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Optimizing Reclaimed Asphalt Pavement (RAP) Content in Unbound Base Aggregate

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16. Abstract:

This report is written to summarize the findings of a study conducted by George Mason University's (GMU) Sustainable Geo Infrastructure (SGI) Research group to evaluate the expansion of using reclaimed asphalt pavement (RAP) and to optimize the RAP content in unbound base aggregate. The particular scope of the research presented in this report has been determined by the VTRC/VDOT team and included a laboratory and a field study.

Prior to the initiation of the research, based on the conversations with Virginia's road building industry it was determined that the research will focus on evaluating what is referred as "fine processed" RAP (100% of the particles finer than 1-inch). Virgin aggregate (VA) used in this study complied with the VDOT's 21A gradation and contained geologically similar aggregate pieces (diabase). During the initial phase, based on the availability, samples of RAP from 14 different asphalt plants in Virginia were collected and characterized. The goal was to assess the similarities and differences between the RAP produced throughout the Commonwealth. The results showed that all RAP samples had similar grain size distribution and primarily contained pieces of aggregate that were of diabase origin. The binder (asphalt) content of the samples ranged from 4.4 to 6.1%. Based on this characterization, samples from three different plants that represented a low, medium, and high binder content RAP were selected for detailed evaluation. The ages of the collected RAP samples were not known and most likely varied.

Laboratory evaluation focused on assessing the performance of the RAP-VA blends against the performance of the 100% VA alone. CBR, resilient modulus (M_r), and permanent deformation (PD) tests were used to evaluate performance. Up to 60% RAP (with three different binder contents) was blended with VA by weight. Results from PD tests showed a threshold where some addition of RAP into VA improved the performance but beyond a specific threshold, the overall performance started to decline. This threshold was determined to be a function of both the percentage of RAP added to a blend and the percentage of the binder content of the 100% RAP used. The optimized maximum RAP percentage in a given blend was determined in this study as up to 20% for RAP with low binder content (i.e., ≤ 4.6%) and up 30% for RAP with high binder content (i.e., ≥ 5.6 %). The reasons why RAP with different binder content resulted in different percent threshold requires further investigation as it could be due to the differences in the age of the RAP, which was not part of the scope of this study.

Field evaluation part of the study involved in constructing an actual roadway with four different base course layers consisting of 20 and 30% RAP-VA blends with low (4.5%) and high (5.7%) binder contents. Sections constructed with VA alone were used as a comparison (control sections). Field study demonstrated that Light Weight Deflectometer and modified speedy moisture content tests are suitable tools to be used for quality control of RAP-VA blends during construction. Performance evaluation in the field was monitored for a year with the embedded instruments and nondestructive tests conducted during and after the construction. Results obtained from the field were in agreement with the laboratory observations.

Based on the findings from laboratory and supported by the field observations, a relationship between the binder content of the 100% RAP and maximum allowed RAP percentage to create RAP-VA blends is created to provide guidelines for implementation.

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FINAL REPORT

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DISCLAIMER

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INTRODUCTION

The use of reclaimed asphalt pavement (RAP) in pavement structure is not a recent topic. RAP has been in use to produce hot mix asphalt (HMA) for decades. The importance of RAP initially emerged due to shortage in petroleum products in 1970s, where using some amount of RAP in the production of asphaltic mixtures became a viable alternative (NCHRP 1978). However, with the implementation of Superior Performing Asphalt Pavements (Superpave) mixture design method in 1990s, the previously determined guidelines for using RAP in asphaltic mixtures became obsolete. The advent of Superpave did not provide guidelines to use RAP in asphaltic mixtures and as a result many DOTs in the U.S. limited the use of RAP to only 15% to produce asphalt mixtures (Copeland 2011). The decrease in being able to use RAP to create asphaltic mixtures augmented the accumulation of RAP stockpiles throughout the U.S. (Copeland 2011). Therefore, researchers focused their attention in finding alternative ways to recycle RAP. Taha et al. (1999) reported that even in late 1990s, early 2000s, United States annually produced 50 million tons of RAP. Later, in less than a decade, several studies suggested that the production of RAP has doubled to a rate of 100 million tons (NCHRP 2004, Copeland 2011). The decline in the usage of RAP to develop HMA created a need to evaluate the potential of recycling RAP in the base course of pavement structure as an unbound aggregate. Even though early accounts of RAP as base aggregates emerged in 1982 (Defoe 1982), majority of the efforts have increased in late 1990s due to the limitations of Superpave method to recycle RAP to create new HMA mixtures. However, large quantities of RAP in the U.S. remain unused because of limited application in asphalt mixtures (Hoppe et al. 2015).

Maher et al. (1997) was one of the first researchers in late 1990s to characterize RAP. The research has focused on evaluating 100% RAP, 100% virgin aggregate (VA) and blends of 85 to 90% RAP with 10 to 15 % VA to create unbound aggregates. The performance evaluations were conducted based on resilient modulus (M_R) tests in the laboratory. The findings of their research indicated that 100% RAP and blends of RAP with VA had higher M_R than the VA. In the early 2000s, many researchers have also attempted to characterize RAP as base course material on the basis of M_R and almost all of them have reported that an increase in RAP percentage in RAP-VA blends enhances the M_R of the VA (MacGregor et al. 1999, Bennert et al. 2000, Kim et al. 2007, Locander 2009, Alam et al. 2010, Bozyurt et al. 2012, Stolle et al. 2014, Domitrović et al. 2016, and Domitrović et al. 2019). Based on these results it was proclaimed by many researchers that 100% RAP and blends of RAP with VA showed higher performance than 100% VA alone because of higher stiffness depicted via modulus results. Considering that M_R is one of the primary parameters for the mechanistic empirical pavement design guide (MEPDG), such conclusion requires important evaluation.

Even though the modulus of the RAP-VA blends increases with the addition of RAP, some researchers pointed out that samples containing RAP tend to accumulate higher permanent deformation (PD) than VA (Bennert et al. 2000, Kim et al. 2007, Jeon et al. 2009, Schaertl et al. 2010, and Edil et al. 2012) and others claim the opposite (Attia 2010). The research efforts claiming adverse effects with RAP included both bench and large-scale tests 9.8-ft x 9.8-ft x 9.8-ft (3-m x 3-m x 3-m) and had outcomes that were in agreement with each other in terms of either completely limiting the use of RAP or to keep the percentage to be at a certain limit. The exact percentage to limit RAP in a given blend varied from one researcher to another. Contrary to many other researchers, the research conducted by Attia (2010) claimed that even 100% RAP had less PD than the 100% VA. The outcome of the above listed previously conducted research point out that unlike the consistent results seen from M_R evaluations, when it comes to PD, there is discrepancy in the literature. One of the major reasons behind these discrepancies may be associated with the fact that most of the researchers did not study the properties of RAP (such as the variation in binder content and gradation). Bilodeau et al. (2013) also claimed that the discrepancy in the literature as it relates to the PD of RAP could potentially be attributed to the properties of RAP (especially to the binder content). However, Bilodeau et al. (2013) did not conduct any tests to prove/support this statement. Additionally, the VA used in these studies varied by gradation and geological make-up. Furthermore, the testing procedures have varied from one study to another in terms of applied stress conditions and the number of applied load cycles as there is no agreed upon standard to conduct PD tests. Therefore, these studies may not be direct comparisons to each other and the results may not just be implemented without further confirmation. To the best of the authors knowledge, no recent study beyond 2013 has focused on the inconsistencies associated with PD or evaluated the effects of binder content of RAP to the overall performance of the RAP-VA blends.

The recent studies focused on evaluating the visco-elastic properties of RAP as it relates to testing the behavior of the RAP under colder and warmer temperatures. Dong and Huang (2014) have focused on testing RAP under 41, 59, and 77 degrees Fahrenheit (5, 15, and 25 degrees Celsius) and evaluated the creep behavior. The magnitude of the applied load simulated 41 psi (283 kPa) stress conditions. The results indicated that at 77^o Fahrenheit (25^o Celsius), the material showed extensive creep behavior. Such observation recommended not to use 100% RAP especially when the temperature of the base course rises above 59^o Fahrenheit (15 ^{o} Celsius). Soleimanbeigi and Edil (2015) have also focused on evaluating the effects of temperature to the creep behavior of the 100% RAP in addition to evaluating the M_R behavior.

The results were similar to the Dong and Huang (2014) where the RAP showed extensive behavior of creep at 72º Fahrenheit (22º Celsius) and above. The temperature also adversely effected the M_R behavior. The outcomes of these studies showed that either the temperature fluctuation of the base course layer has to be controlled throughout the life-time of the roadway or 100% RAP not be used for creating an unbound base course in regions where the temperatures could fluctuate above room temperatures (i.e., 72 to 75º Fahrenheit or 22 to 24º Celsius)

Previous laboratory studies on RAP as base course provides good information about changes in M_R and PD results but does not answer why such discrepancies in the literature exist. It is believed that the discrepancies in the literature occur due to difference in properties of RAP from where it is obtained, and the testing methodology (including sample preparation and PD testing method) adopted in the laboratory. Findings from the literature are important but none of these studies provide guidelines to design an optimum percentage of RAP that may be mixed with unbound base course aggregate such that the performance of the RAP-VA blend is equivalent or better than the VA base course alone.

In terms of long-term field performance evaluation, very limited literature is available where RAP has been used as base course material. Garg and Thompson (1996), Maher et al. (1997), and Saride et al. (2010) are among the researchers who have published summaries of field studies in all cases these studies involved in evaluating the performance of 100% RAP compared against the 100% VA or another type of recycled material. The thickness of the HMA layer in these studies ranged from 3 to 12 inches (76 to 300 mm) and the base course layer ranged from 6 to 9 inches (150 to 230 mm). In all cases subgrade was conditioned to be component either by lime stabilization or use of geosynthetics. Garg and Thompson (1996) study focused on evaluating the performance of the RAP base course as a function of falling weight deflectometer (FWD) and Dipstick® apparatus both of which were implemented at the surface of the HMA layer. The study monitored the performance of the roadway for two years between October 1993 and August 1995. The results of FWD measurements indicated that RAP base course provides adequate structural support based on back-calculated modulus. This finding is consistent with the laboratory studies where addition of RAP improves the elastic modulus. Dipstick® measurements were used to determine the average rut depths. The results showed that along sections constructed with VA, the HMA surface of the north bound lane (NL) showed 10 mils (0.25-mm) of deflection. Measurements along the south-bound lane (SL) was 140 mils (3.56-mm). In the case of sections constructed with RAP base, the average rut depths were 24 mils (0.61-mm) along the NL and 95 mils (2.41-mm) along the SL. Even though the study did not provide evidence related to traffic conditions, it was claimed that higher rutting depths along the SL was attributed to increased traffic loading. When the results from the NL were compared with each other, the section constructed with RAP had approximately 2.4 times more rutting. However, for the SL RAP section showed less rutting than the VA section. The discrepancy between the NL and SL raises questions as increasing the number of traffic cycles does not necessarily justify the changes in observed behavior where in one lane RAP is significantly under performing and in other lane it is the superior section. Maher et al. (1997) not only evaluated the 100% RAP but also included a base course section with 25% RAP – 75% VA blend. The study only focused on comparing the back-calculated elastic modulus of the base course layers and used Seismic Pavement Analyzer for this comparison. The results showed that sections containing RAP showed similar or slightly higher elastic modulus than the section with

VA. This observation was made right after the completion of the construction and the finding is in line with the previous literature from laboratory studies where addition of RAP improves the modulus. However, the study did not include long-term evaluation and investigating the occurrence of rutting over time. Saride et al. (2010) compared the performance of the RAP base course against the performance of the base course constructed with cemented quarry fine (CQF). In this study the performance of the horizontally placed inclinometers was evaluated for 2.5 years after construction. The results showed fairly close performance on both sections where the maximum vertical deformation was identified as 0.39 inches (10-mm). None of the studies indicated any information regarding the binder content of the RAP. Also, all of these studies used RAP with maximum particle size of less than 1.5-inches (38.1 mm) except in the case of Saride et al. (2010), the RAP had maximum particle size of 0.5-inches (12.7 mm). The conclusions from these studies were consistent as it relates to the elastic modulus but were different from each other as it related to rutting and deformations.

