of tire loads are used to calculate ESALs. The contact pressure the tire exerts on the road surface depends on the area of the tire footprint and the total load on that tire. The tire contact pressure is a function of the tire inflation pressure, load on the tire, and area of the tire in contact with the road. Different models have been used for determining the contact pressure of tires. The uniform pressure model states that radius = load/inflation pressure; therefore the contact pressure should be equal to inflation pressure (Roberts, 1987). Uniform contact pressure of the tire footprint can also be assumed without assuming that the contact pressure and the inflation pressure are the same by following pressure = load/area. Another model used for determining the contact pressure is the Tielking Tire Model. This model considers the structural integrity of a tire; therefore it accounts for the non-uniform pressure it exerts on a pavement (Roberts, 1987). The Tielking Tire Model is a model used with Finite Element Analysis (FEA) to analyze the distribution of contact pressure in a tire. FEA provides more accurate representations of the stresses placed on the road from the tires (Tielking et al. 1987).
Aggregate Wearing (Surface) Courses

There are three types of aggregate gradations commonly specified for unpaved roads throughout Pennsylvania. They are PennDOT 2A, PennDOT 2RC, and Driving Surface Aggregate (DSA). The gradation specifications are provided in Table 2.1. PennDOT 2A was originally developed for use as a base under asphalt surfaced roads. It has a maximum aggregate size of 2 inches and can contain up to 10% materials passing the #200 (0.075mm) sieve. PennDOT 2RC has been widely used as fill material or as base material placed under asphalt. The specification is loosely defined and can contain a large amount of fine clay or soil material. The amount of material passing the #200 (0.075mm) sieve is not defined. Until recently, common practice in Pennsylvania has been to use PennDOT 2A as the wearing surface of aggregate-surfaced roads. The development of DSA has challenged the typical use of PennDOT 2A for use on aggregate-surfaced roads.

Driving Surface Aggregate (DSA) is similar to PennDOT 2A but has a maximum aggregate size of 1 ½ inches as opposed to the 2 inches in PennDOT 2A. It is also a more well-graded material with at least 10% passing the #200 (0.075mm) sieve. This gradation is designed to leave minimal voids when compacted and therefore the highest density and highest stiffness of the three aggregate gradations can be achieved. All fines contained in DSA are specified to be rock dust and must not consist of any clay mineral material. Moisture and lack of stiffness become issues with clay. The effect of plastic fines on pavement performance is discussed more in properties affecting stiffness. The DSA specification originally stated that it should contain 10-20% fines passing the #200 (0.075mm) sieve. This was changed in 2006 to specify 10-15% fines passing the #200 (0.075mm) sieve (Bloser, 2007). Table 2-1 shows the aggregate specifications for percent passing sieve sizes #200 through 2 inches. Figure 2-3 is the plot of the high and low target specifications for DSA and PennDOT 2A.
Table 2-1: Aggregate specifications for DSA, PennDOT 2A and PennDOT 2RC. (*denotes a change in the old DSA from the current DSA gradation)

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>DSA (Old)</th>
<th>DSA (Current)</th>
<th>2A</th>
<th>2RC</th>
</tr>
</thead>
<tbody>
<tr>
<td>2&quot;</td>
<td>-</td>
<td>-</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>1.5&quot;</td>
<td>100</td>
<td>100</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>65-90*</td>
<td>65-95</td>
<td>52-100</td>
<td>-</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>-</td>
<td>-</td>
<td>36-70</td>
<td>-</td>
</tr>
<tr>
<td>#4</td>
<td>30-65</td>
<td>30-65</td>
<td>24-50</td>
<td>15-60</td>
</tr>
<tr>
<td>#8</td>
<td>-</td>
<td>-</td>
<td>16-38</td>
<td>-</td>
</tr>
<tr>
<td>#16</td>
<td>15-30</td>
<td>15-30</td>
<td>10-30</td>
<td>-</td>
</tr>
<tr>
<td>#100</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0-30</td>
</tr>
<tr>
<td>#200</td>
<td>10-20*</td>
<td>10-15</td>
<td>0-10</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 2-3: High and Low Target Gradations for DSA and PennDOT 2A

Subgrade Pavement Layer

The subgrade is the bottom layer in a pavement structure. It is generally the least stiff layer, providing the least amount of support to the structure. Pavement structures are designed to distribute traffic loads reaching the subgrade due to its weakness and susceptibility to problems (Huang, 2004; Yoder and Witczak, 1975). The performance of the subgrade is important in all
pavement design but becomes especially pertinent in unpaved roads due to the minimal amount of materials protecting it. Performance issues arise in the structure when the subgrade cannot support the applied loads.

One factor that can weaken a subgrade is moisture. Research has shown that excessive moisture in the subgrade will lead to a weaker material and an inability to support heavy loads. In pavement design, drainage becomes particularly important for preventing entrapped water resulting in high moisture in the subgrade. Moisture can be a problem in low lying areas, areas susceptible to frost heave, or in the presence of poor drainage (Yoder and Witczak, 1975). Proper measures must also be taken to drain water off the road. Moisture will be discussed more with properties affecting stiffness.

According to Huang (2004) and Yoder and Witczak (1975), the California Bearing Ratio (CBR) tests measures the pressure needed to penetrate a soil with a standard-sized piston compared to that of a high-quality crushed rock. In practice, CBR can be used to calculate the stiffness of a subgrade material. This can also be obtained in the field with a Dynamic Cone Penetrometer (DCP). An attempt to correlate the results of DCP testing to a CBR value has been studied by Salgado and Yoon (2003) in a report for the Joint Transportation Research Program. The DCP instrument consists of three parts: a steel rod with a sixty-degree angle cone tip, an eight-kilogram (17.6-lb) drop weight and a drive anvil located 575 mm (22.6 in) from the bottom of the drop weight. The weight is dropped from the drop height to meet the anvil. Each drop is called a blow. The blow count is converted into a depth per blow ratio (mm/blow or in/blow). The depth per blow ratio is then used to estimate CBR. Salgado and Yoon (2003) highlight the following equation to convert from depth per blow (PI) to CBR:

\[
\log(\text{CBR}) = 2.46 - 1.12 \log(\text{PI})
\]
The AASHTO design guide (1993), Huang (2004), and Yoder and Witczak (1975) provide a commonly-used calculation to convert CBR to a resilient modulus (psi) using the equation:

\[ M_R = 1500 \times CBR \]

In order to distribute traffic loads on a low-volume, unpaved road, aggregate is often placed over the subgrade to minimize the stresses placed directly on the subgrade. This allows for some stress distribution of the traffic loads. Boussinesq, Westergaard, or a simple 2:1 calculation can be used to estimate the distribution of loads (Holtz and Kovaks, 1981).

Geosynthetics have proven helpful in minimizing damage to the subgrade due to applied loads. The success of geosynthetics is due to both separation and reinforcement. This makes them suitable for lessening the amount of aggregate that needs to be placed over a weak subgrade. (Shukla and Yin, 2006)

**Material Properties**

Engineering properties of aggregates and soils have been studied to predict and explain fatigue of both paved and unpaved roads. Numerous studies have been performed to find the common properties that affect the strength and stiffness of roads and more specifically unbound aggregate bases. Due to their effect on permanent deformation of the aggregates, the most common properties studied include particle angularity, density, moisture content, gradation, and type of fines (Tutumluer and Pan, 2008; Yoder and Witczak, 1975). In a study by Tutumluer and Pan (2008), the increase and decrease of these properties are controlled and the effect on the resilient modulus or the modulus of elasticity is measured. The modulus of elasticity (E) is a measure of stiffness of the material and the resilient modulus (\( M_R \)) is a general estimation of the elastic modulus considering rapid loading as experienced from traffic loading (Huang, 2004;
Yoder and Witczak, 1975). The correlations between the various aggregate properties and the moduli can be applied to determine how well they will perform in design.

**Angularity**

Angularity is a shape property of aggregate. It is a function of the amount of fractured faces on the pieces of coarse aggregate, or in other words, not possessing rounded edges. Angularity has been known to improve resilient modulus and lessen rutting mainly due to particle interlock (Yoder and Witczak, 1975). Specifications exist in pavement design to ensure a minimum amount of crushed particle faces in the aggregates used for both unbound and bound materials.

Tutumluer and Pan (2008) studied this directly using the angularity index (AI) and surface texture (ST). They used image analysis on aggregate blends to classify the amount of angular faces in the material as well as the depth of micro texture on the surfaces. After testing twenty-one blends targeting the same amount of fines and voids content at a range of five different stress levels, they found that an increase in crushed particles or particles with high angularity led to a decrease in permanent deformation.

**Moisture and Fines Content**

Standard Proctor is an accepted test for finding optimum moisture content and maximum dry density of compacted soils. The test is specified in ASTM D698. Fines content and moisture content are closely related in their effect on the resilient modulus of unbound aggregates and soils. The changes with an increase in the amount of fines as well as changes due to plastic versus non-plastic fines have been studied.
Mishra et al. (2010) confirmed a common observation that plastic fines negatively affect the performance of unbound aggregates. In the study, maximum density and CBRs were measured for aggregate blends with varying degrees of fines for both plastic and non-plastic fines. Although both the fines types achieved higher maximum dry densities, they both observed lower CBR values with increase in moisture content. Important to note in the study is the fact that the relationship between moisture content and CBR was more drastically affected with the introduction of plastic fines. Figure 2-4 shows the results of CBR performed at optimum moisture on the three different aggregate blends used in the study by Mishra et al. (2010). Gravel refers to an aggregate with primarily rounded faces.

![Figure 2-4: Variation of CBR with percent fines content at optimum moisture. (Mishra et al., 2010)](image)

Lekarp et al. (2000) mentions numerous studies by Haynes and Yoder (1963), Hicks and Monismith (1971), Thom and Brown (1987), Barksdale and Itani (1989), Dawson et al. (1996), and Heydinger et al. (1996), that all argue a decrease in resilient modulus is due to an increase in moisture content. The reduction in resilient modulus could be explained by either excess pore
pressure with loading or from a lubricating effect of moisture. Positive pore pressure causes a decrease in effective stress and therefore a decrease in stiffness and strength. The lubricating effect could be due to a decrease in interparticle forces with increasing water content. Another explanation states that as the moisture content creeps closer to saturation, there is a loss in suction forces between particles. In general, it is noted that the resilient modulus decreases with increasing moisture content above the optimum water content.

**Density**

The effect of density or degree of compaction on the permanent deformation and on the resilient modulus has been carried out in several studies. Yoder and Witczak (1975) describe high density materials to be ideal in terms of stability. Significance of density comes into greater consideration when discussing its effect on permanent deformation. Lekarp et al, (2000) discusses studies that found decreased permanent deformation with increased density such as Trollope et al. (1962), Hicks (1970), Robinson (1974), Rada and Witczak (1981), and Kolisoja (1997). Other studies by Thom and Brown (1988) and Brown and Selig (1991) do not agree that density is significant, while Hicks and Monismith (1971) believe that it becomes less significant in materials containing only crushed aggregates. Mishra et al. (2010) notes that the degree of compaction is used in construction as a quality control measure. In general, it has been observed that an increase in the density of the material will result in an increase in strength and stiffness.

**Climate**

Weathering affects all roadways, paved and unpaved. The layer affected most is the subgrade. It is often the weakest layer and also changes substantially with moisture. It can also
become a problem in areas with cold temperatures. These areas have been known to become most vulnerable to frost heave and spring thaw. For aggregate surfaced roads, the traffic loads are placed almost directly on top of the weakened materials with little load distribution. The effect of weather and season on a road is considered in the AASHTO Guide for Pavement Design.

Moisture in the subgrade soil and the matric suction are generally thought to be the major contributing factors of seasonal variation in resilient modulus. Heydinger (2003) also notes that while resilient modulus is higher in frozen soil, it is significantly lower in thawed soils compared to unfrozen soil. This highlights a hysteresis effect dependent on whether the soil is freezing or thawing. The lower modulus in thawed soils is consistent with the parameters used in pavement design, where the spring freeze-thaw is the weakest.

Moisture in soil on days with temperatures low enough to cause freezing can cause a phenomenon known as frost heave. AASHTO defines frost heave as the expansion in moist soils as ice lenses are formed. Frost heave has been known to cause pavement damage but can be prevented in most cases with proper drainage. The base course and subbase course also serve to protect the subgrade from frost.