The replacement of conventional aggregate base course with RAP requires reliable methods to evaluate the quality of compacted base course. Reliable quality control measures for compaction control are as important and as sophisticated as laboratory performance evaluation tests. This is traditionally achieved in the U.S. by using a nuclear density gauge (NDG), where the moisture and wet density of the aggregate are measured, and the dry density is calculated (Smith and Diefenderfer 2008). NDG measures dry density that represents the field conditions and then compares the measurements to the target maximum dry density obtained from the Proctor test that is conducted in the laboratory. However, in the case where the unbound aggregate is created from RAP, as reported in the literature, the NDG does not provide accurate results for moisture content measurements (Viyanant et al. 2004). This is because the binder content of the aggregates within the RAP contains hydrocarbon compounds, which absorb gamma rays from the radioactive source and affects the moisture content results (Smith and Diefenderfer 2008). Therefore, the calculated dry density from the NDG equipment also becomes inaccurate. Because of this limitation, currently the Department of Transportation (DOT) agencies in the U.S. do not have quality control guidance specifically for RAP-VA blends (Hoppe et al. 2015) and this challenge limits the broader use of RAP as base course. Foye (2011) conducted a field study to focus on suitability of using dynamic cone penetrometer (DCP) as a quality control for compaction of the base course layers. The study only gathered data during construction as the focus was to compare the DCP analyses on sections with and without RAP. The binder content of the RAP used in this study was 4.9%. The conclusion of the study stated that the DCP results indicated no rutting or deflection under proof rolling. How such a conclusion was compared against NDG or to other instruments and visual inspection are not been discussed in the study.

Table 1 summarizes the list of Department of Transportation (DOT) agencies within the U.S. that mentions the use of RAP as part of unbound base aggregate in their road and bridge specifications. The list was created by reviewing the specifications of 51 DOT agencies including DC, Puerto Rico, and Hawaii. Details of how each State has established their criteria was not available. Table 1 shows that different State agencies have different limits in terms of maximum particle size allowed. This makes the comparison of the listed thresholds difficult as it is commonly known that the difference in gradation has an effect on the performance of unbound aggregates. Also, some agencies have established their criteria based on maximum percentage of allowed RAP (ignoring the binder content of the RAP) and others prescribed the maximum binder percentage (ignoring the percentage of RAP in a given blend). Considering the discrepancies in the literature as well as the differences in properties of RAP produced from one State to another, it is not surprising that the limits established by agencies vary. It is also important to note that many agencies use nuclear density gauge as part of their quality control tests of compacted RAP containing base course layers even though previous literature indicates problems of using nuclear density gauge with unbound aggregate containing RAP.

Table 1: Summary of DOT Agencies Allowing Use of RAP for Base Course Under Traffic

State	Max. % RAP	Max % Binder Content	Max. Particle Size (inch)	QC/QA Testing	References
Alaska	N/S (designed)[1]	N/S	1.5	Nuclear Gauge	ADOT (2017)[2]
Arkansas	N/S	$2.5 \leq \% \leq 3.0$	1.0	Nuclear Gauge	AHTD (2014)[3]
California	50	N/S	0.75	Nuclear Gauge	Caltrans (2018)[4]
Colorado	100	N/S	2.0	Nuclear Gauge[5]	CDOT (2019)[6]
Connecticut	N/S	2.0	3.5	Nuclear Gauge	ConnDOT (2016)[7]
Florida	N/S	≥ 4.0	3.5	Nuclear Gauge[5]	FDOT (2019)[8]
Minnesota	N/S	3.5	1.5	DCP and LWD	MNDOT (2018)[9]
Montana	40	N/S	2.0	Nuclear Gauge[5]	MDT (2020)[10]
New Hampshire	N/S	1.5	0.75	Sand Cone or Nuclear Gauge	NHDOT (2016)[11]
New Jersey	50	N/S	1.5	Nuclear Gauge	NJDOT (2019)[12]
New Mexico	50	N/S	1.0	Nuclear Gauge	NMDOT (2019)[13]
New York	95[14]	N/S	2.0	Visual inspection[15]	NYSDOT (2019)[16]
North Dakota	100	3.5	1.5	Sand Cone and Rubber Balloon[17]	NDDOT (2019)[18,19]
Texas	20	N/S	2.0	Nuclear Gauge	TxDOT (2014)[20]

Notes:
N/S: Not specified,
[1]Specific design guidelines only provided based on gradation,
[2]ADOT (2017), Section 308 Crushed Asphalt Base Course, Page 118,
http://www.dot.state.ak.us/stwddes/dcsspecs/assets/pdf/hwyspecs/sshc2017.pdf,
[3]AHTD (2014), Section 417 Open Graded Asphalt Base Course, Page 301,
https://www.arkansashighways.com/standard_spec/2014/2014SpecBook.pdf
[4]Caltrans (2018), Section 30 Reclaimed Pavement, Page 411,
https://dot.ca.gov/-/media/dot-media/programs/design/documents/f00203402018stdspecsa11y.pdf
[5]The specifications did not directly provide a method but is implied from the text,
[6]CDOT (2019), Section 703 Aggregates, Page 7-14,
https://www.codot.gov/business/designsupport/cdot-construction-specifications/2019-construction-specifications/2019-specs-book/2019-division-700
[7]ConnDOT (2016), Section M.02, Page 571,
https://portal.ct.gov/-/media/DOT/documents/dpublications/817/Form817Origrevpurchaseinfopdf.pdf?la=en

[8]FDOT (2019), Section 283, Reclaimed Asphalt Pavement Base, Page 231, https://fdotwww.blob.core.windows.net/sitefinity/docs/default-source/programmanagement/implemented/specbooks/july-2019/719ebook.pdf?sfvrsn=bf366a48_2
[9]MNDOT (2018), Section 3138 Aggregate for Surface and Base Course, Page 561, http://www.dot.state.mn.us/pre-letting/spec/2018/2018-spec-book-final.pdf
[10]MDT (2020), Section 301 Aggregate Surfacing, Page 301-1, https://www.mdt.mt.gov/other/webdata/external/const/specifications/2020/SPEC-BOOK/2020-SPEC-BOOK-V1-2.pdf
[11]NHDOT (2016), Section 306, https://www.nh.gov/dot/org/projectdevelopment/highwaydesign/specifications/documents/2016NHDOTSpecBookWeb.pdf
[12]NJDOT (2019), Section 901.10.03, Page 385, https://njdotlocalaidrc.com/perch/resources/standard-specifications-for-road-and-bridge-construction.pdf
[13]NMDOT (2019), Section 303 Base Course, Page 130, https://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/2019_Specs.pdf
[14]No specific guidelines were provided for RAP-VA blends,
[15]Although no density test is required the specifications state that the State reserves the right to perform density tests-method was not described,
[16]NYSDOT (2019), Section 733-06 Reclaimed Asphalt Pavement for Earthwork and Subbase, Page 1282, https://www.dot.ny.gov/main/business-center/engineering/specifications/busi-e-standards-usc/usc-repository/2019_1_specs_usc.pdf
[17]Explicit information about density control was not found but NDDOT's testing methods refer to sand conde and rubber balloon methods,
[18]Original document referred for contract is published in 2014. (1 inch = 25.4 mm)
[19]NDDOT (2019), Section 817 Salvaged Base Course, Page 420, https://www.dot.nd.gov/divisions/environmental/docs/supspecs/fullsupplementalspecswith10012018.pdf
[20]TxDOT (2014), Item 247 Flexible Base, Page 127, ftp://ftp.dot.state.tx.us/pub/txdot-info/des/spec-book-1114.pdf

Based on the above summarized literature it is evident that there is lack of uniformity in the approaches and conclusions drawn when evaluating the performance of RAP and RAP-VA blends to be considered for unbound base coarse aggregate. The problems may be listed as:

- The gradations of the evaluated RAP are not consistent;
- Effects of binder content on RAP performance is not well investigated;
- Definition of performance is not well established; and
- Methods to conduct quality control in the field are not well defined.

PURPOSE AND SCOPE

The overall goal of the research presented in this document was to particularly enable greater use of RAP as an unbound base course in Virginia. Four specific tasks were identified:

Task 1: Develop a performance-based evaluation to relate the performance of a typical VA base course layer with RAP/VA blended base layer. The goal was to determine a threshold of how much RAP may be added to VA before the addition of RAP starts to show signs of adverse effects in terms of the performance of the final product compared to VA.

Task 2: Evaluate the suitability of different field methods to perform quality control during construction geared towards determining guidance on how the material will be accepted in the field. The goal was to assess a tool that may be readily available to VDOT for quality control.

Task 3: Evaluate the in-situ field performance of the RAP-VA blends in the field to confirm that the findings from the laboratory portion of this study (achieved in task 1) can be implemented in the actual roadway construction.

Task 4: Provide assistance to VDOT for developing specifications to implement RAP usage of constructing unbound aggregate base course.

It was agreed that the research activities would involve task 1 to be completed based on the laboratory studies, tasks 2 and 3 based on the field studies with laboratory support, and task 4 be achieved based on the overall findings of the research and feedback from VDOT engineers.

METHODS

Overview

The scope of this study included four tasks; however, the three out of the four tasks had an experimental component that required laboratory tests or field demonstration. Specific methods as it relates to achieving each of these three tasks have been described below in detail.

Methods Implemented to Address Task 1

When it comes to design, the MEPDG refers to characterizing the base course primarily based on modulus. The previous studies indicate that addition of RAP into VA in fact improves the modulus of the VA alone. Yet previous studies also show that adding RAP into VA to create base course adversely result in rutting / permanent deformations. These two findings are opposite to each other. Therefore, in this study both the modulus and deformation characteristics of RAP-VA blends have been specifically evaluated with laboratory tests. The purpose of these tests was to seek answers to address the task 1 where a threshold on how much RAP may be added to VA is determined based on the percentage and the binder content of RAP. The goal was to identify a specific RAP-VA blend that in all three test conditions conducted in this study, the RAP-VA blend shows equal or better performance. The specific methods implemented for each of the three different laboratory tests conducted in this study are described below.

Resilient Modulus (M_R) Test

Resilient modulus (M_R) is defined as the modulus of soil obtained based on the recoverable strain under repeated loading condition. The M_R is expressed by the following relationship:

$$M_R = \frac{\sigma_{dev}}{\epsilon_{res}} \quad \text{Equation 1}$$

where σ_{dev} is the deviator stress and ϵ_{res} resilient strain

When it comes to determining the elastic behavior, many DOTs in the U. S. have recognized resilient modulus test procedure as the basis to determine such behavior (Puppala 2008). It is for this reason great emphasis has been laid in characterizing unbound base aggregates using M_R and bulk stress relationships (Mohammad et al. 1994, Hossain and Lane 2015 and Clayton et al. 2016). The most common testing procedure /protocol to determine M_R has been laid out in detail by AASHTO TP46 (1994) and Witczak (2003) per NCHRP 1-28A. These two documents describe the same procedure and are also followed in this study.

The results obtained from the laboratory tests are plotted as a function of bulk stresses applied during the test. This compiled data set is then fitted with a curve following the procedures and the equation outlined in the MEPDG model (NCHRP 2004) as shown in Equation 2. This equation is then used to determine the resilient modulus (M_R) value corresponding to a specific bulk (θ) and octahedral (τ_{oct}) stresses.

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \qquad \text{Equation 2}$$

where 'p_a' is atmospheric pressure equivalent to 14.7 psi (100 kPa); k_1, k_2, and k_3 are MEPDG model fitting parameters that are obtained from test results.

Definition of bulk and octahedral stresses are provided in Equations 3 and 4 below:

$$\theta = \sigma_1 + \sigma_2 + \sigma_3 \qquad \text{Equation 3}$$

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2} \qquad \text{Equation 4}$$

where σ_1, σ_2, and σ_3 are principal stresses.

Permanent Deformation (PD) Test Protocol

Characterization of unbound material is a complex procedure although there is a great inclination towards modeling the pavement layer in laboratory tests based on simplified elastic approach. This approach is primarily achieved by conducting M_R tests. However, according to Morgan (1966), just relying on M_R and other elastic parameters falls short of truly predicting the performance of a pavement layer. Xiao and Tutumluer (2012) also suggest that the resilient properties are not enough to characterize all granular aggregate materials and that the PD tests be considered as part of characterization of unbound aggregate base (UAB) material. Hence it is not surprising that in recent laboratory evaluations, PD tests are considered as important as M_R tests (Mohammad et al. 1994, Paute et al. 1996, Muhanna et al. 1998, Puppala et al. 1999 and 2005, and Xiao and Tutumluer 2012). This is because PD tests provide information regarding parameters to predict failure of flexible pavement (Bennert et al. 2000). However, there is no standard testing method in the U.S. to conduct PD tests on UAB materials. Therefore, prior to establishing a testing protocol, an extensive literature survey and numerical methods were conducted in this study.

The sample preparation technique for PD test is similar to the conventional M_R test, yet stress conditions and sequence of load applications are varied to simulate actual pavement distresses (permanent deformation and rutting potential). The parameters required for running a permanent deformation test include representative stress levels in the middle of the base course layer, type of loading impulse, and number of cycles in loading sequence.

The representative stress levels in the middle of the base course layer had to be specifically determined in this study. In order to determine this information, Barksdale and Itani (1989), Mohammad et al. (2006), Chow (2014) recommended the use of multilayer solution where the overlain asphalt layer is modeled as a thin layer representing low traffic pavement design conditions. Therefore, multi-layer and finite element model (FEM) techniques have been adopted in this study to determine the representative stress levels in typical pavements (low traffic) in Virginia. The results of multi-layer solutions obtained in this study have been presented in Table 2 along with the published data in literature. These stress conditions were then tested in a PD test to determine the condition that results in higher PD (worst case scenario).

To determine the type of loading impulse, findings from the Virginia Smart Road project were consulted. Loulizi et al. (2002) conducted field tests as part of that project and determined that a haversine or normalized bell-shaped curves are relevant to illustrate the vertical compressive stress pulse from a moving vehicle. Therefore, PD tests in this study were conducted with haversine loading impulse. The number of cycles in loading sequence was established as 10,000 load cycles based on prior literature (Tanyu et al. 2005, Puppala et al. 2005, Mohammad et al. 2006, Chow 2014).

Table 2: Representative Stress Conditions in a Typical Low Traffic Pavement

Pavement Section	**Depth (in)**	**Representative stress conditions in the middle of base layer**		**$\sigma_1 = \sigma_d + \sigma_3$ (psi)**	**Reference**
		σ_3 (psi)	**σ_d (psi)**		
4-in HMA 8-in Base	8	5	16	21	Calculations performed in GMU
5-in HMA 8-in Base	9	4	12	16	Calculations performed in GMU
2-in HMA 8-in Base	6	6	30	36	Barksdale and Itani (1989)
3-in HMA 8-in Base	7	5	25	30	Chow (2014)

Note: σ_1 and σ_3 represent the vertical and horizontal (confinement) stresses and σ_d represents the deviator stress. (in: inches, psi= pounds per square inch, 1-inches = 25.4 mm, 1 psi = 6.89 kPa)

To determine the strain rates that result from the stress conditions presented in Table 2, four different PD tests were conducted with the VA used in this study (21A VDOT aggregate).

Each sample was conditioned for 500 cycles (σ_3 = 15 psi (103 kPa) and σ_1 = 30 psi (207 kPa) prior to the 10,000 cycle test. The results of these laboratory tests are shown in Figure 1. The results showed that testing conditions with 25 psi (173 kPa) deviator stress (i.e., σ_1 = 30 psi (207 kPa) and 5 psi (34 kPa) confining stress resulted in highest strain rate and greater permanent strains.

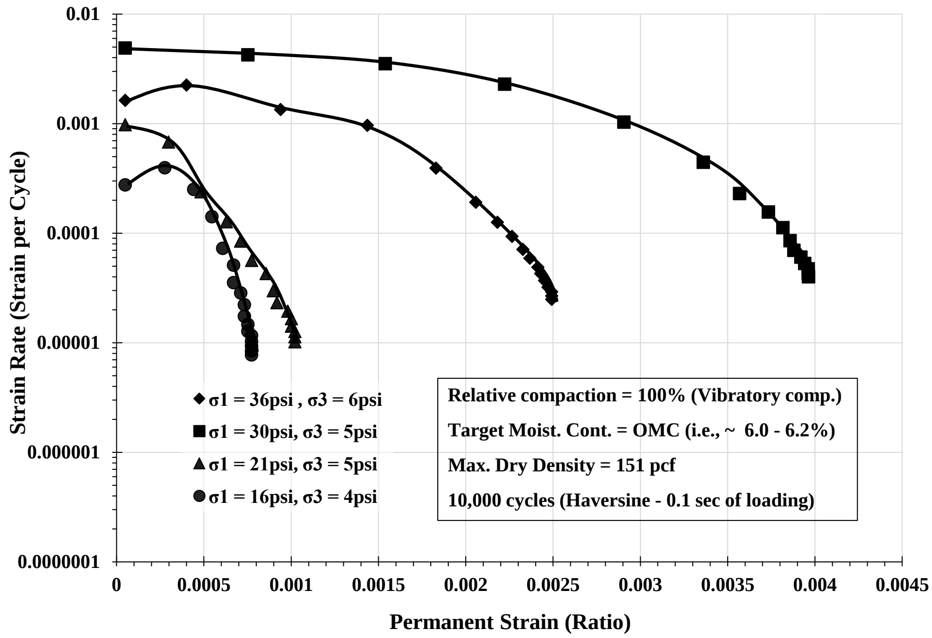


Figure 1: Determination of Stress Levels That Would Yield Maximum Strain in the Permanent Deformation Test

Based on the above-described evaluations and the approval of the Technical Review Panel (TRP) of this project, the following testing protocol has been established in this study to evaluate the performance of the RAP and RAP-VA blends (Ullah and Tanyu 2019).

Table 3: Selected Parameters for Permanent Deformation Test

PD test parameter	Selected Value	Reference
Stress Level	σ_1 = 30 psi, σ_3 = 5 psi (1 psi = 6.89 kPa)	Chow 2014
Shape of impulse loading	Haversine load impulse	Loulizi et al. 2002
Conditioning cycles	500 cycles	Tanyu et al. 2005
Loading cycles	10,000 cycles	Tanyu et al. 2005 Puppala et al. 2005, Mohammad et al. 2006, Chow 2014

Note: σ_1 and σ_3 represent the vertical and horizontal (confinement) stresses.

California Bearing Ratio (CBR) Test

CBR test is a common test conducted by the VDOT to quickly assess the stiffness of the UAB. The testing procedure is outlined in VTM-8 (2014) and ASTM D1883 (2016). These procedures were followed in this study. CBR test evaluates the stiffness of the material in terms of static load application, therefore it is not a direct comparison to understand the performance of the base course material under repeated traffic loads. Nevertheless, CBR tests were included into the testing program in this study because the CBR test results provide some indication on the rutting potential of the material as the higher CBR values indicate stiffer ground conditions. Also, many DOTs are familiar with this test and the values associated with the results.

Methods Implemented to Address Task 2

When it comes to quality control, there is a discrepancy in the literature that although some DOTs refer to the use of nuclear density gauge with no further instructions, the previous researchers indicate a limitation for this device to be used in the presence of RAP. This limitation is primarily attributed to the incorrect determination of the water content of the RAP and RAP-VA blends by the NDG. However, the literature also indicates the need for specific calibration requirements even when the NDG is only used to determine wet density in the case that the tested material contains hydrocarbons as in the case of RAP. Combination of these issues create a complex problem requiring the user to sort it out. Therefore, as an alternative to NDG, other commonly available tools for construction quality control were considered. The most commonly available of these tools include DCP, soil stiffness gauges (SSG), and light weight deflectometer (LWD). The findings from the literature show that DCP is not always suitable to be used with aggregates as the individually larger particles affect the repeatability of the results (Siekmeier 2009, Hossain and Apeagyei 2010). Although SSG could be used for this purpose, when used with aggregate particles, the repeatability of results often is questioned and the depth of influence of the equipment becomes a limiting factor compared to the typical lift thicknesses of base course layers (Alshibli et al. 2005 and Chang et al. 2011). Based on the initial evaluations conducted in this study, the TRP for the project has agreed that LWD seemed to be a suitable instrument to be evaluated to address the task 2. Therefore, this research has focused on assessing the implementation of LWD as a quality control tool for the UAB constructed with RAP-VA blends. However, in order to be able to confirm the suitability of this equipment, three evaluation methods had to be implemented: large-scale model evaluation (LSME), bench scale evaluation in the laboratory, and field scale model evaluation (FSME). The purpose of the LSME was to create a testing platform where only the performance of the RAP-VA blends could be tested without the interference of the concrete floor in the laboratory. Bench scale laboratory tests were conducted to develop target modulus values that could be used in the field. FSME was conducted to relate the effects of the depth of influence of the LWD on to the field conditions where the thickness of the constructed base course could be thinner and the LWD results be influenced by the layer below the base course (e.g., subbase or subgrade).

In addition to evaluating the suitability of LWD to assess the compaction of the unbound base course, another tool had to be evaluated to confirm that the base course has the proper moisture content. This is because even though LWD measures modulus (not density as in the case of NDG), modulus of compacted materials is also affected by the moisture content (Haider et al. 2014*)*. LWD equipment does not provide such information. Therefore, this study has also evaluated the use of speedy moisture content tool to determine the moisture content of the UAB constructed with RAP-VA blends.