Improving Material Properties by Pavement Layers

Driving Surface Aggregate

Generally, unbound aggregate bases are designed to have sufficient permeability to allow for drainage under the AC or PCC, but still be dense enough to allow for strength. With DSA being the wearing course, permeability is less a factor of concern and density becomes priority. Yoder and Witczak (1975) stated that the wearing course of an unpaved road should be designed
such that the water will not permeate and instead run off the roadway. It is imperative that no fines contain any clay mineral content due to the nature of such soils to attract and retain water.

A study was performed with the Center for Dirt and Gravel Roads at Penn State University by Stephen M. Bloser comparing the three most common aggregate gradations used for wearing courses in Pennsylvania. The study concluded that PennDOT 2RC was not a good choice for use as wearing course on unpaved roads. DSA was mixed with 5% clay fines by weight to create the PennDOT 2RC placed in the study. The plastic fines were the most likely cause of deterioration on this section of road. Based on accumulated rut depths, DSA performed the best over the course of the study with the lowest amounts of rutting measures. Although the PennDOT 2A showed slightly greater rutting, the purpose of this study was not to say one aggregate was superior to the other (Bloser, 2007).

**Full-Depth Reclamation**

Full-depth reclamation (FDR) is one of the many means in which a subgrade can be rehabilitated. It is usually used on asphalt cement (AC) roads that are considered beyond repair or where the damage extends beyond the surface of the pavement into the subgrade. The process is thought to be environmentally conscious due to in-place recycling of materials. There are multiple ways in which full depth reclamation can be completed.

After a road is deemed a good candidate for FDR, a machine called a reclaimer is run over the road. The reclaimer breaks up the layers of pavement and mixes it with the base and subgrade layers. The depth of pulverization is typically within six to ten inches but can exceed twelve inches. (Portland Cement Association (PCA), 2012) An aggregate, bitumen, or cementitious additive is mixed in with the pulverized road materials and compacted to form a stable road base.
The Construction Process

The process of full-depth reclamation (FDR) starts with determining if it is the most effective means of rehabilitation for an AC pavement. The next step is to select and determine the proper dosage of an aggregate, bitumen, or cementitious additive needed for the pavement system. Once these preliminary steps are performed, the road construction can begin with the pulverization of the pavement followed by the addition of stabilizer to the reclaimed material. The product is shaped to the proper cross section and grade. Finally, the finished stabilized base needs to be cured properly to achieve the strength predicted by initial tests.

For determining the proper dosage of stabilizer, the evaluation of pavement distresses, core samples, in situ strength of underlying materials, and budget are taken into consideration. Prusinski and Bhattacharja (1999) summarize testing methods for determining the current and desired material properties for cement and lime stabilized road bases. For cement, the best methods for determining the proper dosage is by a Plasticity Index (PI) reduction test, an unconfined compressive strength (UCS) tests, and/or the California bearing ratio (CBR) tests.

After the stabilized base design is determined, the physical work can begin. First, an existing pavement structure is pulverized with the use of a road reclaiming machine. The machine breaks up the pavement materials and crushes them to desired sizes. The depth of reclamation can vary from project to project. It is typically between six to ten inches and includes the wearing surface and underlying materials down to the subgrade. The materials can be stabilized by mechanical, chemical or bituminous means.
Addition of Stabilizer

Mechanical stabilization includes only the addition of granular material with the reclaimed pavement. Chemical stabilization uses cement and/or cementitious materials such as fly ash, lime and calcium chloride to improve base strength or change the original properties of the soil. The resultant product in chemical stabilization is referred to as soil-cement when strengths produced are between 300-800 pounds per square inch (psi) (Prusinski and Bhattacharja, 1999). As well as adding strength, using cement additives in the process may improve frost and moisture susceptibility. Bituminous stabilization makes use of asphalt emulsions and foamed asphalt. Chemical and bituminous stabilization are often combined to produce high-strength, water resistant road bases. (Kearney and Huffman, 1999)

Cross Section

The cross section of the pavement is achieved after mixing the stabilization additives. Kearney and Huffman (1999) suggest a three-pass sequence of rolling compaction, first with a vibratory, smooth drum roller, then with a pneumatic-tire roller, and the final pass with a tandem, steel-wheeled roller without vibration. Research by Christensen (1969) highly recommends prompt compaction of cement-treated soils to achieve the highest possible compressive strengths.

After compaction, curing of the reclaimed material is recommended. Proper curing can be completed with water dampening or membraning. In water dampening, also called moist curing, the surface of the reclaimed material is sprinkled with water to maintain a damp condition. Membraning cures the pavement by sealing the surface, typically with a cutback asphalt or diluted, slow-setting asphalt emulsion. Rafalko et al. (2007) also discusses the effect of common cement admixtures on soil-cement. Calcium chloride can be used as an accelerator;
however, with a low suggested dosage, it has been found to have little effect on the soil-cement set time.

It is recommended that the reclaimed material be cured for seven days before a new surface is applied. Heavy trucks and construction vehicles should be limited during the curing period to avoid structural damage.

FDR should not be performed in rain or when rain is predicted. (PCA, 2005) The moisture can affect the reaction of the chemicals with the reclaimed material. It should also be noted that although laboratory tests prove the improvement in strength and durability with some stabilizers, in the field, the design may not perform the same due to construction errors or inability to be accurate with construction machinery and other uncontrollable factors.

**Cement Stabilization**

Lotfi and Witczak (1985) conducted a study to compare the resilient modulus of cement-stabilized dense-graded aggregate (DGA) and cement-stabilized soil. The effect of aggregate gradation was examined and it was concluded that the resilient modulus is higher for aggregate gradation on the upper limit of the specification, with more fines, than on the lower limit of the specification, requiring zero percent fines. The highest resilient modulus values on cement-stabilized DGA were found with the mix containing the upper limit gradation and the highest cement contents used in the study. The study outlines a difference in stabilization between DGA and soil. It validates the need for preliminary design steps to be taken in the construction process of FDR. Pavement base materials can be stabilized by many means of chemical and bituminous methods. The amount of material stabilized can include many pavement layers including an AC wearing course, base course, subbase and subgrade depending on layer thicknesses. This would be common with pavements designed for medium to high traffic loads. Commonly, low volume
roads have very thin chip seal, AC overlays or granular base material as a wearing course. For the latter situation, soil chemistry can best describe the stabilization process. With more pavement layers reclaimed, the resultant base can be classified as cement-stabilized aggregate base rather than cement-stabilized soil. Due to the variation of the applications of FDR, both stabilization processes will be discussed in more detail.

**Cement-Stabilized Soils**

A look at stabilization in clay soils is important for FDR pavement systems reclaimed that have no overlay or very thin AC overlays. Prusinski and Bhattacharja (1999) outline the mechanism of stabilizing clay soils with cement and lime. Typically clay soils cause problems in pavements and need to be stabilized when they are considered expansive or have high plasticity indexes (PI). Cement is an effective stabilizer because when mixed with clay soil it goes through four important processes necessary for improvement of the clay. The four processes needed for stabilization include: cation exchange, flocculation and agglomeration, cementitious hydration, and pozzolanic reaction.

In cation exchange, the calcium ions present from calcium silicate oxide phases in cement hydration are available for exchange with the sodium and potassium ions present in the double layer of a plastic soil. Since the divalent calcium ions replace the monovalent cations, the double layer thickness is decreased. A reduction in the double layer thickness is directly related to the decrease in plasticity of the clay. Figure 2-5 represents the reduction of a double layer. Calcium ions are also important for flocculation and agglomeration.
Flocculation and agglomeration are also responsible for reducing the plasticity of the clay soil. The reduction in the double layer during cation exchange aids in the rearrangement of the clay particles from a flat, parallel structure to an edge to face structure. Agglomeration refers to the formation of aggregates from the clay particles. This serves to improve the texture of the soil. In addition to lower plasticity, flocculation and agglomeration increase the internal friction between clay particles adding to increase shear strength.

Cement hydration refers to the production of calcium silica hydrates (CSH) and calcium hydroxides when tricalcium silicate (C3S) and dicalcium (C2S) silicate react with water. Portland cement also forms calcium-aluminum-hydrate (CAH) during hydration. The CSH and CAH cement the flocculated clay particles together to provide strength.

Late strength gain in the cement-stabilized system is due to the pozzolanic reactions in the high-pH environment. Silica oxide (SiO$_2$) and alumina oxide (Al$_2$O$_3$) react slowly with the calcium hydroxides to produce more CSH and CAH in the soil. Pozzolanic reaction reduces the system’s susceptibility of calcium leaching and reversal of the stabilization process.

Rafalko et al. (2007) discusses the use of admixtures to soil-cement materials. Common additives in PCC are water reducers, superplasticizers, dispersants and accelerators. These admixtures help with workability, strength and set time of portland cement. Some of the
additives and secondary stabilizers in the study did not prove to be effective in soil cement-stabilization. The study also confirmed cement as being a consistently effective stabilizer.

**Cement-Stabilized Aggregate**

When considering a reclaimed pavement consisting of thicker pavement layers and higher amounts of recycled asphalt material (RAP), discussion on the stabilization differs from that of clay soils. The reclaimed pavement can vary from 100% RAP material to a combination of RAP, granular base material and subgrade soils (Kearney and Huffman, 1999). As granular material and RAP increase, the stabilization relies less on reducing the plasticity of the subgrade material and more on bond strength between aggregates.

A study by Guthrie et al. (2007) concludes that the effects of cement and RAP are interdependent on unconfined compressive strength (UCS). An optimal ratio of RAP to cement content was determined indicating the need for higher percentages of cement with increased amounts of RAP material. The study indicates that weaker bonds may exist between RAP aggregate and cement paste due to the asphalt coating of the aggregate.

**Pavement Analysis**

Many pavements can be analyzed for their resistance to distress using accelerated pavement testing (APT), a method used to accelerate the wear of traffic on a pavement section. Further evaluation of pavements and pavement materials include testing stiffness with non-destructive testing such as with using a lightweight deflectometer (LWD) and evaluating the wear of the pavement with a condition assessment.
Accelerated Pavement Testing

Accelerated pavement testing (APT) began as an approach to test and predict pavement performance quickly as opposed to waiting several years for a road to show signs of fatigue. There are several methods of APT that have been developed and are used today. The first method is use of a test track facility. Test tracks exist all over the world and are used to test many aspects of pavement failure. A vehicle is usually driven around the track until the design life criteria are met and the fatigue is analyzed. Full-scale models can also be used to mimic traffic loading. Small-scale APT devices, such as the one-third-scale Model Mobile Load Simulator (MMLS3) use the concept of a full-scale APT device but on a smaller scale in order to more conveniently test pavements.

Model Mobile Load Simulator (MMLS3)

The MMLS3 is modeled at a one-third scale of a full-sized APT device. Scaling factors have been determined using dimensional analysis and conducting experiments to finalize the concept. The factors most important to modeling pavements were determined using Buckingham’s Pi theorem to find dimensionless parameters and assuming linear elastic materials. It was found that length (depth) should be scaled on the order of 1:N and the load scaled 1:N². The tire pressure and material properties on the pavement system remain the same. Velocity was not affected by the scaling and can remain the same between the full-scale and the model if the effects of inertia can be ignored (Kim et al. 1998).

The MMLS3 has four 300-millimeter (11.8-inch) wheels that run circularly on a track inside the device. Each tire can produce a wheel load up to 2.7 kN (607 lbs) and hold a tire pressure of up to 800kPa (116 psi). Figure 2-6 is a picture of the MMLS3. It can run at speeds
varying from 1800-7200 axles per hour and 2.5 m/s. With the use of a temperature control chamber, the MMLS3 can be run from a temperature as low as -5 degrees Celsius up to 65 degrees Celsius (23-149 degrees Fahrenheit). A testing bed allows for asphalt cores to be tested. Pavements can be tested with water for susceptibility to moisture damage. A profilometer is used throughout testing at specified intervals to determine changes in the surface of the pavement. (Hugo, 2005)

Studies performed by Epps Martin et al. (2003) and Smit et al. (2003) attempted to validate the use of the MMLS3 as a tool for predicting pavement performance. The studies concluded that scaling is not necessary if deformation is only present in the top 100mm of the asphalt cement (AC) layer and that the AC layer and base layer are on top of stiff subgrade material so that rutting is not a result of subgrade deformation. Factors such as weather and wheel wander pattern should be accounted for between full-scale and scaled testing. Smit et al. (2003) found relative performance between the materials as a better assessment of the comparative results. Testing with the MMLS3 has proven useful for surface testing; for example, pavement markings, raveling and wear of surface aggregates.