The subsequent sections describe the details of the LWD equipment used in this study, LSME, bench scale laboratory evaluation, FSME, and evaluation of the speedy moisture content device.

Light Weight Deflectometer (LWD) Testing Device

LWD operates based on the principals of wave propagation created by a weight falling on a circular plate that is placed over the soil (Fleming et al. 2000, Ebrahimi and Edil 2013). Dynatest manufactured the equipment used in this study, model no. 3031 (Dynatest 2010), although a variety of other similar equipment exists in the market (Khosravifar 2015). The LWD tests in this study were conducted using the standard 22 lb (10 kg) weight, which was dropped from the maximum height of 33.5-in (0.85 m) on to the plate. The plate diameter of the equipment used in this study was interchangeable between 6-in (0.15) and 12-in (0.3 m), which resulted in the depth of influence of the equipment to be approximately equal to the diameter of the plate used (Mooney and Miller 2009). At each testing location, the weight of the LWD was dropped eight times, and the average of the last six drops was used in evaluating the elastic modulus and stiffness values. The first two drops were not used in the calculations to determine the average modulus because, at this stage, the equipment establishes full contact with the testing surface. The precision (or the repeatability) of the results from the LWD tests were determined based on the coefficient of variation (COV) values that were calculated from the last six drops of the weight at the same location. The target for maximum acceptable COV was conservatively set as 15% per ASTM E2583 (2015) guidelines.

Large Scale Model Evaluation (LSME) in the Laboratory

In order to replicate the field conditions as closely as possible within a laboratory environment and to eliminate the effects of LWD's depth of influence, a test pit that is 4 ft (1.2 m) long, 2.5 ft (0.8 m) wide, and 3 ft (0.9 m) high was used to conduct LSME (Figure 2). This set-up allowed the materials to be conditioned and compacted simulating the field conditions. The purpose of the LSME was to create a uniform layer that was thicker than the depth of influence of the LWD so that the modulus of RAP or RAP–VA blends could be determined without the influence of any kind of boundary conditions. The dimension of the test pit was selected based on the evaluations that showed that the chosen dimensions would eliminate the boundary effects, where the applied load in the middle propagates towards the edge but does not bounce back. To confirm this effect, during the experimentation, stress measurements were also obtained from vertically and horizontally placed earth pressure cells. This information confirmed that when the load is dropped from the LWD instrument at the top layer of the LSME, no significant stresses are recorded from the earth pressure cells. This information was used to confirm the 2:1 stress distribution assumption and the depth of influence of the LWD used in this study.

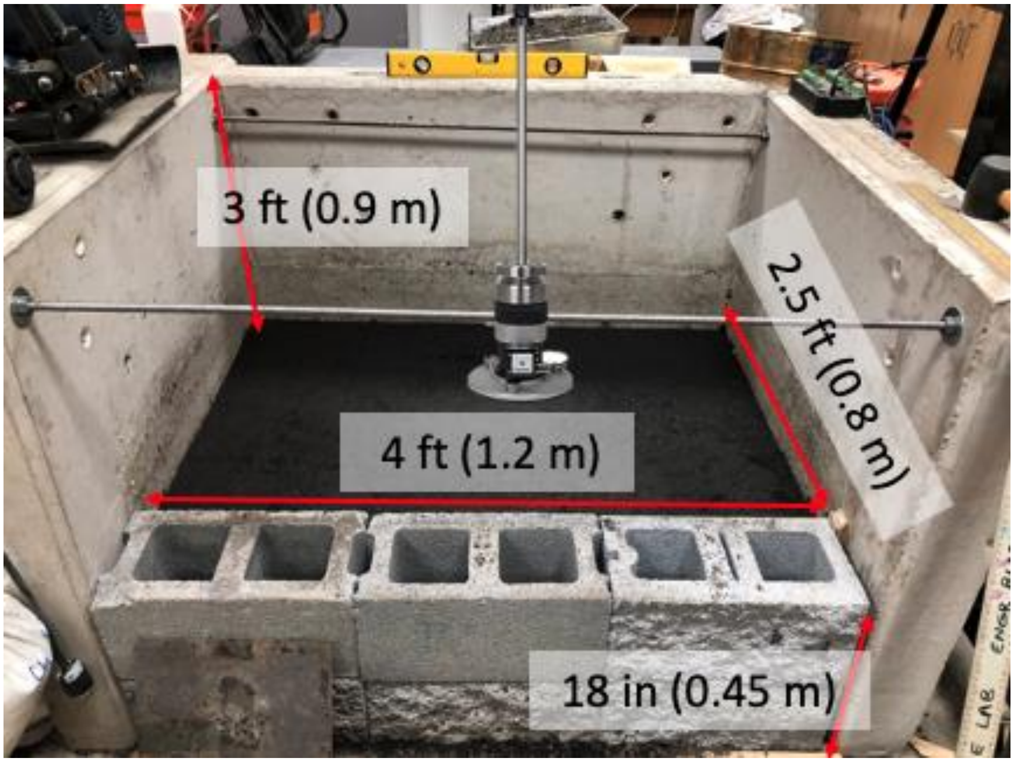


Figure 2: Large Scale Model Evaluation (LSME) Set-Up Used to Test Suitability of Light Weight Deflectometer (LWD) in George Mason University's Sustainable Geo Infrastructure (SGI) Laboratory

LSME tests were conducted with 100% VA, 100% RAP, and RAP-VA blends. All tested materials were first mixed using a cement mixer (simulating pugmill conditions) and water (based on their optimum moisture content) was added. After this process, the material was then spread into the test pit with a shovel in lifts. Each layer was targeted to be 6-in (0.15 m) thick after compaction. The compaction of each layer was achieved by using an electric plate compactor, which had a depth of influence of 8-in (0.2 m), as reported by the manufacturer. Sand cone test (per ASTM 1556) was used in each layer to confirm the consistency of the proper compaction. All materials were placed into LSME in three lifts resulting in a total height of 18-in (0.45 m). Each lift was compacted to relative compaction of 95% and at optimum moisture content. LWD tests were conducted at each 6-in (0.15 m) thick layer to confirm the effects of the layer below. Hence the only data used to confirm the suitability of LWD was the results obtained from the last layer (where the depth of influence does not affect the results). The modulus values obtained in this fashion were recorded as E_{LSME} in this study.

Bench Scale Evaluation in the Laboratory

Performing LWD tests in a test pit based on LSME was important to evaluate the depth of influence of the LWD equipment. The purpose of the bench scale evaluations was to find a suitable testing protocol in the laboratory where target modulus values could be obtained so that in the case of field measurements, these target values could be used to determine pass or fail decisions. This was an important evaluation as not all users of LWD may have a LSME set-up, where a true target value could be determined. Figure 3 shows the set-up of the bench scale evaluations where LWD equipment was directly placed over the conventional 6-in (0.15 m) compaction mold.

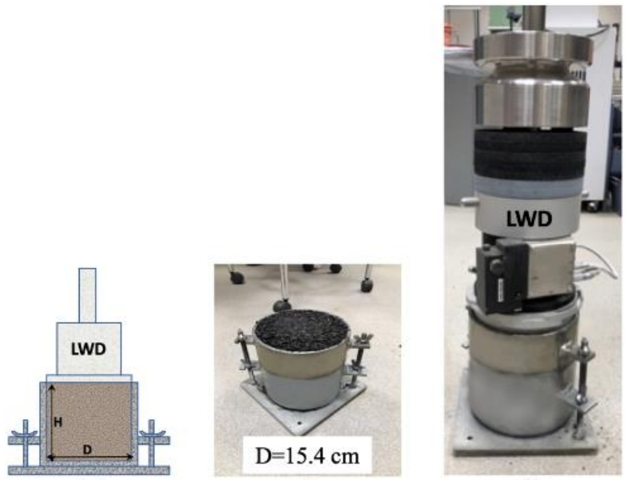


Figure 3: (a) Schematic of the Light Weight Deflectometer (LWD) Test Set-Up with Compaction Mold, (b) Compaction Mold with the Sample, and (c) Photograph of the Assembled Light Weight Deflectometer (LWD) over the Mold

The LWD equipment used in this study had the capability to reduce the diameter of the footprint to 6-in (0.15 m), which allowed the LWD's bottom plate to directly fit over the material compacted in the mold (Figure 3c). It is recognized that the elastic modulus values measured in a compaction mold (E_{Mold}) may have boundary condition issues due to the size of the set-up (assuming that a similar 2 to 1 stress distribution). Therefore, a correlation is created in this study to relate the LSME results to the values obtained from the compaction mold.

In the compaction mold, all materials were compacted in equally divided three lifts using a vibratory compactor to the same target relative compaction (density) used in the LSME. The methods used to obtain modulus values from the mold were exactly the same as in LSME. The elastic modulus from the mold tests (E_{Mold}) was estimated using Equation 5 as determined from previous studies (Khosravifar 2015) as:

$$E_{Mold} = \left(1 - \frac{2\,v^2}{1-v}\right) \frac{4H}{\pi\, D^2} k \qquad \text{Equation 5}$$

where; H is the height of the mold, D is the diameter of the plate/mold, *v* is Poisson's ratio of the material placed into the mold, k is the soil stiffness provided by LWD which is estimated as the ratio of the applied peak load (F_{Peak}) to the corresponding measured deflection (δ).

Field Scale Model Evaluation (FSME)

In order to validate the correlation between E_{Mold} and E_{LSME} and to assess that LWD equipment can be used even in the case of a base course layer that is thinner than the depth of influence of the LWD (i.e., creating a multi-layer effect) a field trial was implemented. This effort is referred to herein as field-scale model evaluation (FSME). In this evaluation, the base course was placed into a wooden box that was 0.7 ft (0.2 m) high, 4 ft (1.2 m) wide, and 4 ft (1.2 m) long (Figure 4). The box was directly placed over the subgrade in the field. The subgrade classified as low plastic silt (ML), and both the subgrade and the base course were compacted with the same handheld electric compactor that was used during the LSME.

LWD tests were performed on both subgrade and the unbound aggregate placed inside the wooden box representing the base course layer in actual construction. The results obtained were evaluated in light of implementing the laboratory determined target modulus value to confirm suitable compaction as well as how to handle the LWD measurements when the conditions result in a multi-layer scenario. The subsequent section describes the details of evaluating the multilayer systems.

Evaluating the multilayer system

The thickness of the base course layer may vary depending on the design. During construction, typically, the maximum base course thickness installed in one lift is determined based on the depth of influence of the compaction equipment. When the first lift of the base course is placed on the subgrade and compacted, in the majority of the circumstances, its thickness will be smaller than the depth of influence of the LWD. Since each lift of the base course must be tested, this condition will create a multilayer system when the LWD measurements are taken from the surface of the base course. Additionally, the total base course thickness may create a multilayer system (Han 2014) because, for secondary roads in the U.S., the typical pavement design guide requires the minimum thickness of the base course to be 6-in (0.15 m) (VDOT Road and Bridge Standards 2016). In order to account for the multilayer system in the evaluation of the LWD results, a model must be created to take into consideration the influence of the elastic modulus of both of the materials (e.g., subgrade and base course) within the influence zone of the LWD (Figure 5).