Figure 2-6: A picture of the MMLS3 set up for laboratory use (a) and a picture of a single tire (one of four) used to apply the traffic loads (b).
Chapter 3

Experimental Methods

Both laboratory and field tests were used to evaluate unpaved road materials. The laboratory experiments included testing surface aggregates, Driving Surface Aggregate (DSA) and PennDOT 2A, with the one-third scale Model Mobile Load Simulator (MMLS3). Field experiments were performed on four roads in Erie County, Pennsylvania to evaluate properties and conditions of subgrade materials. Testing was also performed in the field using the MMLS3 to test surface aggregate in more natural conditions.

The laboratory pavement section was constructed in a wooden box (shown in Figure 3-1a) with dimensions 40x103x15 inches. The total depth of the box used for accelerated pavement testing with the MMLS3 was determined by analysis using a software program called “Kenlayer” (Huang, 1996). The subgrade soil was compacted in two lifts with a vibratory roller to a total depth of nine inches at near optimum moisture content of approximately 14.4% and to a density ranging from 96.5 - 99.4% of the maximum dry density. The driving surface aggregate (DSA), was placed at eight inches and compacted to six inches at optimum moisture content ranging from 98 - 101.4% of the maximum dry density. PennDOT 2A was placed in two lifts and hand-tamped after each lift. Additionally, the MMLS3 was run for 500 cycles in order to compact the material further. The aggregate was placed at 6.7% moisture content and compacted to a density ranging 93.2 - 93.5% of the maximum dry density for lightweight testing. PennDOT 2A was placed similarly for heavy traffic testing with density ranging from 91.0 - 93.7% of the maximum dry density. The locations for the density readings for the subgrade, the DSA and both the PennDOT 2A placements are shown in Figure 3-8. The field experiments with the MMLS3 were performed on a lightweight vehicle setting over a pavement section with a stiff base material and no existing
surface present. The field test sections were compacted by the Erie Conservation District according to the material construction specifications set by The Center for Dirt and Gravel Roads and PennDOT.

**Tire Considerations**

Length and width for the laboratory experimental road section structure shown in Figure 3-1b, were designed by determining for the best fit of the MMLS3 and considering effect of boundary conditions. The depth was determined by calculation and analysis of the effects of contact pressure on the proposed road materials. In order to determine the contact pressure and load that would best represent the proposed traffic, a full-size tire footprint of lightweight vehicles was compared to the model-sized tire footprint of the MMLS3. The structure had dimensions 40”x 103”x 15”.

Figure 3-1: The experimental road section box used for testing road surface aggregates with the MMLS3 in the laboratory. Depth (a) is 15” and length and width (b) are 103” and 40”.
Two passenger cars and a lightweight truck were used in the experiment to find the area of the tire in contact with the road surface. The rear passenger-side wheel was used on the 2004 Chevrolet Cavalier, LS Sport passenger car and the front, driver’s-side tire of a 2005 Honda Civic. A 2001 Chevrolet Suburban was used to represent a lightweight truck. The front, driver-side wheel was also used to obtain the contact area. The vehicle tires were driven through mud, jacked up, and then placed onto a clean sheet of poster board. The area of the tire in contact with the ground left behind a mark that was then measured and the contact area was calculated. Two tire inflation pressures were used, one to indicate pressure in cold tires, and one to indicate pressure in hot tires or over-inflated tires. The “hot” and “cold” tire pressures tested from the Honda Civic were found to be equivalent. Figure 3-2 shows the tire prints from the vehicles.

![Figure 3-2: Tire footprints from several passenger cars and lightweight trucks used to determine approximate pressures the vehicle tires have in contact with the pavement. a) Chevy Cavalier b) Chevy Suburban c) Honda Civic](image)

The approximate weight of the vehicle displaced on the wheel divided by the contact area was used to calculate the contact pressure. Weights for the Honda Civic and the Chevy Cavalier were obtained from the vehicles. This information was not available on the 2001 Chevy
Suburban; therefore, the curb weight was obtained from two websites: Edmunds.com, Inc (n.d.) and Kelley Blue Book (2012). Both publish an identical curb weight for the vehicle.

The Chevy Cavalier tires have more of an oval contact area, where the Honda Civic and Chevy Suburban left more of a “rounded, rectangular” shape. For the purposes of this study, the area for the Honda Civic and the Chevy Suburban were assumed to be the average between oval and rectangular tire print areas (shown in italics). The average contact pressure was calculated from the average contact area, not the average between the contact pressures. The vehicle weight was assumed to be evenly distributed even though the front tires may carry more weight than the rear tires. An exact ratio was not available and therefore not used. Contact pressures were compared to the contact pressure of the MMLS3. Tables 3-1 and 3-2 show the calculated areas and contact pressures of the tires tested.

Table 3-1: Calculated contacts pressures for several different tires and cars including a Chevy Cavalier, a Chevy Suburban, a Honda Civic, and the MMLS3.

<table>
<thead>
<tr>
<th>Vehicle</th>
<th>Tire Pressure (psi)</th>
<th>Tire Print Length (in)</th>
<th>Tire Print Width (in)</th>
<th>Oval Area (in²)</th>
<th>Rect'l Area (in²)</th>
<th>Vehicle Weight (lb)</th>
<th>Tire Load (lb)</th>
<th>Contact Pressure - Oval (psi)</th>
<th>Contact Pressure - Rect'l (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Car - Chevy Cavalier</td>
<td>33</td>
<td>4.375</td>
<td>5.75</td>
<td>15.8</td>
<td>20.1</td>
<td>2779</td>
<td>694.75</td>
<td>35.2</td>
<td>27.6</td>
</tr>
<tr>
<td>Car - Chevy Cavalier</td>
<td>52.5</td>
<td>3.5</td>
<td>5.5</td>
<td>18.9</td>
<td>24.1</td>
<td>2779</td>
<td>694.75</td>
<td>46.0</td>
<td>36.1</td>
</tr>
<tr>
<td>LW Truck - Chevy Suburban</td>
<td>33</td>
<td>8</td>
<td>7.75</td>
<td>48.7</td>
<td>62.0</td>
<td>4914</td>
<td>1228.5</td>
<td>25.2</td>
<td>19.8</td>
</tr>
<tr>
<td>LW Truck - Chevy Suburban</td>
<td>41</td>
<td>7.5</td>
<td>6.375</td>
<td>37.5</td>
<td>47.8</td>
<td>4914</td>
<td>1228.5</td>
<td>32.7</td>
<td>25.7</td>
</tr>
<tr>
<td>Car - Honda Civic</td>
<td>33</td>
<td>5.75</td>
<td>5.75</td>
<td>26.0</td>
<td>33.1</td>
<td>2680</td>
<td>670</td>
<td>25.8</td>
<td>20.3</td>
</tr>
<tr>
<td>MMLS3 (heavy)</td>
<td>90</td>
<td>2.9</td>
<td>2.9</td>
<td>6.6</td>
<td>8.4</td>
<td>n/a</td>
<td>607</td>
<td>91.9</td>
<td>72.2</td>
</tr>
<tr>
<td>MMLS3 (light)</td>
<td>65</td>
<td>3.5</td>
<td>2.95</td>
<td>8.1</td>
<td>10.3</td>
<td>n/a</td>
<td>461</td>
<td>56.88</td>
<td>44.65</td>
</tr>
</tbody>
</table>

28.8

80.9

50.0
Stresses were modeled using Kenlayer, a pavement analysis program used to analyze stresses in a pavement (Huang, 2004). It was used to analyze a depth where the stresses from the MMLS3 loading could be negligible. Input variables included material depths, modulus of elasticity, Poisson’s ratio, tire contact radius, and tire pressure. The depth of the surface layer, DSA or PennDOT 2A, was six inches, and the subgrade depth was modeled as infinite. Using modulus of elasticity values obtained from www.pavementinteractive.org, the modulus of elastic was assumed to be 30,000 psi (206.8 Mpa), for the surface aggregates and 4,500 psi (31 Mpa) for the subgrade. Based off values obtained from Hugo et al. (2007) and Hugo (2005) outlining the parameters of the MMLS3, the tire contact radius was assumed to be 1.3 inches and the pressure assumed to be 100 psi.

| Table 3-2: | Comparison of tire loads, contact areas, contact pressures, and tire pressures for the MMLS3 and passenger vehicle. |
|---|---|---|---|
| MMLS3 (Heavy) | MMLS3 (Light) | Passenger Vehicle |
| **Load (lbs)** | 607 | 461 | 670-1230 |
| **Contact Area (in²)** | 7.5 | 9.2 | 16-49 in² |
| **Contact Pressure (psi)** | 81 | 50 | 22-46 psi |
| **Tire Pressure (psi)** | 90 | 65 | 33-52.5 psi |

A modeled stress less than 1 psi was determined to be small enough to represent a negligible stress. The program was set to analyze the top of the subgrade down to a depth of sixty inches. Figure 3-3 shows the graph of the stress output. At this depth, the bottom of the box...
would not reinforce the pavement materials. A depth of fifteen inches was selected for the box depth.

![Stress in Sublayer](image)

**Figure 3-3**: Graphical representation of stress data from Kenlayer analysis assuming a 100psi contact pressure from the MMLS3 tire.

**Laboratory Testing**

The MMLS3 was utilized in the laboratory experiments for testing the aggregate surfaces. A laboratory road section was constructed with a subgrade classified as Clay of Low Plasticity (CL) in the Unified Soil Classification System (A-4(5)) in AASHTO (Tang, 2011). Two aggregate surfaces, DSA and PennDOT 2A, were placed and exchanged on top of the subgrade. The road materials were compacted in a wooden box structure built specifically for the testing. The change of the surface profile with increasing number of cycles was determined using the profilometer for DSA and a manual profiler for PennDOT 2A. The profilometer used on the DSA was broken at the time of the PenDOT 2A testing.

Both the profilometer and the profiler measure depth from a set point, or datum. The profilometer operates with a wheel that runs on a track transversally along the surface and
measures the depth from the wheel track to the point where the wheel touches. The profiler is a
digital caliper that measures the depth from a bar set transverse to the pavement as it is moved
manually across the pavement section. The profilometer and the profiler set-up are shown in
Figure 3-4.

Figure 3-4: The profilometer set-up used to measure the surface profile of the laboratory DSA
(shown) and field pavement sections (a) and the profiler set-up used to measure the laboratory
PennDOT 2A (b).

The profile was measured between predetermined intervals of cycles that changed with
increasing numbers of cycles. This measurement was performed at five cross sections along each
laboratory pavement test section and three cross sections for each field pavement test section, as
shown in Figure 3-5. The data output was a depth measurement (mm) in relation to a datum.
Straight-forward use of the profilometer and the profiler data provided surface profile of the
pavement as it changed with increasing number of cycles due to rutting and the pushing of
material. Further analysis provided a rut depth with respect to the number of cycles.
Surface Aggregates

Sieve analysis was used to find the gradation of the DSA and PennDOT 2A aggregate. Aggregate materials of different gradations were mixed onsite to obtain a gradation that closely matched the DSA gradation used by Bloser (2007). PennDOT 2A was ordered from Glenn O. Hawbaker’s limestone quarry in Centre County, Pennsylvania and checked for a gradation meeting the specification of PennDOT 2A. The gradation was also compared to the PennDOT 2A gradation used by Bloser (2007). ASTM D698 was used to find the optimum moisture content and the maximum dry density of the DSA and PennDOT 2A aggregate gradations used for the laboratory experiments. Construction specifications defined by The Center for Dirt and Gravel Road Studies and PennDOT for the respective aggregate material were followed.

Figure 3-5: MMLS3 testing setup in the laboratory (a) and in the field (b) marked with locations of surface profile measurements.
Driving Surface Aggregate

The MMLS3 was run on the DSA material at both a lightweight traffic load and heavy-traffic load for 100,000 passes each. An attempt was made to duplicate the gradation that was close to the gradation of the same material in the study completed by Bloser (2007). The DSA was placed in one eight-inch lift and compacted to six inches with a portable vibratory roller, as to follow construction specifications for the material as closely as possible.

Table 3-4 shows the percent passing on the various sieve sizes used to obtain a gradation for DSA. Percent passing for the control is the gradation that was used in the previous study and the percent passing for the experiment is the gradation used in this MMLS3 laboratory experiment. The target high and target low are the high and low values for percent passing according to the specification for DSA. Figure 3-6 is the graph of these gradations plotted on a linear-log scale.

Table 3-4: Comparison of the DSA aggregate gradation used in the lab testing box to the DSA aggregate gradation used in the control study by Bloser (2007). The high and low target specifications on each sieve for DSA can be determined.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>in</td>
</tr>
<tr>
<td>50.8</td>
<td>2”</td>
</tr>
<tr>
<td>38.1</td>
<td>1.5&quot;</td>
</tr>
<tr>
<td>19.1</td>
<td>3/4&quot;</td>
</tr>
<tr>
<td>9.525</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>4.75</td>
<td>#4</td>
</tr>
<tr>
<td>2.36</td>
<td>#8</td>
</tr>
<tr>
<td>1.18</td>
<td>#16</td>
</tr>
<tr>
<td>0.15</td>
<td>#100</td>
</tr>
<tr>
<td>0.075</td>
<td>#200</td>
</tr>
</tbody>
</table>
ASTM D698, standard proctor test, was performed on the DSA material in order to find the optimum moisture and maximum dry density of the DSA. Figure 3-7 shows the standard proctor results for the DSA used in the laboratory. A nuclear density gauge was used to evaluate the percent of maximum dry density and moisture after the material was placed. As shown in Figure 3-8b, measurements were taken 36 inches from the end of the box and 11.5 inches from the side of the box, at both ends of the testing box.

The nuclear density gauge measurement was compared to the maximum dry density of 145 lb/ft$^3$ at an optimum moisture content of 6.7% for the DSA. The compaction in the pavement section was at 101.4% of the maximum dry density compaction at 6.5% moisture content at testing location number one and 98.9% of the maximum dry density compaction with 6.7% moisture content at testing location number two.
Figure 3-7: Standard proctor test results for the DSA used in the laboratory pavement section.

Figure 3-8: Location of nuclear density gauge measurements to determine the percent compaction for the subgrade (a), DSA (b), PennDOT 2A – Lightweight traffic setting (b), and PennDOT 2A – Heavy traffic setting (d). (Schematic is not to scale.)
Since the distress from the lightweight MMLS3 test was minimal (shown in Figure 3-9) the DSA did not have to be removed between testing the lightweight traffic load and the heavy traffic load. Instead, the MMLS3 was displaced six inches to the side of the original testing. Upon removal of the material, the subgrade was checked for rutting and it was noted that there was no deformation in the subgrade and it was able to be reused for PennDOT 2A testing. Figure 3-9b shows rutting after the MMLS3 testing with heavy traffic loading. The rutting did not deform the subgrade.

Figure 3-9: Rutting after 100,000 cycles of the MMLS3 on DSA at the lightweight traffic setting (a) and a cross section of rutting in DSA from the MMLS3 after heavy traffic loading (b).
**PennDOT 2A**

The MMLS3 was run on the PennDOT 2A material at a lightweight traffic load for 100,000 passes. It was run on a heavy-traffic load for 30,000 passes and stopped due to a limitation with the MMLS3 being unable to provide an accurate load due to the depth of the formed rut. The PennDOT 2A aggregate gradation used by Bloser (2007) was a DSA gradation with 5% clay mineral fines added to the mix. This was explained as possibly changing the normal performance of the PennDOT 2A; therefore, it was not duplicated for this study and the PennDOT 2A material was ordered from a quarry. The material was placed in one lift and hand-tamped for initial compaction. Final compaction was completed by running the MMLS3 on lateral wander for 500 cycles. The aggregate had to be dried overnight before final compaction in order to avoid pumping. Compaction was performed as to follow construction specifications for tailgated PennDOT 2A, in which the material is compacted by vehicles driving over the material. (Bloser, 2007)

Table 3-5 shows the percent passing on the various sieve sizes used to obtain a gradation for the PennDOT 2A. Percent passing for the control is the gradation that was used in a Bloser (2007) study and the percent passing for the experiment is the gradation used in the MMLS3 laboratory experiment. The target high and target low are the high and low values for percent passing according to the specification for PennDOT 2A. Figure 3-10 is the graph of these gradations plotted on a linear-log scale.

---

**Table 3-5:** Comparison of the PennDOT 2A aggregate gradation used in the laboratory testing box to the PennDOT 2A aggregate gradation used in the control study by Bloser (2007). The high
Similarly to DSA, ASTM D698, standard Proctor test, was performed on the PennDOT 2A material in order to find the optimum moisture and maximum dry density. Figure 3-11 shows the standard proctor results for the PennDOT 2A used in the laboratory. The maximum dry density was 139.7 lb/ft$^3$ and the optimum moisture for the gradation was 7.8%. Construction specifications for PennDOT 2A do not require it to be placed at optimum moisture.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
<th>Control</th>
<th>Experiment</th>
<th>Target Low</th>
<th>Target High</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>in</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50.8</td>
<td>2&quot;</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>38.1</td>
<td>1.5&quot;</td>
<td></td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19.1</td>
<td>3/4&quot;</td>
<td>85</td>
<td>87.8</td>
<td>52</td>
<td>100</td>
</tr>
<tr>
<td>9.525</td>
<td>3/8&quot;</td>
<td>55</td>
<td>65.8</td>
<td>36</td>
<td>70</td>
</tr>
<tr>
<td>4.75</td>
<td>#4</td>
<td>34</td>
<td>45.1</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>2.36</td>
<td>#8</td>
<td>24</td>
<td>30.0</td>
<td>16</td>
<td>38</td>
</tr>
<tr>
<td>1.18</td>
<td>#16</td>
<td>14</td>
<td>20.7</td>
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<td>30</td>
</tr>
<tr>
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<td>9</td>
<td>8.0</td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>0.075</td>
<td>#200</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3-10: Aggregate gradation for PennDOT 2A. Solid line is the result of the sieve analysis performed on the aggregate used in the experimental road section in the testing box. The control indicates the gradation used in comparison study.
The PennDOT 2A for lightweight traffic was placed in the testing box near optimum moisture at 6.5% moisture. The nuclear density gauge measured the density at 93.2% of the maximum dry density at location number one of the compacted area and at 93.5% of the maximum dry density at location number 2. The locations for the nuclear density gauge readings are shown in Figure 3-8c. Due to extent of the distress caused by the MMLS3 during the lightweight traffic loading and the test needing to be run in the same location, the aggregate had to be removed and re-compacted. The same procedure used for lightweight traffic loading was used for the heavy traffic testing. The moisture content at the time of the PennDOT 2A placement for heavy traffic testing was aimed to be at optimum moisture; however, the scale used to measure the weight did not show enough significant digits and the moisture was shown not to change. The material was placed wet and the sample weighed 0.9lbs. The estimated moisture content was between 5.3 - 7.2%. Figure 3-12 shows the rutting after the lightweight traffic loading and depth of the aggregate affected by the rut formation. The subgrade was not affected during the lightweight traffic loading on PennDOT 2A and did not have to be replaced.
After final compaction for heavy traffic testing, the density of the tested section was at 91.0% of the maximum dry density at location number 1 of the compacted section and at 93.7% of the maximum dry density at location number 2. The locations for the nuclear density gauge readings are shown in Figure 3-8d.

Figure 3-12: Rutting after 100,000 cycles of the MMLS3 on PennDOT 2A at the lightweight traffic setting (a) and a cross section of rutting in PennDOT 2A from the MMLS3 (b).

Field Testing

Surface aggregate testing with the MMLS3, condition assessments, and stiffness measurements were completed on four roads located in Erie County, Pennsylvania as shown in Figure 3-13. Two of the roads were located in Waterford Township, one was located in Union City, and was one located in Albion. The untreated road (Benson Road) and the cement-stabilized full-depth reclamation (FDR) road (Himrod Road) were located in Waterford Township. The DSA road (Church Road) was located in Union City and the cement-stabilized FDR road with a tar and chip surface treatment (Reservoir Road) was located in Albion. All four roads were located within thirty miles of one another.
The condition assessments and the stiffness testing on the four roads were used to evaluate the performance of full-depth reclamation on unpaved roads. Pavement testing on a DSA and an Ohio base material with specifications that meet that of a PennDOT 2A aggregate base was completed in the field with the MMLS3 in order to determine performance of these aggregate surfaces when exposed to more natural conditions. An Ohio base material was used due to the location of the quarry typically being used by Erie County.

**Surface Aggregates**

Sieve analysis was performed on the aggregate used in the field studies in order to check the variation in the material, however there was less control over the gradation than that used in the laboratory. This part of the experiment was used as a check to explain any unusual behavior of the surface aggregate. Table 3-6 and Figure 3-14 show the sieve analysis of DSA in comparison with the specifications and with the aggregate used in Bloser (2007).
Table 3-6: Comparison of the DSA aggregate blend used in the field to the aggregate gradation used in the control study by Bloser (2007). The high and low target specifications on each sieve for DSA are defined.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>in</td>
</tr>
<tr>
<td>50.8</td>
<td>2&quot;</td>
</tr>
<tr>
<td>38.1</td>
<td>1.5&quot;</td>
</tr>
<tr>
<td>19.1</td>
<td>3/4&quot;</td>
</tr>
<tr>
<td>9.525</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>4.75</td>
<td>#4</td>
</tr>
<tr>
<td>2.36</td>
<td>#8</td>
</tr>
<tr>
<td>1.18</td>
<td>#16</td>
</tr>
<tr>
<td>0.15</td>
<td>#100</td>
</tr>
<tr>
<td>0.075</td>
<td>#200</td>
</tr>
</tbody>
</table>

Figure 3-14: Aggregate gradation for the DSA used in the field (solid line) in comparison with the gradation used in Bloser (2007) and the high and low target values for the specification.

Table 3-7 and Figure 3-15 show the sieve analysis and specification of the PennDOT 2A material used in the field, in comparison with a gradation used in a previous study on surface aggregates.
Table 3-7: Comparison of the PennDOT 2A used in the field to the aggregate gradation used in the control study by Bloser (2007). The high and low target specifications on each sieve for DSA are defined.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td>in</td>
</tr>
<tr>
<td>50.8</td>
<td>2&quot;</td>
</tr>
<tr>
<td>38.1</td>
<td>1.5&quot;</td>
</tr>
<tr>
<td>19.1</td>
<td>3/4&quot;</td>
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<td>#4</td>
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<tr>
<td>2.36</td>
<td>#8</td>
</tr>
<tr>
<td>1.18</td>
<td>#16</td>
</tr>
<tr>
<td>0.15</td>
<td>#100</td>
</tr>
<tr>
<td>0.075</td>
<td>#200</td>
</tr>
</tbody>
</table>

Figure 3-15: Aggregate gradation for the PennDOT 2A used in the field (solid line) in comparison with the gradation used in Bloser (2007) and the high and low target values for the specification.
**Condition Assessments and Stiffness Testing**

Four roads in Erie County, Pennsylvania were evaluated for material properties and condition. Table 3-8 provides general information on the roads. A lightweight deflectometer (LWD) was used to measure the elastic modulus, and a condition assessment following the procedure outlined by the US Army Corp of Engineers, Assessment of Unpaved Roads (1995) was performed to rate the “drivability” of roads throughout the year.

Table 3-8: Location and information of the four roads evaluated for pavement stiffness and road condition.

<table>
<thead>
<tr>
<th>Road</th>
<th>Location</th>
<th>Surface</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Benson</td>
<td>Waterford, PA</td>
<td>Natural</td>
<td>0.8 mi</td>
</tr>
<tr>
<td>Himrod</td>
<td>Waterford, PA</td>
<td>Full Depth Reclamation</td>
<td>1.4 mi</td>
</tr>
<tr>
<td></td>
<td></td>
<td>with cement stabilization</td>
<td></td>
</tr>
<tr>
<td>Church</td>
<td>Union City, PA</td>
<td>Driving Surface Aggregate</td>
<td>0.6 mi</td>
</tr>
<tr>
<td>Reservoir</td>
<td>Albion, PA</td>
<td>Full Depth Reclamation</td>
<td>1.0 mi</td>
</tr>
<tr>
<td></td>
<td></td>
<td>with cement stabilization and</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>tar and chip surface treatment</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3-13 shows the location of the four roads studied in Erie County, Pennsylvania. Benson Road was the control road having a natural (untreated) surface and Himrod Road was a full depth reclamation road (FDR) with cement stabilization. Both were located in Waterford Township. The DSA road was located in Union City and the FDR road with a tar and chip surface was located in Albion. Tar and chip is an asphalt surface treatment where a thin layer of asphalt is applied to the road and covered with aggregate (Yoder and Witczak, 1975). The surface-treated roads were compared to the control road without a surface treatment. The tar and chip surface was included to compare how well the FDR with cement stabilization performed with and without a seal. The DSA road was used to obtain knowledge of materials and performance for a road in between the control road and the FDR road without a surface.
LWD measurements were taken once every month in the spring months, from February to May, in order to capture the effect of spring thaw on the pavement systems. Measurements were also taken once during the summer months and once during the fall months. Measurements with the LWD are not possible in winter months due to the presence of ice and snow. The device was dropped five times at approximately every quarter-mile on each of the roads. Data from the LWD testing and the condition assessments were evaluated and analyzed both separately and in conjunction with each other.
Chapter 4

Results

The results from the MMLS3 testing on surface aggregates and the stiffness and condition assessment results from the field tests were evaluated and are presented separately in the following sections. R² values and rut depths from surface measurements taken during and after the MMLS3 testing were compared. Data from the field testing, the Impact Stiffness Modulus (ISM) and the condition ratings, were also compared.