When the weight of the LWD is dropped, it is expected that the soil will undergo a settlement, and in the case of LWD measurements, this settlement is referred to as surface deflection. Balunaini et al. (2013) used a finite element approach to provide a method to relate this surface settlement to the applied load and elastic properties of the layers. Balunaini et al. (2013) have compared their finite element results to Steinbrenner's (1934) classical approach to estimate surface settlement for a multilayer system and showed that there was an almost perfect match. Hence in this study, Steinbrenner's approach given in Equation 6 was implemented. The re-arrangement of this equation allows the estimation of the elastic modulus of the top layer (Equation 7).

$$\Delta_z|_2 = q_{surface} D \left[\frac{(1-v_1^2)}{E_1}(I_{p1} - I_{p0}) + \frac{(1-v_2^2)}{E_2}(I_{p2} - I_{p1}) \right] \qquad \text{Equation 6}$$

$$E_1 = \frac{(q_{surface} D)(1 - v_1^2)(I_{p1} - I_{p0})}{\Delta_z|_2 - (q_{surface} D)\left[\frac{(1 - v_2^2)}{E_2}(I_{p2} - I_{p1})\right]} \quad \text{Equation 7}$$

where, E_1 is equal [illegible] the top layer, and E_2 represents the measured elastic modulus of the bottom layer (as shown in Figure 1), $\Delta_z|_2$ is the settlement of the two-layer system that is measured by the LWD equipment, $q_{surface}$ is the applied stress by the LWD, D is the diameter of LWD plate, ν_1 and ν_2 are the Poisson's ratio of top and bottom layers. Ip_0, Ip_1, and Ip_2 are symbolizing the corresponding settlement factors for various depths.

Figure 4: Field Scale Model Evaluation (FSME) Testing System

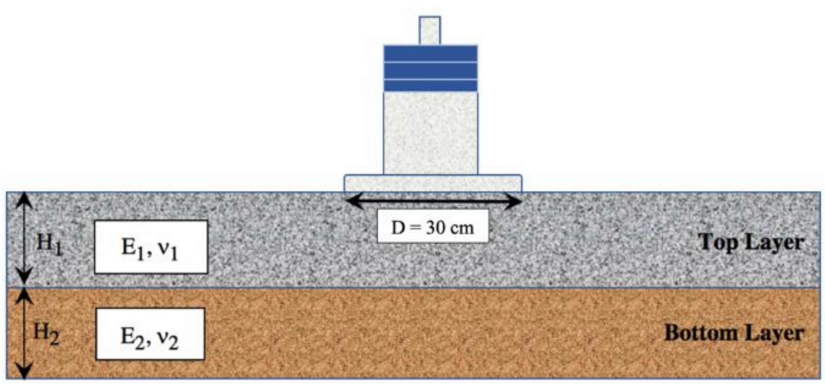


Figure 5: Schematic of the Multilayer System

Equation 7 provides an opportunity to back-calculate the modulus of the base course when the multi-layer scenario exists because the load applied from the LWD is known, and the surface deflection is measured by the equipment. For this calculation to be possible, the modulus of the subgrade and the Poisson's ratios of both the subgrade and base course has to be known. In this study, it is proposed that the best way to determine the elastic modulus of the subgrade is to measure this value with LWD directly on the surface of the subgrade prior to the placement of the base course at the same location where LWD measurements for the base course will then be conducted. It is further proposed that based on previous literature, the Poisson's ratios of the base course and subgrade were assumed as 0.3 and 0.45 respectively unless specific laboratory values are determined (Schwartz et al. 2017, Steinbrenner 1934).

The settlement factors referred to in the above equations were developed by Vesic (1963) as a function of Poisson's ratio and depth of the layers. At the surface ($z = 0$), Ip_0 equals to zero. At the interface of the two layers (which corresponds to the bottom of the first layer, $z = H_1$), the settlement factor is represented by Ip_1. At the bottom of the second layer, which is defined by the LWD's depth of influence ($z = H_1+H_2$), the settlement factor is represented by Ip_2. The settlement factors (Ip_1 and Ip_2) that were used in this research are given in Table 4.

Table 4: Settlement Factors Used in the Multilayer Analyses in This Study

Depth ratio (z/a)	Settlement Factors (see Eq. 6)	
	Ip_1 (Base course)	Ip_2 (Subgrade)
0.5	0.253	0.229
1.0	0.505	0.459
1.5	0.565	0.524
2.0	0.624	0.588
2.5	0.659	0.626
3.0	0.694	0.664
3.5	0.718	0.691
4.0	0.743	0.718

Note: z/a = ratio of depth to the radius of the LWD plate

Speedy Moisture Content Testing Equipment

The speedy moisture content device works based on the principals of moisture of the material (free water) to react with calcium carbide that is added to the device to form acetylene gas as depicted in the following chemical reaction below:

$$CaC_2(s) + 2H_20(l) \rightarrow Ca(OH)_2(s) + C_2H_2(g)$$
$$Calcium\ carbide + water \rightarrow calcium\ hydroxide + acetylene\ gas$$

Once the acetylene gas (C_2H_2) is generated, the pressure created by forming this gas is recorded by the pressure gauge on the speedy moisture content test device (AASHTO 2014). The higher the moisture in the material, the higher the measured pressure reading becomes. The standard procedure on how to conduct the speedy moisture content tests is outlined in the AASHTO standard T217 (AASHTO 2014), where empirical charts are used to relate the gas pressure to a moisture content of the material. The standard procedure to perform this test suggests placement of 20 grams of sieved (No. 4) soil into the speedy moisture test device chamber, adding three scoops of calcium carbide, two ball bearings that are 31.75 mm in diameter, and spinning the material for 3 minutes followed by a one-minute resting time (AASHTO 2014). The ball bearings that are placed inside the speedy moisture content device with the soil is used to pulverize the sample.

AASHTO T217 (2014) states that the speedy moisture content method may not be accurate for granular materials that contain particles larger than No. 4 (4.75mm) sieve. This is because the coarser particles may not be totally pulverized by the ball bearings used in the test, and this discrepancy may affect the accuracy of the test. The aggregates used in this study had particles larger than the No. 4 sieve. If the coarser fraction of the particles is removed, this would satisfy the maximum particle requirement for the test but would also change the percentages of particle distribution in a given blend. This challenge was overcome by increasing the sample weight to become a minimum of 30 grams (as opposed to the 20 g suggested by the AASHTO T217 standard). This amount was determined empirically based on the number of tests conducted in this study. This procedure resulted in a deviation from the AASHTO T217 (AASHTO 2014) standard, and therefore a correction factor had to be created for the charts suggested by AASHTO T217 (2014) to estimate moisture contents. This correction was achieved by dividing the pressure reading from the chamber of the speedy moisture device to the ratio of the weight of the sample placed into the chamber in grams to the reference amount (i.e., 20 grams). The procedure described herein is referred as modified speedy moisture content test. In addition to the speedy moisture tests, each sample was also air-dried, and the results were compared against the moisture content values estimated from the speedy moisture test. This comparison was used to confirm the accuracy of the measurement from the speedy moisture test to estimate air-dried moisture values. The results were not compared against the oven drying method, as oven drying does not provide the most accurate results for the RAP and RAP-VA blends.

Methods Implemented to Address Task 3

The purpose of the field-testing program was to verify the applicability of the developed methodology in this research to confirm that the laboratory driven RAP thresholds (as determined from task 1) were correct in a given blend. Additionally, the field-testing program was implemented to confirm the use of the LWD and speedy moisture content devices as a quality control equipment (as determined from task 2) and to evaluate the field performance of the RAP–VA blends under the influence of traffic and seasonal effects.

The project involved the construction of a road section at Minnieville, Virginia, to create test sections with RAP blended with VA to create the base course for a roadway. The location of the site was near US-234. Six sections were constructed but they only covered one of the westbound lanes (Figure 6).

Each test section was 150 ft (46 m) long and 12 ft (3.65 m) wide. Figure 7 shows the area within the westbound lane and the activities during construction. In all sections, the thickness of the HMA layer was 9.5 in (0.24 m) and base course was 6-in (0.15 m) thick. There was no subbase at the site and the subgrade classified as low plastic silt based on the Unified Soil Classification System (USCS). The average unsoaked CBR of the subgrade was 20, and the average Atterberg Limits were LL=46 and PL=34 (PI=12). Two of the six sections were constructed with 100% VA (classified as 21A) and the remainder were constructed with blends of RAP and VA with two different RAP percentages and binder contents. Only one of the 100% VA section was instrumented but both sections were tested for performance evaluation after construction. As part of this research, two different field methods to blend RAP and VA have been documented. One method that is achieved at the quarry using the front end-loader and pugmill and another method conducted at the asphalt plant based on utilizing cold bin silos and shaker. Summary of these procedures and the comparison to the results obtained from laboratory blending are provided in the Appendix. Although VDOT in general prefers the pugmill mixing, due to the proximity of the source, RAP used in the Minnieville field site was blended with VA based on the method that was available at the asphalt plant. This was an acceptable method based on the observations summarized in the Appendix.

Figure 6: Site Plan Layout for Field Test Sections

The specific percentages and binder contents were determined from the results of the study in task 1. Field-testing program consisted of three different evaluations: (1) quality control with LWD and speedy moisture content tests, (2) instrumentation embedded into the base course layer to evaluate the effects of seasonal changes within a year period, and (3) testing after the construction of the roadway using LWD, FWD, and IRI equipment to measure the surface rutting and back calculated modulus values of the base course. Subsequent sections describe the methods used for each of these evaluations.

Figure 7: Photos of Activities From Field Construction

Quality Control During Construction

LWD and speedy moisture tests were performed to control the compaction and moisture content of the base course sections as part of the quality control during construction. In terms of LWD measurements, each test section was divided into four subsections, and at each of these subsection locations, LWD tests were performed on both subgrade and base layers. For each LWD test location along the base course, moisture content tests were also conducted with the speedy moisture content device. To also confirm the results from the speedy moisture device, additional grab samples were obtained and air dried at GMU laboratory. The results from the air drying and speedy moisture tests were then compared with each other to determine the accuracy of the field measurements. To confirm the limitations, nuclear density gauge was also used in all sections to estimate the dry unit weight and the moisture content of the materials used for base course construction. The readings were obtained using the direct transmission method and by setting up the depth of the nuclear probe to penetrate approximately 3.15 in (80 mm) into base course layer (approximate mid-height). The water content estimates obtained from the nuclear density gauge were compared against the moisture content values determined from the speedy moisture content and air-dry tests.