MMLS3 – Aggregate Testing

Five cross section measurements of the surface profile were taken after each set of MMLS3 cycles; starting at the 200mm mark of the wheel path length and taken every 150mm until 800mm was reached as shown in Figure 3-5. The cross section measurements were taken with the profilometer for DSA and with a manual profiler for PennDOT 2A. Both the profilometer and the profiler measure depth from a set point, or datum. The profilometer operates with a wheel that runs on a track transversally along the surface and measures the depth from the wheel track to the point where the wheel touches. The profiler is a digital caliper that measures the depth from a bar set transverse to the pavement as it is moved manually across the pavement section. The profilometer (profiler) set-up is shown in Figures 3-4 and 3-5. The surface profiles of one profilometer (profiler) cross section position for DSA and PennDOT 2A are shown in Figures 4-1, 4-4, and 4-7.
A rut depth was calculated from the surface profile measurements. The depth across the wheel path was averaged for each cycle, each profilometer (profiler) cross section position, and material. The average depths at each cycle were subtracted from the initial reading (0 cycles) to find the rut depth with respect to the number of cycles as shown in Figures 4-2, 4-5, and 4-8. The rut depth for all profilometer (profiler) cross section positions are shown in Figures 4-3, 4-6, and 4-8.
4-9. “Heavy” refers to the MMLS3 at the heaviest setting of 607lbs (2.7kN) and a tire pressure of 90 psi. “Light” refers to the MMLS3 setting at the lightest load of 461lbs (2.05kN) and a wheel pressure of 65 psi. “Natural” refers to the MMLS3 testing on the aggregates placed in the field. Appendix B contains the full results from the MMLS3 testing on the aggregate surfaces.

The rut depth calculations were expected to develop a consistent trend (i.e. logarithmic trend). In this case, it was anticipated that majority of the rutting would occur in the first half of the cycles and eventually “even-out” as the number of cycles increased.

Figure 4-2: Rut depths on DSA (a) and PennDOT 2A (b) calculated from the surface profile data.
When plotted together, it is apparent that the rut depth varied between the profilometer (profiler) cross section positions from 200mm to 800mm while the trends may or may not have remained similar. This is shown in Figure 4-3.

Figure 4-3: Resulting rut depths of all five profilometer (profiler) cross sections from MMLS3 lightweight traffic loading on DSA (a) and PennDOT 2A (b).
The process of plotting the surface profile and the rut depth vs. the number of cycles was repeated for the MMLS3 heavy traffic testing and testing in the field. The graphs are shown in Figures 4-4 to 4-9.

Figure 4-4: Surface profiles from the MMLS3 heavy traffic loading on Driving Surface Aggregate (DSA) (a) and on PennDOT 2A (b), at the 650 mm profilometer (profiler) position.
Figure 4-5: Rut depths on DSA (a) and PennDOT 2A (b) calculated from the surface profile data.
Figure 4-6: Resulting rut depths of all five profilometer (profiler) cross sections from MMLS3 heavy traffic loading on DSA (a) and PennDOT 2A (b).
Figure 4-7: Surface profiles from the field testing of the MMLS3 lightweight traffic loading on DSA (a) and on PennDOT 2A (b), at the 350 mm profilometer (profiler) position.
Figure 4-8: Rut depths on DSA (a) and PennDOT 2A (b) calculated from the surface profile data.

Erie DSA - Natural 2 (350)

y = 1.3468ln(x) - 7.0006
R² = 0.9678

PennDOT 2A - Natural 2 (350)

y = 0.4651ln(x) + 10.584
R² = 0.1052
Figure 4-9: Resulting rut depths of three profilometer (profiler) cross sections from the two days of field testing with MMLS3 lightweight traffic loading on DSA (a) and PennDOT 2A (b).
A logarithmic trend line was fit to evaluate the calculated rut depth data. Table 4-1 compares the $R^2$ values of the DSA and the PennDOT 2A used in the laboratory testing. Table 4-2 compares the $R^2$ values of the logarithmic trend line for the DSA and PennDOT 2A used in the field.

Table 4-1: The $R^2$ values from the logarithmic regression fit for number of cycles vs. rut depth. The five cross section positions (as depicted in Figure 3-5) are recorded for both aggregate surfaces, DSA and PennDOT 2A used in the laboratory testing.

<table>
<thead>
<tr>
<th>LABORATORY</th>
<th>Position</th>
<th>$R^2$ Value</th>
<th>DSA</th>
<th>PennDOT 2A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lightweight Traffic</td>
<td>200</td>
<td>0.2707</td>
<td>0.9590</td>
<td></td>
</tr>
<tr>
<td></td>
<td>350</td>
<td>0.4190</td>
<td>0.0049</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>0.4178</td>
<td>0.3232</td>
<td></td>
</tr>
<tr>
<td></td>
<td>650</td>
<td>0.1695</td>
<td>0.0052</td>
<td></td>
</tr>
<tr>
<td></td>
<td>800</td>
<td>0.4328</td>
<td>0.5026</td>
<td></td>
</tr>
<tr>
<td>Heavy Traffic</td>
<td>200</td>
<td>-</td>
<td>0.8298</td>
<td></td>
</tr>
<tr>
<td></td>
<td>350</td>
<td>0.3159</td>
<td>0.8525</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>0.5130</td>
<td>0.8973</td>
<td></td>
</tr>
<tr>
<td></td>
<td>650</td>
<td>0.8994</td>
<td>0.9079</td>
<td></td>
</tr>
<tr>
<td></td>
<td>800</td>
<td>0.9358</td>
<td>0.0802</td>
<td></td>
</tr>
</tbody>
</table>

The $R^2$ value is related to how well the regression equation fits the data. A number closer to 1.0 is correlated with good predictability of the pavement rutting. A lower value indicated a less reliable fit of the regression equation and therefore, the pavement rutting is less predictable.
Some sections of DSA surface measurements showed faulty profilometer readings and were removed when evaluating the data. Profile measurement of the test section loaded with heavy traffic on DSA at the 200mm position had to be removed altogether due to a technical issue with the profilometer. The 1000 and 2000 cycle measurements had to also be removed at the 350mm position for the DSA on the heavy traffic setting for the same reason. The initial reading of the lightweight setting at 800 mm position was also removed from the DSA data. Removing this cycle set showed a better representation of the surface profile and led to a better fit of the logarithmic trend line.

In addition to finding a potential trend for the rutting formation in the unbound aggregates, the rut depth was also noted. Table 4-3 shows the final rut depths for the MMLS3 laboratory testing at each profilometer (profiler) cross section position. Some cross sections ended with a final rut depth that was not the maximum throughout the test. The maximum rut depths, if different from final rut depth, are shown in parenthesis in Table 4-3. The rut depth at the 650 mm profiler position was calculated as a negative number (shown in Figure 4-3b) and

Table 4-2: The $R^2$ values from the logarithmic regression fit for number of cycles vs. change in rut depth. The three profilometer cross section positions (as depicted in Figure 3-5) are recorded for both aggregate surfaces, DSA and PennDOT 2A used in the field testing.

<table>
<thead>
<tr>
<th>Position</th>
<th>$R^2$ Value</th>
<th>$R^2$ Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>0.6689</td>
<td>0.9846</td>
</tr>
<tr>
<td>350</td>
<td>0.6898</td>
<td>0.9975</td>
</tr>
<tr>
<td>500</td>
<td>0.5617</td>
<td>0.9942</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Position</th>
<th>$R^2$ Value</th>
<th>$R^2$ Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>0.9515</td>
<td>0.0724</td>
</tr>
<tr>
<td>350</td>
<td>0.9678</td>
<td>0.1025</td>
</tr>
<tr>
<td>500</td>
<td>0.9867</td>
<td>0.2398</td>
</tr>
</tbody>
</table>

Some sections of DSA surface measurements showed faulty profilometer readings and were removed when evaluating the data. Profile measurement of the test section loaded with heavy traffic on DSA at the 200mm position had to be removed altogether due to a technical issue with the profilometer. The 1000 and 2000 cycle measurements had to also be removed at the 350mm position for the DSA on the heavy traffic setting for the same reason. The initial reading of the lightweight setting at 800 mm position was also removed from the DSA data. Removing this cycle set showed a better representation of the surface profile and led to a better fit of the logarithmic trend line.

In addition to finding a potential trend for the rutting formation in the unbound aggregates, the rut depth was also noted. Table 4-3 shows the final rut depths for the MMLS3 laboratory testing at each profilometer (profiler) cross section position. Some cross sections ended with a final rut depth that was not the maximum throughout the test. The maximum rut depths, if different from final rut depth, are shown in parenthesis in Table 4-3. The rut depth at the 650 mm profiler position was calculated as a negative number (shown in Figure 4-3b) and
therefore removed due to a possible error in the initial reading. Table 4-4 shows the rut depth results of the field testing.

Table 4-3: Final laboratory rut depths measured with the profilometer and the profiler and averaged across the wheel path of the MMLS3. DSA was run to 100,000 cycles for both lightweight traffic and heavy traffic testing. PennDOT 2A was run to 100,000 cycles for lightweight traffic testing and to 30,000 cycles for heavy traffic testing.

<table>
<thead>
<tr>
<th>LABORATORY</th>
<th>Position</th>
<th>Rutting (mm)</th>
<th>DSA</th>
<th>PennDOT 2A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lightweight Traffic</td>
<td>200</td>
<td>1.00</td>
<td>15.50 (17.00)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>350</td>
<td>1.50</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>1.50</td>
<td>1.50 (2.00)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>650</td>
<td>1.75</td>
<td>1.50 (-1.0)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>800</td>
<td>2.00</td>
<td>6.00 (7.00)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Heavy Traffic</th>
<th>Position</th>
<th>Rutting (mm)</th>
<th>DSA</th>
<th>PennDOT 2A</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>200</td>
<td>-</td>
<td>15.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>350</td>
<td>12.25</td>
<td>10.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>2.50</td>
<td>9.50</td>
<td></td>
</tr>
<tr>
<td></td>
<td>650</td>
<td>2.75</td>
<td>12.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>800</td>
<td>5.25</td>
<td>3.5 (1.75)</td>
<td></td>
</tr>
</tbody>
</table>

Table 4-4: Measurement of rut depths as a result of the MMLS3 wheel path during operation in the field. Measurements were taken with the profilometer and averaged across the wheel path. DSA was run to 31,300 cycles on day 1 and to 31,500 cycles on day 2. PennDOT 2A was run to 19,500 cycles on day 1 and to 21,500 cycles on day 2.

<table>
<thead>
<tr>
<th>FIELD</th>
<th>Day 1</th>
<th>Position</th>
<th>Rutting (mm)</th>
<th>DSA</th>
<th>PennDOT 2A</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>200</td>
<td>0.50</td>
<td>9.75</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>350</td>
<td>1.00</td>
<td>10.75</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>0.50</td>
<td>9.25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Day 2</th>
<th>Position</th>
<th>Rutting (mm)</th>
<th>DSA</th>
<th>PennDOT 2A</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>200</td>
<td>10.00</td>
<td>15.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>350</td>
<td>6.75</td>
<td>17.75</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>5.50</td>
<td>13.00</td>
<td></td>
</tr>
</tbody>
</table>
It is important to note the difference in the number of cycles when comparing the rut depths. The heavy traffic loading on the PennDOT 2A test had to be ended at 30,000 cycles instead of 100,000 due to the rut extending to a depth where the MMLS3 could no longer apply the calibrated load. The field testing of DSA was stopped at 31,300 cycles on day 1 and 31,500 cycles on day 2. The field testing for PennDOT 2A was stopped at 16,500 cycles on day 1 and 21,500 cycles on day 2. It is also important to note that the field tests were not completed on the same days, although similar weather conditions were in interest when selecting the testing dates. Weather for all field testing is documented in Appendix C.

Next, the final rut depths from each position were averaged to find a total average rut depth for each MMLS3 setting. Overall average rut depths are summarized in Table 4-5. The rut depths were used for comparing the performance between DSA and PennDOT 2A. There is not enough data correlating the rut from the MMLS3 testing and full-scale testing; therefore, the MMLS3 rut depths cannot be converted into a predicted full-scale rut depth.