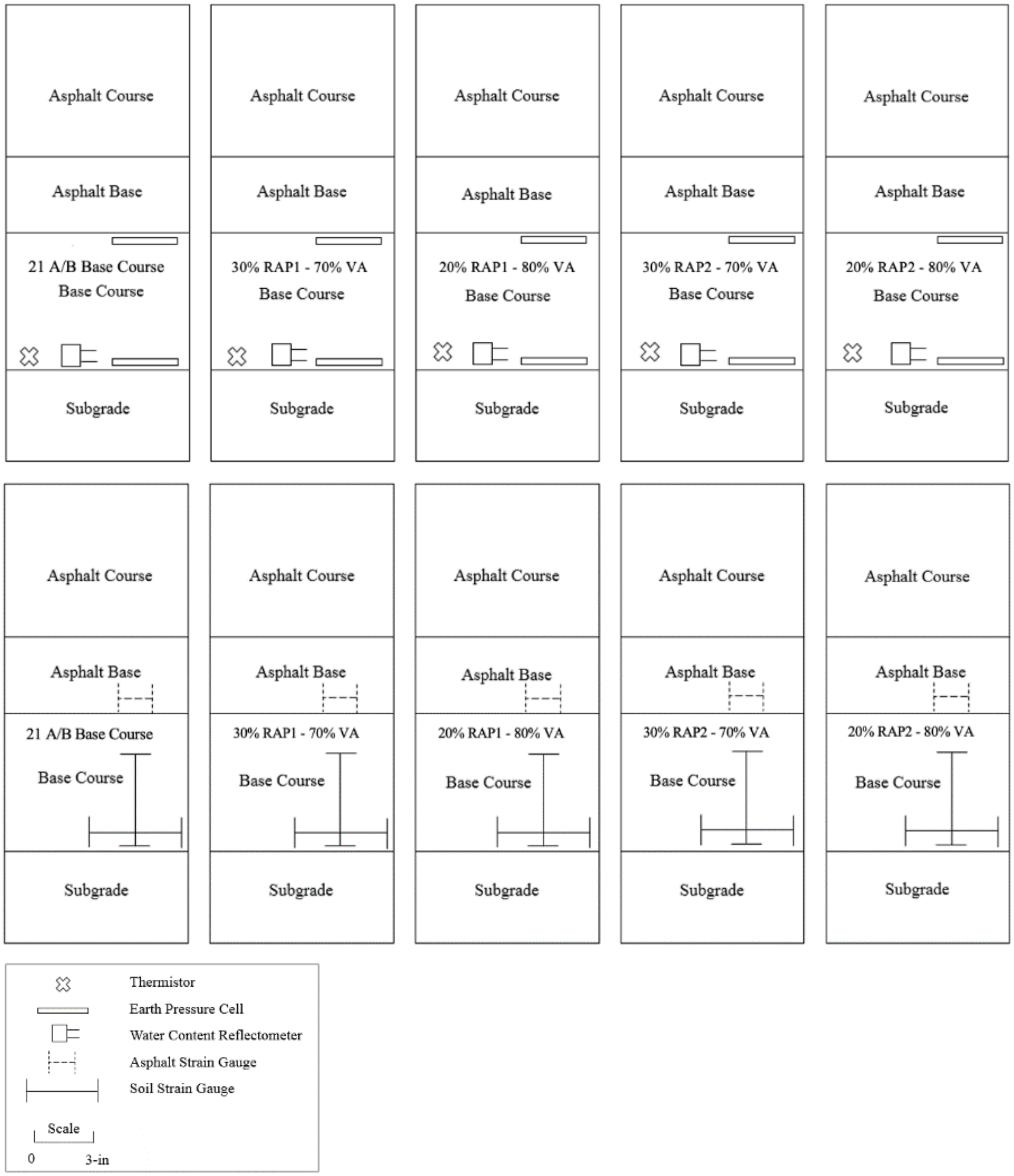
Subgrade was characterized based on the soil samples collected along the length of the test sections. Samples were obtained before and after compaction of the ground. Grab samples obtained before compaction were used to evaluate grain size distribution and Atterberg limits. Samples obtained with density tubes were used to determine density and moisture content after compaction. Additionally, LWD, dynamic cone penetration (DCP), soil stiffness gauge (SSG), and sand cone tests were also performed after the subgrade was compacted. Grab samples were reconstituted in the laboratory based on field density and water contents and were also tested to determine the CBR of the subgrade in unsoaked condition (as observed in the field).

The thicknesses of the base course layers were thinner than the depth of influence of the LWD equipment. Therefore, all LWD measurements from the top of the base course layer had to be interpreted to take into account the multi-layer effect and to back calculate the specific modulus of the base course. Therefore, implementation of the multilayer system procedure proposed in this study was necessary in order to estimate the modulus of the base course aggregate. However, in order to implement this approach, it was necessary that the LWD measurements from the subgrade and base course be conducted at a location that was very close to each other. In order to achieve this, locations of the subgrade tests were marked in the field using red flags along the side of the road. At each of these locations, sticks with colored flags were placed into the ground, and the height of the sticks remaining above the subgrade was measured before the base course is placed. After placement and compaction of the base course layer, the remaining height of the sticks was re-measured to calculate the thickness of the base course at that particular location. LWD tests were then conducted on top of the base course for each test point, where previously subgrade was characterized. The modulus determined at these locations represented the multilayer that consisted of both the base course and the subgrade. The procedures implemented to perform LWD testing in the field were exactly the same as for the laboratory tests and FSME. At each location, completing the set of LWD measurements took about 5 to 7 minutes to obtain and analyze data.

Instrumentation Embedded into the Base Course

Test sections 1 thru 5 at the field site were instrumented with a set of performance monitoring and meteorological devices to establish a correlation between the response of the material and causality factors. The response of the bases constructed with RAP-VA blends were compared to VA as control material. The installed instruments have been divided into two categories, performance monitoring and meteorological devices. Performance monitoring devices measure changes in pressure and deformations within the pavement structure. Deformation changes within the base course are recorded both horizontally and vertically. While meteorological devices record water content and temperature changes. Both sets of instruments record data on an hourly basis. All sections constructed with RAP-VA blends and one of the control sections constructed VA were instrumented with both type of devices. Meteorological devices measure changes in water content and temperature within the base course layer.

Knowing changes in water content and temperature provides understanding between causality factors and performance of the base course in terms of variation pressure and deformation. Earth pressure cells and strain gauges within the base course provide information related to the performance of the base course material. Figure 8 provides schematics of instrumentation layout deployed within the base course layer of the test sections. Figure 9 shows the pictures of the instruments installed in the field.



Note: Section 6 with VA was not instrumented

Figure 8: Instrumentation Layout of Test Sections

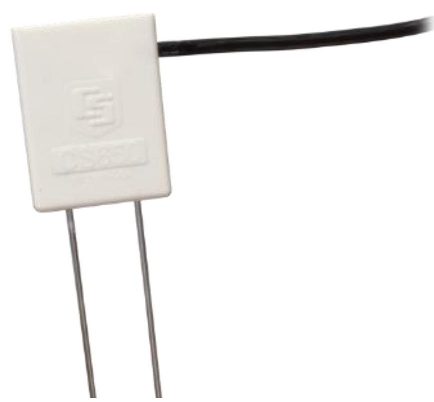
(a) Soil Displacement Gauge

(b) Soil Displacement Gauge

(c) Earth Pressure Cell

(d) Asphalt Strain Gauge

(e) Data Acquisition System



(f) Water Content Reflectometer

Figure 9: Installed Instrumentations

Testing Conducted After Construction

After the completion of the field test site (placement of the HMA layer), each test section was evaluated with FWD test, International Roughness Index (IRI) profile tester, and LWD tests. The construction of the site was completed in September 2018 and the surveys were conducted in November 2018, April 2019, July 2019, and December 2019 to capture the changes that may be due to seasonal changes within a one year period.

LWD equipment used at the field site was the same equipment used for the laboratory evaluations and was owned and operated by GMU's SGI research team (Figure 10a). The locations of each LWD test were in line with the previous locations where LWD tests were conducted at the base and subgrade levels. Two different LWD plate diameters were used in this testing program as 6-in (0.15 m) and 12-in (0.3 m). Using the 6 in (0.15 m) plate helped capture only the elastic modulus of asphalt layer without having the influence of the base course and using the 12-in (0.3 m) plate helped determine the modulus of the base course via multi-layer back calculation procedure. LWD can produce load between 3,370 and 8,500 lbf (6 and 15 kN). The first two drops of the LWD tests were ignored and remaining six drops were used in the analyses. The same target COV that was use in the laboratory program was also implemented in the field to confirm repeatability. At each section, three LWD tests were conducted.

FWD tests were conducted by Wood Environment and Infrastructure Solutions, Inc (Wood). The tests were conducted with Dynatest model 8002 equipment (Figure 10b). Deflections were recorded at 0, 8, 12, 18, 24, 36, 48, 60, and 72 inches, (0, 20, 30, 46, 61, 91, 122, 152, and 183 cm) from the center of the load plate. The diameter of the FWD plate diameter was 12 in (30 cm). At each location six readings were obtained however the first one of the readings was not used in the analyses. Similar to LWD at each section, three FWD tests were conducted at each test section. During each test, at each location 6,000, 9,000, and 12,000 lbf (24, 40, and 54 kN) loads were produced. The data obtained from the FWD tests were provided to GMU's SGI research group and the interpretation of this data was performed by the GMU team in light of additional analyses performed with a software program referred as Modulus 6.0.

IRI tests were also conducted by Wood (Figure 10c). The IRI values were obtained from a single longitudinal profile measured with a road profiler in both the inside and outside wheel paths of the pavement. In this study, two IRI tests were performed at each testing period along each longitudinal profile. The average of these two IRI statistics is reported as the roughness of the pavement section. This practice outlines standard procedures for measuring longitudinal profile and calculating the IRI for highway pavement surfaces to help produce consistent estimations of IRI for network-level pavement management (ASTM E1926). Tests were performed at a vehicle speed of 50 mph (80 km/h). The start and end of the measurements were consistently marked at the same locations each time the surveys were conducted. The results of the surveys were provided to GMU by Wood and the interpretation of the results were made by GMU.

(a) **(b)** **(c)**

Figure 10: Testing Devices Used in This Research (a) Light Weight Deflectometer, (b) Falling Weight Deflectometer, and (c) International Roughness Index

RESULTS

The results of the research described in this report have been grouped in four different categories as presented in the subsequent sections. These sections were outlined to present the results associated with characterizing the RAP and VA materials used in this study followed by sections associated with addressing the tasks 1, 2, and 3.

Characterization of Materials

RAP is a recycled material and the properties vary from location to location. In order to characterize the RAP in Virginia, based on the coordination with VTRC and Virginia's Asphalt Industry, the research team has visited fourteen different asphalt plants and obtained samples. The specific locations where the samples were collected were determined based on the abundant availability of RAP at a given location. Approximate locations of these plants are outlined in Figure 11. As can be seen from this figure, the locations coincidentally were clustered mostly in the northern Virginia section. At each of these locations, RAP has been produced by the plants following the production of fine processed RAP. During production, GMU's research team was present at the plant. Samples have been obtained immediately after this production. The results from the grain size distribution analyses (per ASTM D6914) of all the samples are shown in Figure 12.