Table 4-5: Overall average rut depth for both materials at the three MMLS3 test settings.

<table>
<thead>
<tr>
<th>MMLS3 SETTING</th>
<th>Cycles (DSA/PennDOT 2A)</th>
<th>RUTTING (MM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lab – Light Setting</td>
<td>100,000/100,000</td>
<td>1.55</td>
</tr>
<tr>
<td>Lab – Heavy setting</td>
<td>100,000/30,000</td>
<td>5.69</td>
</tr>
<tr>
<td>Field – Light setting</td>
<td>31,500/21,500</td>
<td>7.42</td>
</tr>
</tbody>
</table>

The results of the overall average rut depths from the lightweight testing in the laboratory and day 2 in the field were compared to the previous study by Bloser (2007). Table 4-6 shows the average rut depth for the MMLS3 in the laboratory, in the field, and from the study completed by Bloser (2007). The aggregates tested in the laboratory were not exposed to the same factors as the aggregates in the field and resulted in much lower rut depths. The most comparable rut
depths are those found between the MMLS3 field testing and the previous full-scale study by Bloser (2007). The number of cycles is important to consider when comparing the rut depths. The roads studied by Bloser (2007) were subjected to approximately 65,000 vehicles (comparable to 32,000 MMLS3 cycles) since the lightweight traffic loading was set to mimic passenger vehicle tires and not ESALs.

Table 4-6: Total rut depths from the lightweight laboratory testing and the field testing compared to results from Bloser (2007).

<table>
<thead>
<tr>
<th>MMLS3 SETTING</th>
<th>RUTTING (MM)</th>
<th>Cycles (DSA/PennDOT 2A)</th>
<th>DSA</th>
<th>PennDOT 2A</th>
</tr>
</thead>
<tbody>
<tr>
<td>MMLS3 (Laboratory)</td>
<td></td>
<td>100,000/30,000</td>
<td>1.55</td>
<td>6.88</td>
</tr>
<tr>
<td>MMLS3 (Field)</td>
<td></td>
<td>31,500/21,500</td>
<td>7.42</td>
<td>15.25</td>
</tr>
<tr>
<td>Bloser (2007)</td>
<td></td>
<td>~ 32,500 (65,000veh)</td>
<td>10.92</td>
<td>15.24</td>
</tr>
</tbody>
</table>

**Condition Assessments and Stiffness Testing**

Condition assessments were performed according to the US Army Corp of Engineers Manual for Unsurfaced Road Maintenance (1996). An assessment was completed once a month from February to May, once in the summer (August) and once in the fall (October). Assessments were taken on sections of road that were best representative of the overall condition of the road. Two assessments were taken per mile of road. Table 4-7 shows the average values from each road for each assessment. Condition ratings above 85 are considered to be in “Excellent” condition where conditions between 70 and 85 are considered to be in “Very Good” condition. Ratings from 55 to 70 are considered to be in “Good” condition.
Figures 4-10, 4-13, 4-17, and 4-20 depict the results from the individual condition assessments for each road. The most common form of distress was rutting and pot holes. The results from the condition assessments were used in conjunction with the stiffness testing to determine the effect of seasons on each road and therefore road materials.

The stiffness of four roads with varying surface materials was measured with the lightweight deflectometer (LWD) approximately every quarter-mile along the road. The stiffness of a road indicated how well the road could handle the stresses from traffic loading. Since the spring-thaw typically lowers the stiffness of the subgrade, measurements were taken once a month from February to May. Measurements were also taken once during the summer (August) and once during the fall (October) to obtain knowledge on the stiffness of the road outside the spring-thaw period. The LWD was dropped five times at each location. The first drop of the LWD weight was the seating drop and therefore discarded when averaging the results. The result from each drop was divided by the force of the falling weight to get the impact stiffness modulus (ISM). The ISM was used to compare the results from each test location.

Figures 4-12, 4-16, 4-19, and 4-22 show the average ISM results taken along each road. Snow was present on most of Benson Road in February 2011 and therefore a stiffness measurement was only able to be taken at one location along the road. Appendix A contains the average stiffness results for the LWD drops at each location.
Reservoir Road (Full-Depth Reclamation with Tar and Chip Surfacing)

The average condition rating for Reservoir Road varied between 69 and 92.5 throughout the duration of the study. The largest contributor to a decrease in condition rating was improper drainage.

Distresses observed on Reservoir Road remained apparent during the duration of the study. Maintenance was performed between May and summer (August) to fill in potholes and deep corrugations. The lowest condition assessment was in April and the best condition was in summer, after maintenance had been performed to fill in deep potholes and corrugations.

![Reservoir Road](image)

Figure 4-10: Average condition ratings observed once a month from February to May, once in the summer (August) and once in the fall (October). Ratings were taken from two locations along Reservoir Road.

The stiffness on Reservoir Road remained consistent throughout the study period with the exception of location 1, which had high variability and may have raised the average of the stiffness along the road for the February, March, and summer runs. Figure 4-11 shows the calculated ISM results from the LWD dropped along Reservoir Road.
The results of the LWD from each location were averaged together and the standard deviation was calculated. Figure 4-12 shows the average and standard deviation of ISM results along Reservoir road.

Figure 4-11: Results of the lightweight deflectometer for Reservoir Road (FDR with tar and chip surface) operated monthly from February to May, once in the summer (August), and once in the fall (October).

Figure 4-12: Averaged results and standard deviation of the lightweight deflectometer for Reservoir Road (FDR with tar and chip surface). Measurements were taken once a month from February to May, once in the summer (August) and once in the fall (October).
Himrod Road (Full-Depth Reclamation with Cement-Stabilization)

The condition assessments for Himrod Road remained between 70 and 80 throughout the duration of the study period. (February - October). Condition assessments above 85 are considered “Excellent” and assessments between 70 and 85 are considered to be in “Very Good” condition. The most notable forms of distress having the greatest influence on the condition are rutting and improper cross section. Figure 4-13 conveys the average condition ratings for Himrod Road.

![Average condition ratings for Himrod Road](image)

Figure 4-13: Average condition ratings observed once a month from February to May, once in the summer (August) and once in the fall (October). Ratings were taken at three locations along Himrod Road.

Himrod Road was in good condition for the majority of the road but it had a few sections that were not typical with the rest, and in particularly poor condition. It was noted that the soil-cement was not visible in these sections. Figure 4-14 is a picture of a suspected FDR failure taken along Himrod Road.
The ISM values vary considerably along the length of the road and the variability is confirmed by the calculation of the standard deviation of the measurements from the average for the length of the road. Figures 4-15 and 4-16 show the calculated ISM values for each test location and the average values along the road for each month, respectively. The ISM values tended to be high in February and reached the lowest averages from March through May, with the lowest absolute value for the road being 0.04 measured in March.

Figure 4-14: A section of Himrod Road experienced potholes, rutting, and poor drainage during a March 2011 spring thaw. (severity is not typical of the entire road)

![Himrod Road](image)

Figure 4-15: Results of the lightweight deflectometer for Himrod Road (FDR) operated monthly from February to May, once in the summer (August), and once in the fall (October).
Church Road (Driving Surface Aggregate)

The condition ratings for Church Road remained between “Excellent” and “Good” condition throughout the duration of the study period. (February - October). Condition assessments above 85 are considered “Excellent” and assessments between 70 and 85 are considered to be in “Very Good” condition. The lowest condition rating was observed in May and the highest was observed in the fall. The most notable forms of distress having the greatest effect on the condition are potholes and rutting. Figure 4-12 conveys the average condition ratings for Church Road.

Figure 4-16: Averaged results and standard deviation of the lightweight deflectometer for Himrod Road (FDR). Measurements were taken once a month from February to May, once in the summer (August) and once in the fall (October).
The ISM values for Church Road were the lowest in March on average and remained relatively low through May. The lowest absolute value was 0.01 recorded in both March and May. Figures 4-18 and 4-19 show the calculated ISM values for each test location and the average values along the road for each month, respectively.

Figure 4-17: Average condition ratings observed once a month from February to May, once in the summer (August) and once in the fall (October). Ratings were taken at two locations along Church Road.

Figure 4-18: Results of the lightweight deflectometer for Church Road (DSA) operated monthly from February to May, once in the summer (August), and once in the fall (October).
The condition ratings for Benson Road improved and then leveled-off during the duration of the study. Benson Road started with an average condition rating of 65, which is considered to be “Good” but improved to “Very Good” with condition ratings between 70 and 85 from March through the fall. The most notable forms of distress having the greatest influence on the condition were potholes and rutting. Figure 4-20 conveys the average condition ratings for Benson Road.

Figure 4-19: Averaged results and standard deviation of the lightweight deflectometer for Church Road (DSA). Measurements were taken once a month from February to May, once in the summer (August) and once in the fall (October).
The lowest average and absolute ISM values were calculated for the April and May stiffness measurements. The lowest average value calculated was 0.01 and the lowest average ISM value calculated was 0.023. Figures 4-21 and 4-22 show the calculated ISM values for each test location and the average values along the road for each month, respectively.

![Benson Road Condition Ratings](image1)

**Figure 4-20**: Average condition ratings observed once a month from February to May, once in the summer (August) and once in the fall (October). Ratings were taken at two locations along Benson Road.

![Benson Road ISM Values](image2)

**Figure 4-21**: Results of the lightweight deflectometer for Benson Road (natural surface) operated monthly from February to May, once in the summer (August), and once in the fall (October).
Figure 4-22: Average results and standard deviations of the lightweight deflectometer for Benson Road (natural surface). Measurements were taken once a month from February to May, once in the summer (August) and once in the fall (October).
Chapter 5

Discussion

MMLS3 Testing Discussion

Laboratory tests using the MMLS3 to simulate traffic were conducted on laboratory pavement sections at a lightweight traffic setting, at a heavy traffic setting, and at a lightweight traffic setting in the field. Surface profile measurements were taken at five distances along the MMLS3 wheel path in the laboratory and at three distances along the wheel path in the field. Rut depths were calculated from the surface profile measurements and then averaged. The tests were performed on both DSA and PennDOT 2A aggregates.

For lightweight traffic in the laboratory, DSA was very minimally affected by the traffic loading and showed slight aggregate loss of smaller particles. The PennDOT 2A at the light setting showed some compaction paired with aggregate loss of both small and large particles. The resulting rut formation from the lightweight traffic testing in the laboratory is shown in Figure 5-1. \( R^2 \) values remained low for DSA with values that ranged from a 0.2707 – 0.4328. PennDOT 2A had a large range of \( R^2 \) values from a 0.0052 – 0.9590. Although the range was wide for PennDOT 2A, the cross section with the highest showed a deeper rut and more apparent compaction when related to the surface profile. Rut depths ranged from 1.0mm – 2.0mm (0.04in – 0.08in) for DSA and from 1.5mm – 17.0mm (0.06in – 0.66in) for PennDOT 2A. Average rut depths were 1.55mm (0.06in) and 6.88mm (0.27in) for DSA and PennDOT 2A, respectively.
For heavy traffic loading in the laboratory, both DSA and PennDOT 2A showed a large amount of aggregate loss and compaction. Figure 5-2 shows the resulting rut from 100,000 cycles on DSA and 30,000 cycles on PennDOT 2A. The rut became too deep in the PennDOT 2A for the MMLS3 to apply a calibrated load after 30,000 cycles. $R^2$ values ranged from 0.3159 – 0.9358 for DSA and from 0.0802 – 0.9079 for PennDOT 2A. Although the range was wide for PennDOT 2A, four out of five $R^2$ values were between 0.8298 and 0.9079. Rut depths in DSA ranged from 2.5mm – 12.25mm (0.1in – 0.48in) with an average of 5.96mm (0.22in). Rut depths on PennDOT 2A ranged from 3.5mm – 15.25mm (0.14in – 0.60in) and averaged 10.10mm (0.40in).
Field studies with the aggregates exposed to natural conditions showed that rut formation had a stronger logarithmic trend with $R^2$ values on day 2 of testing than on day 1. Day 1 of field testing on DSA had shown a slightly similar trend to the laboratory tests, where the rut depth formed inconsistently, but had an $R^2$ values ranging from 0.5617 – 0.6898. $R^2$ values from day 2 of DSA testing ranged from 0.9846 – 0.9975. It was also noted that day 2 of field testing on DSA had wetter conditions than on day 1 due to a recent rainstorm. Day 1 of field testing on PennDOT 2A had $R^2$ values that ranged from 0.9846 – 0.9975 and day 2 had values that ranged from 0.0724 – 0.2398. After evaluating the surface profiles associated with these lower $R^2$ values on PennDOT 2A, it was apparent that the majority of the rutting seemed to occur within the first 1000 cycles.

After testing the aggregates in the field it became apparent that moisture in the materials could have helped reduce aggregate loss and caused more compaction under MMLS3 loading. It is assumed that the field test sections contained more moisture than the laboratory test sections due to environmental exposure. This may have aided in producing higher $R^2$ values in the field.
Trend lines that showed the best fit ($R^2$ values closer to 1.0) had more apparent compaction when checked with the surface profiles. In profilometer (profiler) cross sections where compaction was not as apparent, such as lightweight traffic testing and day 1 field testing on DSA, the $R^2$ values tended to be consistently lower. It is possible that rutting due to compaction tended to follow a logarithmic trend whereas rutting due to loss of aggregate tended to be more sporadic with inconsistent change of rut values, but the $R^2$ values varied too much to draw a definite conclusion.

MMLS3 testing in the field was considered more consistent with real-life rutting distress on unbound aggregate roads than laboratory testing. Average rut depths in both materials were higher in the field than in the lightweight traffic testing in the laboratory. This is assumed to be caused by the presence of moisture available in the field. Average rut depths in the lightweight traffic laboratory testing and the field testing were compared with a previous study by Bloser (2007). The rut depths for PennDOT 2A were nearly the same, but the amount of vehicle passes associated with the MMLS3 field testing is approximately two-thirds the estimated vehicle passes that had traveled on the roadway used in the Bloser (2007) study. The average rutting for DSA in

Figure 5-3: Resulting rut from the MMLS3 lightweight traffic testing in the field on DSA (a) and PennDOT 2A (b).
the MMLS3 field testing is approximately two-thirds the rut depth measured on DSA in Bloser (2007) at approximately the same number of estimated vehicles passes on the Bloser (2007) DSA. Before any conclusion on this comparison can be made, future research is needed to determine the correlation between scaled accelerated testing and full-scale testing on unbound aggregate roads.

**LWD and Condition Assessment Discussion**

All of the roads had sections that varied from that of excellent condition to sections of fair or very good condition. The road that varied the most was Himrod Road which had sections without distress and sections with distress great enough to affect traffic through the area. The variability of stiffness measurements agree with the variability in condition along the length of the road.

Himrod and Reservoir Roads were constructed using full-depth reclamation with cement stabilization. Both roads showed a high value for impact stiffness modulus (ISM) in February, which lowered considerably for the following months. Reservoir Road showed improvement in ISM in the summer, but still remained lower than in February. The high ISM value could be due to either environment or measurement before deterioration of the road material.

Overall, Reservoir Road had consistently higher ISM values and therefore showed superior stiffness when compared with the other roads. Himrod Road also showed higher ISM values than the roads with untreated bases and surfaces. This can attest to the increased stiffness of the soil-cement compared to an untreated base; however, the results cannot be generalized over the entire stretch of the road.

The roads with untreated bases, Church Road and Benson Road, had a stiffness decrease that is typical during the spring-thaw period. For both roads, this occurred between March and
May. Reservoir Road and Himrod Road experienced a slightly more consistent stiffness throughout the duration of the spring months. This is compatible with the hypothesis that cement-treated bases are more resistant to freeze-thaw; however, the condition assessments did not necessarily match.

While most of the roads experienced some decrease in condition rating over the spring, it was most profound on Reservoir Road; the road with a sealed and treated base and the highest stiffness measurements. In the case of untreated surfaces and base material, the distresses have a chance to “even out” as the road becomes slightly stiffer and material can compact at or near optimum moisture. Severe rutting and potholing can become less severe as the materials around it compact and move. Maintenance is also easier to perform on these roads when necessary. In the case of treated base and surface material, the stiffer materials do not push around as much, leaving any distress, such as rutting and potholes, in the pavement until maintenance is performed. With that, the distresses can continually degrade over a period of time. Figure 5-4 shows the degradation of a single distress (corrugation) on Reservoir Road from February to May. Maintenance was performed on this distress between May and the summer evaluation completed in August.
Considering that some sections of Himrod Road remained “intact” while others sections were barely passable, some construction methods may have failed. In a typical FDR construction application, the cement base is sealed by some means, usually by an asphalt cement or tar and chip, protecting it from losing moisture. In the case of Himrod Road, the dust suppressant responsible for sealing the surface was photo sensitive and degraded with UV light, causing the cement hydration to stop before it could fully and continually react with the water. As with portland cement concrete, curing is also important in cement-treated bases in order to optimize the cement hydration and strength.

Calcium leaching is also likely to have occurred on Himrod Road. In an environment with available water and pumping action to force water into the system, the calcium hydroxide...
dissolved out of the soil-cement and left a weakened product to be subject to traffic. Figure 5-5 is a picture of possible calcium leaching next to intact hydrated cement-treated base.

Figure 5-5: A picture taken on Himrod Road showing a section with a vein of cement still intact alongside a vein of the road with no apparent cement.

Condition assessments can lack in comparison due to the amount of traffic each road experiences. Roads were selected based on location and surface, not based on average daily traffic (ADT). Although Himrod Road had sections in poorer condition than other roads, it also experienced more traffic than Church Road and Benson Road. Other factors that could have affected the condition of the roads include the amount of canopy, slope, type of traffic and proximity to source of water.
Chapter 6

Conclusions

The performance of unbound aggregate roads was evaluated by comparing two wearing course materials and evaluating full-depth reclamation (FDR) as a construction practice for unpaved road bases. The wearing course materials compared were Driving Surface Aggregate (DSA) and PennDOT 2A. A sealed FDR road base with tar and chip was compared to an unsealed FDR road base with no wearing course.

For wearing course comparison, DSA performed better than PennDOT 2A in both the laboratory settings and in natural conditions under lightweight traffic conditions. Rut depths in DSA were consistently smaller than in PennDOT 2A. Results from this study were comparable to a previous study on the unbound aggregate road materials by Bloser (2007). These results could be due to the construction practice commonly used with PennDOT 2A, which does not require a vibratory roller and does not have to meet a density specification due to its nature as a base material for paved roads. Performance of the PennDOT 2A could also be affected by typically having less material passing the #200 sieve than DSA which would also result in a less dense wearing course. Due to the consistent better performance in this comparison, more strict specifications and sole use as a wearing course material, it is recommended that DSA be considered over PennDOT 2A for use on an unbound aggregate road. Conclusions regarding the predictability of the rut formation in regards to aggregate loss and compaction could not be made confidently. Future research on unbound aggregate roads can include simultaneous scaled and full-scale tests on unbound aggregate roads to evaluate the predictability of these materials to reach failure. Similar tests have been conducted on thin asphalt pavements (Epps et al., 2003).
Full Depth Reclamation (FDR) was found to work satisfactorily on a road that is sealed with a tar and chip seal. Although a tar and chip seal has advantages for sealing the road, it makes maintenance harder to perform as well as more necessary, which may not be ideal for a road. The unpaved roads with untreated bases experienced more potholes and rutting, however, these distresses tended to be less severe without maintenance by the end of the spring thaw period.

The road that remained unsealed after FDR, Himrod Road, likely experienced calcium leaching or reversal of cement hydration resulting in areas that did not hold up well over the spring thaw period. Construction should be heavily monitored during FDR projects. It was observed that care was not taken to ensure proper curing along Himrod Road during the time of construction which could have led to the problems experienced later. FDR has potential to provide positive results for an unpaved road, as it was seen to hold up very well over the spring in the area along Himrod Road that was surrounded by water on both sides. The presence of this water during construction may have aided in cement hydration where measures were not taken to keep the system moist along the other sections of the road.

Future research can include testing roads where an FDR practice was used as a base and DSA as a wearing course. This type of practice has been observed in areas subjected to heavy truck traffic aiding the oil and gas industry. The DSA can potentially act to help seal the cement-treated base and provide an easily maintained road, while also providing better resistance against rutting than PennDOT 2A.
References


Salgado, R., & Yoon, S. (2003). *Dynamic Cone Penetration Test (DCDT) for Subgrade Assessment*. West Lafayette: Joint Transportation Research Program, Indiana Department of Transportation, and Purdue University. doi:10.5703/1288284313196


Appendix A

Lightweight Deflectometer Results

The following tables and figures show the results of the stiffness testing completed with the lightweight deflectometer on Himrod Road, Reservoir Road, Church Road, and Benson Road from February–October 2011. Measurements taken from February to May were to capture the spring thaw season. Summer measurements were taken in the beginning of August, and fall measurements were taken at the end of September into the beginning of October.

Himrod Road

Figure A-1: Average stiffness results for LWD drops at specified locations on Himrod Road for February 2011.
Figure A-2: Average stiffness results for LWD drops at specified locations on Himrod Road for March 2011.

Figure A-3: Average stiffness results for LWD drops at specified locations on Himrod Road for April 2011.
Figure A-4: Average stiffness results for LWD drops at specified locations on Himrod Road for May 2011.

Figure A-5: Average stiffness results for LWD drops at specified locations on Himrod Road for summer 2011.
Figure A-6: Average stiffness results for LWD drops at specified locations on Himrod Road for fall 2011.
Reservoir Road

Figure A-7: Average stiffness results for LWD drops at specified locations on Reservoir Road for February 2011.

Figure A-8: Average stiffness results for LWD drops at specified locations on Reservoir Road for March 2011.
Figure A-9: Average stiffness results for LWD drops at specified locations on Reservoir Road for April 2011.

Figure A-10: Average stiffness results for LWD drops at specified locations on Reservoir Road for May 2011.
Figure A-11: Average stiffness results for LWD drops at specified locations on Reservoir Road for summer 2011.

Figure A-12: Average stiffness results for LWD drops at specified locations on Reservoir Road for fall 2011.
Figure A-13: Average stiffness results for LWD drops at specified locations on Church Road for February 2011.

Figure A-14: Average stiffness results for LWD drops at specified locations on Church Road for March 2011.
Figure A-15: Average stiffness results for LWD drops at specified locations on Church Road for April 2011.

Figure A-16: Average stiffness results for LWD drops at specified locations on Church Road for May 2011.
Figure A-17: Average stiffness results for LWD drops at specified locations on Church Road for summer 2011.

Figure A-18: Average stiffness results for LWD drops at specified locations on Church Road for fall 2011.
Figure A-19: Average stiffness results for LWD drops at specified locations on Benson Road for February 2011.

Figure A-20: Average stiffness results for LWD drops at specified locations on Benson Road for March 2011.
Figure A-21: Average stiffness results for LWD drops at specified locations on Benson Road for April 2011.

Figure A-22: Average stiffness results for LWD drops at specified locations on Benson Road for May 2011.
Figure A-23: Average stiffness results for LWD drops at specified locations on Benson Road for summer 2011.