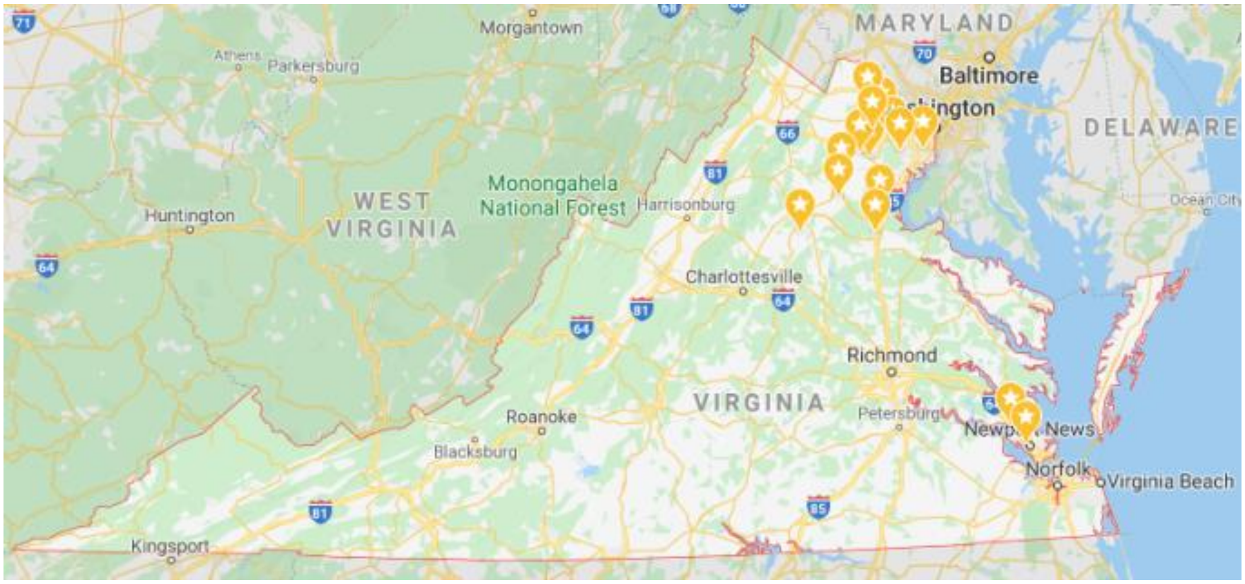


Figure 11: Approximate Locations of Asphalt Plants Where Reclaimed Asphalt Pavement (RAP) Samples Were Obtained

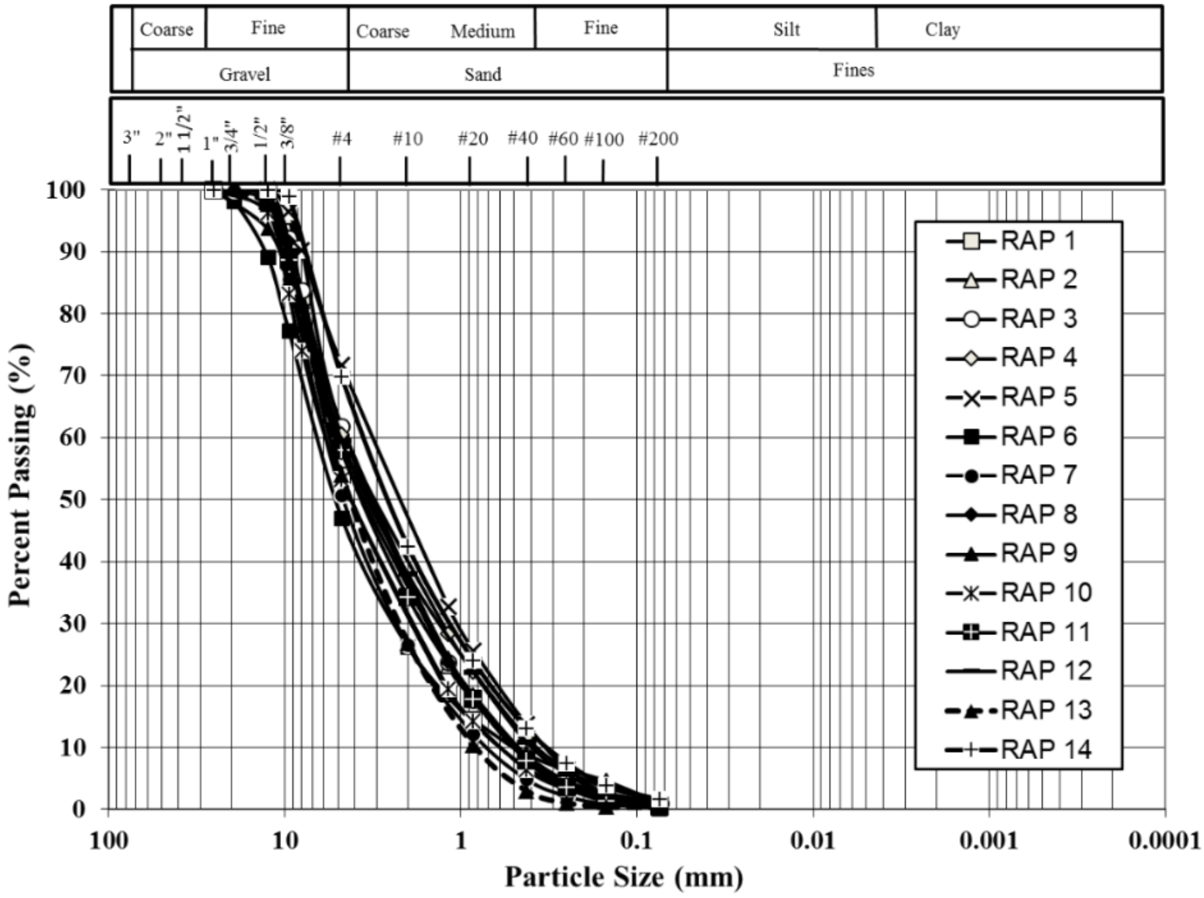


Figure 12: Grain Size Distribution of Collected Reclaimed Asphalt Pavement (RAP) Samples

Figure 13 presents the variation of the binder content of the collected samples from 14 different locations (each asphalt plant where the samples were obtained was designated with a number in the x-axis of this figure). These binder contents were obtained following the procedures described in the standard test method for asphalt content of asphalt mixtures by ignition method ASTM D6307. The variation of the binder content at each location have been determined from the results of at least 10 specimens and are shown with box plots with whiskers. The line within each box plot indicates the mean value of RAP binder content and the shaded areas above and below the mean value show the variation of the binder content up to one standard deviation. The ends of the whiskers present the minimum and maximum binder content values from each RAP sample location. Based on the whiskers, the binder content of RAP in Virginia range between 4.4% (lowest recorded value) and 6.1% (highest recorded value). However, these values represent the extreme ends of the binder content spectrum. When the mean values from each location are evaluated, the range of binder contents become 4.6 to approximately 5.8%. Within this range, relative to each other, RAP samples of 1, 3, 4, and 10 are considered as RAP with “high binder” content (mean values ranging from 5.6 to 5.8%) and samples of 2, 8, 11, and 14 are considered as RAP with “low binder” content (mean values ranging from 4.6 to 4.8%). All other samples are considered as RAP with “medium binder” content (mean values ranging between 5.1 and 5.4%).

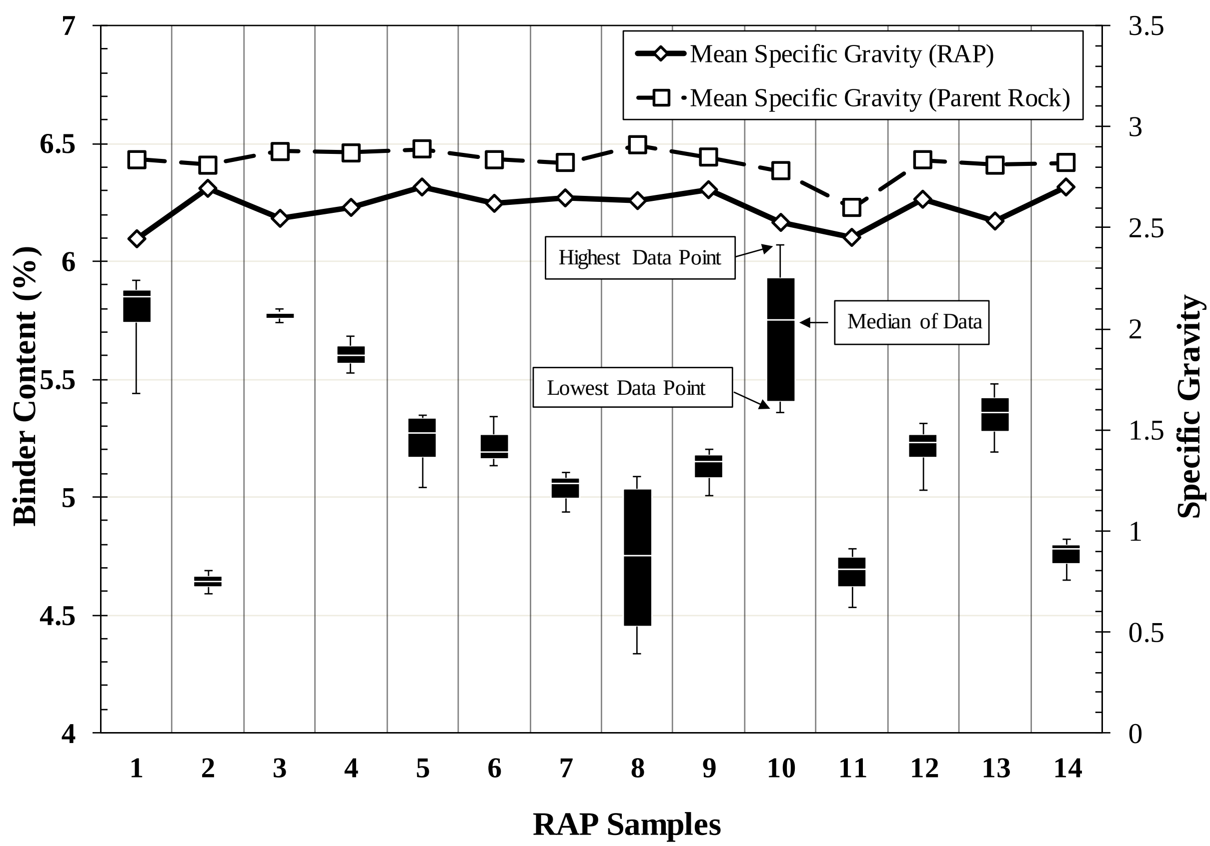


Figure 13: Binder Content and Specific Gravity of Collected 100% Reclaimed Asphalt Pavement (RAP) Samples

Figure 13 also presents the specific gravity of the RAP and the parent rock of the aggregate within the asphalt (RAP). The specific gravity values have been determined following standard test method for bituminous mixtures, ASTM D2041. The specific gravity of RAP was determined from the tests conducted with the samples directly from the field and the specific gravity of the parent rock of the aggregate within RAP was determined from the samples after the asphalt coating of the RAP was burned (exposing the aggregate within the RAP).

Based on the results shown in Figures 13 and discussions with the project's TRP members, in order to evaluate the effects of binder content on the overall performance of the RAP-VA blends, RAP1 (representing the high binder condition), RAP2 (representing the low binder content), and RAP5 (representing a range in between high and low binder content range in Virginia) samples were selected for the detailed performance evaluation and optimization. All of these RAP samples contained aggregate pieces of same origin, which was diabase. Table 5 presents the results of the index property tests conducted on these RAP samples as well as the VA collected in this study.