Figure A-24: Average stiffness results for LWD drops at specified locations on Benson Road for fall 2011.
Table A-1: ISM (kg/mm) results calculated from the LWD measurements at three locations of Himrod Road

<table>
<thead>
<tr>
<th></th>
<th>Himrod Loc - 1</th>
<th>Himrod Loc - 2</th>
<th>Himrod Loc - 3</th>
<th>Himrod Loc - 4</th>
<th>Himrod Loc - 5</th>
<th>Himrod Loc - 6</th>
<th>Average</th>
<th>StDev</th>
</tr>
</thead>
<tbody>
<tr>
<td>February</td>
<td>0.4</td>
<td>0.37</td>
<td>0.25</td>
<td>0.58</td>
<td>0.3</td>
<td>0.31</td>
<td>0.368</td>
<td>0.117</td>
</tr>
<tr>
<td>March</td>
<td>0.19</td>
<td>0.05</td>
<td>0.02</td>
<td>0.04</td>
<td>0.03</td>
<td>0.28</td>
<td>0.102</td>
<td>0.108</td>
</tr>
<tr>
<td>April</td>
<td>0.17</td>
<td>0.1</td>
<td>0.03</td>
<td>0.11</td>
<td>0.06</td>
<td>0.13</td>
<td>0.100</td>
<td>0.050</td>
</tr>
<tr>
<td>May</td>
<td>0.05</td>
<td>0.08</td>
<td>0.05</td>
<td>0.04</td>
<td>0.07</td>
<td>0.3</td>
<td>0.098</td>
<td>0.100</td>
</tr>
<tr>
<td>Summer</td>
<td>0.09</td>
<td>0.14</td>
<td>0.1</td>
<td>0.08</td>
<td>0.15</td>
<td>0.07</td>
<td>0.105</td>
<td>0.033</td>
</tr>
<tr>
<td>Fall</td>
<td>0.25</td>
<td>0.14</td>
<td>0.12</td>
<td>0.03</td>
<td>0.14</td>
<td>0.12</td>
<td>0.133</td>
<td>0.070</td>
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</tbody>
</table>

Table A-2: ISM (kg/mm) results calculated from the LWD measurements at four locations of Reservoir Road

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<thead>
<tr>
<th></th>
<th>Reservoir Loc - 1</th>
<th>Reservoir Loc - 2</th>
<th>Reservoir Loc - 3</th>
<th>Reservoir Loc - 4</th>
<th>Average</th>
<th>StDev</th>
</tr>
</thead>
<tbody>
<tr>
<td>February</td>
<td>1.79</td>
<td>0.37</td>
<td>0.25</td>
<td>0.27</td>
<td>0.670</td>
<td>0.749</td>
</tr>
<tr>
<td>March</td>
<td>0.65</td>
<td>0.31</td>
<td>0.04</td>
<td>0.15</td>
<td>0.288</td>
<td>0.266</td>
</tr>
<tr>
<td>April</td>
<td>0.22</td>
<td>0.26</td>
<td>0.32</td>
<td>0.33</td>
<td>0.283</td>
<td>0.052</td>
</tr>
<tr>
<td>May</td>
<td>0.35</td>
<td>0.21</td>
<td>0.19</td>
<td>0.23</td>
<td>0.245</td>
<td>0.072</td>
</tr>
<tr>
<td>Summer</td>
<td>1.14</td>
<td>0.36</td>
<td>0.31</td>
<td>0.33</td>
<td>0.535</td>
<td>0.404</td>
</tr>
<tr>
<td>Fall</td>
<td>0.25</td>
<td>0.14</td>
<td>0.35</td>
<td>0.15</td>
<td>0.223</td>
<td>0.098</td>
</tr>
</tbody>
</table>
Table A-3: ISM (kg/mm) results calculated from the LWD measurements at three locations of Church Road

<table>
<thead>
<tr>
<th></th>
<th>Church Loc - 1</th>
<th>Church Loc - 2</th>
<th>Church Loc - 3</th>
<th>Average</th>
<th>StDev</th>
</tr>
</thead>
<tbody>
<tr>
<td>February</td>
<td>0.08</td>
<td>0.02</td>
<td>0.04</td>
<td>0.047</td>
<td>0.031</td>
</tr>
<tr>
<td>March</td>
<td>0.01</td>
<td>0.03</td>
<td>0.01</td>
<td>0.017</td>
<td>0.012</td>
</tr>
<tr>
<td>April</td>
<td>0.03</td>
<td>0.03</td>
<td>0.05</td>
<td>0.037</td>
<td>0.012</td>
</tr>
<tr>
<td>May</td>
<td>0.01</td>
<td>0.04</td>
<td>0.03</td>
<td>0.027</td>
<td>0.015</td>
</tr>
<tr>
<td>Summer</td>
<td>0.04</td>
<td>0.04</td>
<td>0.06</td>
<td>0.047</td>
<td>0.012</td>
</tr>
<tr>
<td>Fall</td>
<td>0.04</td>
<td>0.06</td>
<td>0.1</td>
<td>0.067</td>
<td>0.031</td>
</tr>
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Table A-4: ISM (kg/mm) results calculated from the LWD measurements at three locations of Benson Road

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<thead>
<tr>
<th></th>
<th>Benson Loc - 1</th>
<th>Benson Loc - 2</th>
<th>Benson Loc - 3</th>
<th>Average</th>
<th>StDev</th>
</tr>
</thead>
<tbody>
<tr>
<td>February</td>
<td>0.06</td>
<td></td>
<td></td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>March</td>
<td>0.14</td>
<td>0.08</td>
<td>0.12</td>
<td>0.11</td>
<td>0.03</td>
</tr>
<tr>
<td>April</td>
<td>0.05</td>
<td>0.01</td>
<td>0.01</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>May</td>
<td>0.01</td>
<td>0.03</td>
<td>0.03</td>
<td>0.02</td>
<td>0.01</td>
</tr>
<tr>
<td>Summer</td>
<td>0.19</td>
<td>0.03</td>
<td>0.11</td>
<td>0.11</td>
<td>0.08</td>
</tr>
<tr>
<td>Fall</td>
<td>0.07</td>
<td>0.04</td>
<td>0.04</td>
<td>0.05</td>
<td>0.02</td>
</tr>
</tbody>
</table>
Appendix B

MMLS3 Results

Laboratory Testing

The following figures show the results from the MMLS3 testing completed on DSA and PennDOT 2A material. The testing was completed in the laboratory at a lightweight traffic setting and at a heavy traffic setting. Field testing was only completed at the lightweight traffic setting.
Driving Surface Aggregate

**Lightweight Traffic Loading**

Figure B-1: Lightweight MMLS3, Driving Surface Aggregate testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 200mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.
Figure B-2: Lightweight MMLS3, Driving Surface Aggregate testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 350mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.
Figure B-3: Lightweight MMLS3, Driving Surface Aggregate testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 500mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.
Figure B-4: Lightweight MMLS3, Driving Surface Aggregate testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 650mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.
Figure B-5: Lightweight MMLS3, Driving Surface Aggregate testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 800mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.
Heavy Traffic Loading

Figure B-6: Heavy MMLS3, Driving Surface Aggregate testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 350mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.
Figure B-7: Heavy MMLS3, Driving Surface Aggregate testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 500mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.
Figure B-8: Heavy MMLS3, Driving Surface Aggregate testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 650mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.

\[ y = 0.5136 \ln(x) - 3.3017 \]

\[ R^2 = 0.8994 \]
Figure B-9: Heavy MMLS3, Driving Surface Aggregate testing.  
a) Surface profile of the pavement section up to 100,000 cycles taken at the 800mm tire position.  
b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line

\[ y = 0.9921 \ln(x) - 6.7145 \]

\[ R^2 = 0.9358 \]
PennDOT 2A

Lightweight Traffic Loading

Figure B-10: Lightweight MMLS3, PennDOT 2A testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 200mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.

\[ y = 2.7247 \ln(x) - 15.199 \]

\[ R^2 = 0.959 \]
Figure B-11: Lightweight MMLS3, PennDOT 2A testing.  

a) Surface profile of the pavement section up to 100,000 cycles taken at the 350mm tire position.

b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.

Mathematical expression:

\[ y = -0.029 \ln(x) - 6.7518 \]

\[ R^2 = 0.0049 \]
Figure B-12: Lightweight MMLS3, PennDOT 2A testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 500mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.

\[ y = 0.5138 \ln(x) + 0.2395 \]

\( R^2 = 0.3232 \)
Figure B-13: Lightweight MMLS3, PennDOT 2A testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 650mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.
Figure B-14: Lightweight MMLS3, PennDOT 2A testing. a) Surface profile of the pavement section up to 100,000 cycles taken at the 800mm tire position. b) Change in rut depth over 100,000 cycles plotted along with a logarithmic trend line.
Heavy Traffic Loading

Figure B-15: Heavy MMLS3, PennDOT 2A testing. a) Surface profile of the pavement section up to 30,000 cycles taken at the 200mm tire position. b) Change in rut depth over 30,000 cycles plotted along with a logarithmic trend line.
Figure B-16: Heavy MMLS3, PennDOT 2A testing. a) Surface profile of the pavement section up to 30,000 cycles taken at the 350mm tire position. b) Change in rut depth over 30,000 cycles plotted along with a logarithmic trend line.
Figure B-17: Heavy MMLS3, PennDOT 2A testing. a) Surface profile of the pavement section up to 30,000 cycles taken at the 500mm tire position. b) Change in rut depth over 30,000 cycles plotted along with a logarithmic trend line.
Figure B-18: Heavy MMLS3, PennDOT 2A testing. a) Surface profile of the pavement section up to 30,000 cycles taken at the 650mm tire position. b) Change in rut depth over 30,000 cycles plotted along with a logarithmic trend line.
Figure B-19: Heavy MMLS3, PennDOT 2A testing. a) Surface profile of the pavement section up to 30,000 cycles taken at the 800mm tire position. b) Change in rut depth over 30,000 cycles plotted along with a logarithmic trend line

\[ y = 0.1181 \ln(x) + 1.3049 \]

\[ R^2 = 0.0802 \]
Field

Driving Surface Aggregate

Natural Conditions – Day 1

Figure B-20: Field MMLS3, testing at lightweight setting – day 1. a) Surface profile of the pavement section up to 31,300 cycles taken at the 200mm tire position. b) Change in rut depth over 31,300 cycles plotted along with a logarithmic trend line.
Figure B-21: Field MMLS3, testing at lightweight setting – day 1. a) Surface profile of the pavement section up to 31,300 cycles taken at the 350mm tire position. b) Change in rut depth over 31,300 cycles plotted along with a logarithmic trend line.
Figure B-22: Field MMLS3, testing at lightweight setting – day 1. a) Surface profile of the pavement section up to 31,300 cycles taken at the 500mm tire position. b) Change in rut depth over 31,300 cycles plotted along with a logarithmic trend line
Figure B-23: Field MMLS3, testing at lightweight setting – day 2. a) Surface profile of the pavement section up to 31,300 cycles taken at the 200mm tire position. b) Change in rut depth over 31,300 cycles plotted along with a logarithmic trend line.
Figure B-24: Field MMLS3, testing at lightweight setting – day 2. a) Surface profile of the pavement section up to 31,300 cycles taken at the 350mm tire position. b) Change in rut depth over 31,300 cycles plotted along with a logarithmic trend line.
Figure B-25: Field MMLS3, testing at lightweight setting – day 2. a) Surface profile of the pavement section up to 31,300 cycles taken at the 500mm tire position. b) Change in rut depth over 31,300 cycles plotted along with a logarithmic trend line.
PennDOT 2A

Natural Conditions – Day 1

Figure B-26: Field MMLS3, testing at lightweight setting – day 1. a) Surface profile of the pavement section up to 16,500 cycles taken at the 200mm tire position. b) Change in rut depth over 16,500 cycles plotted along with a logarithmic trend line.
Figure B-27: Field MMLS3, testing at lightweight setting – day 1. a) Surface profile of the pavement section up to 16,500 cycles taken at the 350mm tire position. b) Change in rut depth over 16,500 cycles plotted along with a logarithmic trend line.

\[ y = 0.6852 \ln(x) + 4.0179 \]

\[ R^2 = 0.9975 \]
Figure B-28: Field MMLS3, testing at lightweight setting – day 1. a) Surface profile of the pavement section up to 16,500 cycles taken at the 500mm tire position. b) Change in rut depth over 16,500 cycles plotted along with a logarithmic trend line.
Natural Conditions – Day 2

Figure B-29: Field MMLS3, testing at lightweight setting – day 2. a) Surface profile of the pavement section up to 21,200 cycles taken at the 200mm tire position. b) Change in rut depth over 21,200 cycles plotted along with a logarithmic trend line.
Figure B-30: Field MMLS3, testing at lightweight setting – day 2. a) Surface profile of the pavement section up to 21,200 cycles taken at the 350mm tire position. b) Change in rut depth over 21,200 cycles plotted along with a logarithmic trend line

PennDOT 2A - Natural 2 (350)

a) Surface profile

b) Rut depth vs. number of cycles

Regression equation: $y = 0.4651\ln(x) + 10.584$

R² = 0.1052
Figure B-31: Field MMLS3, testing at lightweight setting – day 2. a) Surface profile of the pavement section up to 21,200 cycles taken at the 500mm tire position. b) Change in rut depth over 21,200 cycles plotted along with a logarithmic trend line.
Appendix C

Weather Conditions for Field Testing

The following figures show the weather during the months LWD and condition assessments were performed on the roads in Erie County, Pa. The months observed include February, March, April, May, August, September and October 2011.

Figure C-1: Weather forecast near roads included in the field studies. February testing was performed between February 17th and 18th.
Figure C-2: Weather forecast near roads included in the field studies. March testing was performed between March 8th and 10th.
Figure C-3: Weather forecast near roads included in the field studies. April testing was performed between April 9th and 11th.
Figure C-4: Weather forecast near roads included in the field studies. May testing was performed between May 16th and 18th.
Figure C-5: Weather forecast near roads included in the field studies. Summer testing was performed between August 8th and 10th.
Figure C-6: Weather forecast near roads included in the field studies. Fall testing was performed between September 27th and October 1st.