Table 5: Mechanical Properties of Reclaimed Asphalt Pavement (RAP) and Virgin Aggregate (VA)

Properties	**RAP 1**	**RAP 2**	**RAP5**	**As-Is VA**
D_{10} (in)	0.020	0.020	0.014	0.003
D_{30} (in)	0.078	0.070	0.040	0.030
D_{50} (in)	0.120	0.130	0.083	0.140
D_{60} (in)	0.213	0.180	0.122	0.280
Coefficient of Uniformity	10.65	9.00	8.71	93.00
Coefficient of Curvature	1.43	1.36	0.94	1.10
Gravel (%)	45.0	40.0	29.0	46.0
Sand (%)	53.5	57.8	68.2	42.0
Fines (%)	1.5	2.2	2.8	12.0
Plasticity Index	Non-Plastic	Non-Plastic	Non-Plastic	1.7
Liquid Limit (%)	Non-Plastic	Non-Plastic	Non-Plastic	19.2
USCS Classification	SW	SW	SW	SW-SM
AASHTO Classification	A-1-a	A-1-a	A-1-a	A-1-a
Binder content (%)	5.60 - 5.80	4.50 - 4.70	5.20 – 5.40	N/A
Specific Gravity	2.43	2.60	2.70	2.85
Max Dry Density (pcf) (ASTM D7382)	121	124	122	151
Optimum Moisture Content (%)	5.5	5.5	5.4	6.2

Note: 1 in = 25.5 mm, 1 pcf = 16.02 kg/m3

Figure 14 shows the grain size distribution of the RAP1, RAP2, and RAP5 compared to the range of grain size distribution obtained from all collected RAP samples (the range is shown as upper and lower bounds). When compared with each other, the fines content of all three RAP samples (particles that coincide with #200 sieve) were similar to each other but the percentage of the sand sized particles vary for RAP5 in comparison to RAP1 and 2 samples. Overall comparison of these gradations with the gradation of 100% VA shows that the gradations of RAP differ from the gradation of the VA (Figure 14).

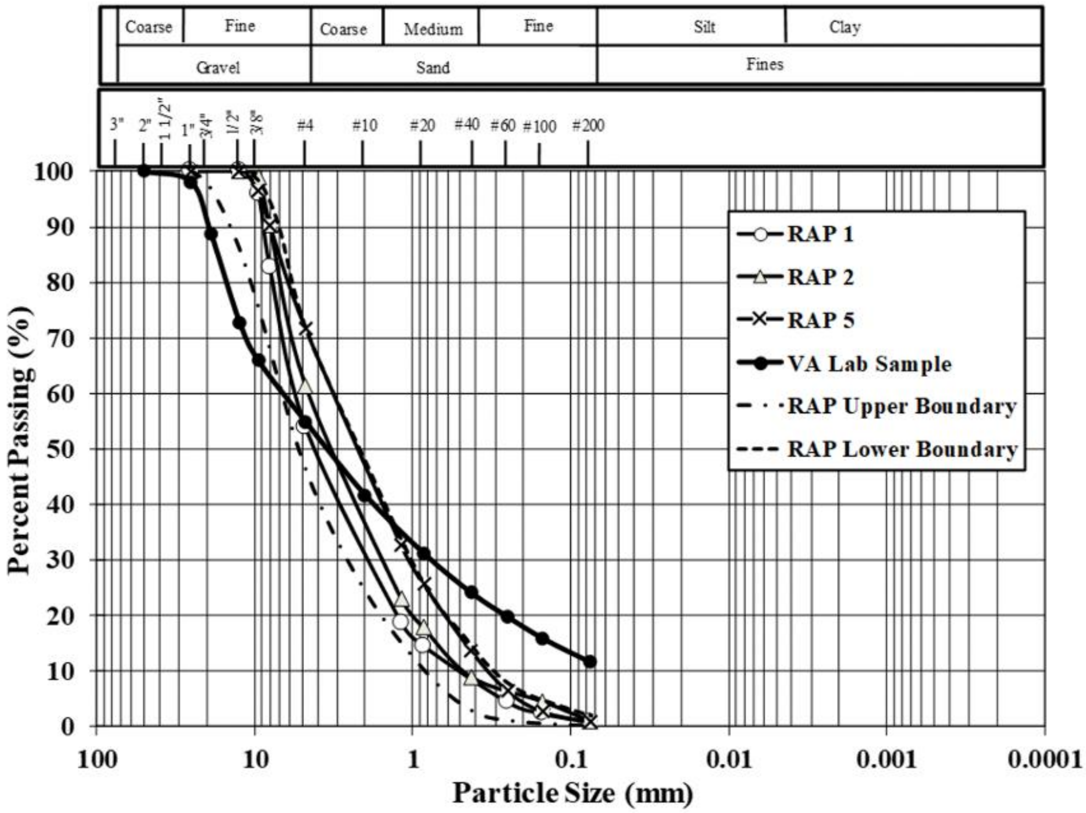


Figure 14: Grain Size Distribution of Reclaimed Asphalt Pavement (RAP) and Virgin Aggregate (VA) Samples Selected for This Study to Create RAP-VA Blends

Considering the differences in grain size distributions of these materials, it was determined that before detailed evaluations were performed, it was necessary to have an agreed upon approach for the most suitable blending method. Therefore, as part of the characterization stage, two different blending approaches were evaluated to determine an optimized method. In one approach, herein referred as engineered mixture gradation, regardless of the RAP percentage, the blends were created to result in one specific gradation. In the other approach, RAP and VA samples were blended by weight. Considering the differences in their as-is grain size distributions ((Figure 14), this approach resulted in RAP-VA blends with gradations that slightly differed from one another based on the percentage of the RAP blended with VA. This approach is referred as as-is gradation method in this report. Results associated with evaluation of both of these approaches are presented in the subsequent sections. For both approaches, RAP and VA were blended to create blends of 20, 30, 40, 50, and 60% RAP and 80, 70, 60, 50, and 40% VA respectively. Performance evaluation of the blends created with these two different approaches was conducted based on the PD test results.

Results from Blending RAP-VA Based on Engineered Mixture Design Gradation

The RAP-VA blends created based on engineered mixture design approach were developed based on the previously suggested method referred as Fuller's curves in the literature (Skermer and Hillis 1970). This method is based on creating a gradation with optimum gravel content where gravel particles end up having particle to particle contact as opposed to floating in a matrix of sand and finer sized particles. Therefore, based on this approach regardless of the RAP percentage in a given RAP-VA blend, the grain size distribution stays the same. Figure 15 presents the engineered design gradation used in this study for the RAP-VA blends and how this gradation compared to the gradations of the 100% RAP and VA (shown as "as-is gradations") and the VDOT's upper and lower gradation limits for 21A aggregate. The results show that the engineered gradation fits within the VDOT's 21A aggregate gradation boundary and produces a material that has approximately 8% fines, 47% sand sized particles, and 45% aggregate sized particles.

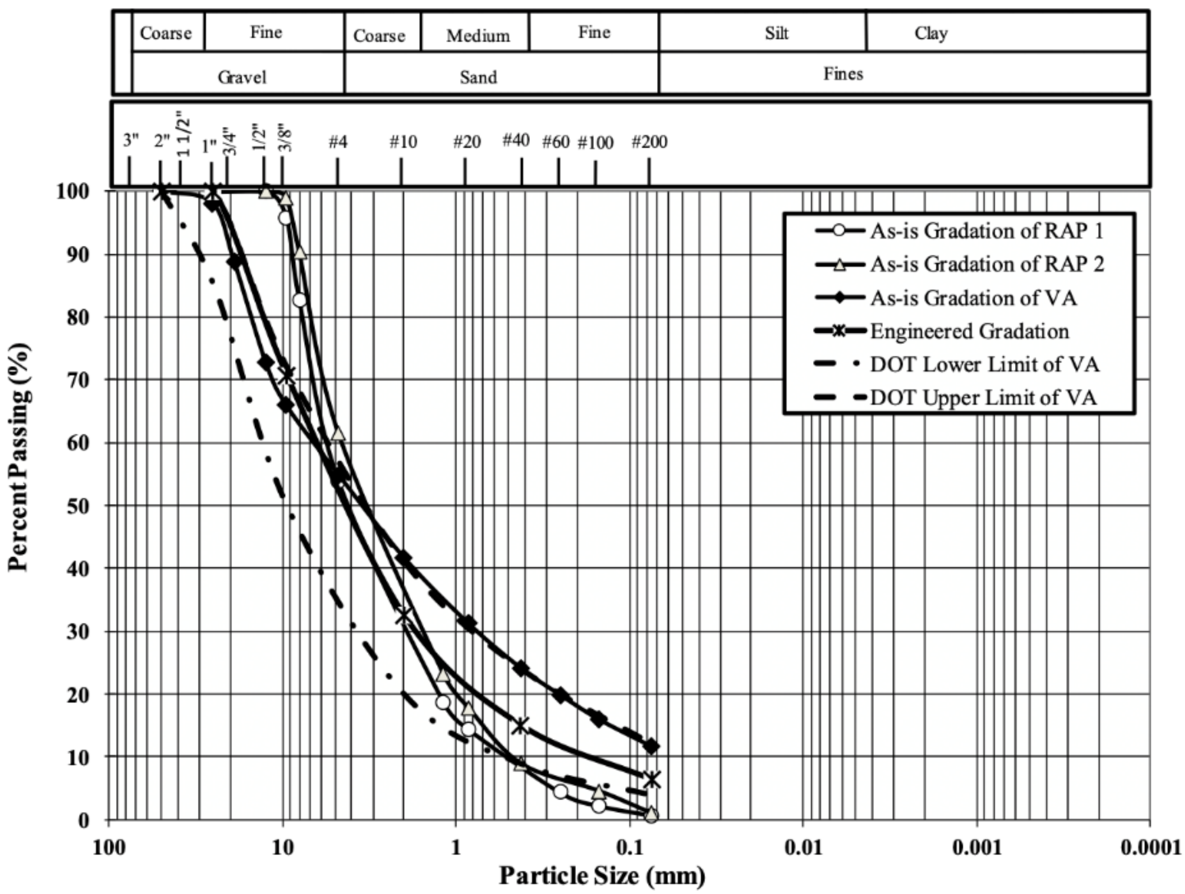


Figure 15: Grain Size Distribution of Reclaimed Asphalt Pavement (RAP) and Virgin Aggregate (VA) Samples Compared with Engineered Gradation

PD test results of RAP-VA blends (with RAP 1 - high binder content and RAP2 - low binder content) created based on engineered mixture design approach are presented in Figures 16a and b. The results show that total accumulated strains at the end of 10,000 load cycle increases with increase of RAP content (i.e. 20 to 60%) regardless of the characteristics of RAP having high (Figure 16a) or low (Figure 16b) binder contents. Although, blends of RAP1 – VA show slightly better outcomes than blends of RAP2 – VA. This observation indicates that RAP with higher binder contents perform better than RAP with lower binder contents. Such direct comparison between RAP blends is possible because both RAP samples contain the same diabase origin aggregate pieces. When the accumulated strains from RAP-VA blends are compared against the accumulated strains from 100% VA, the results show that if the RAP-VA blends are created based on engineered mixture design approach, regardless of the RAP type, the addition of RAP results in deformations that are worse than the deformations observed from 100%VA. This observation shows that creating RAP-VA blends based on engineered gradation approach is not desirable.

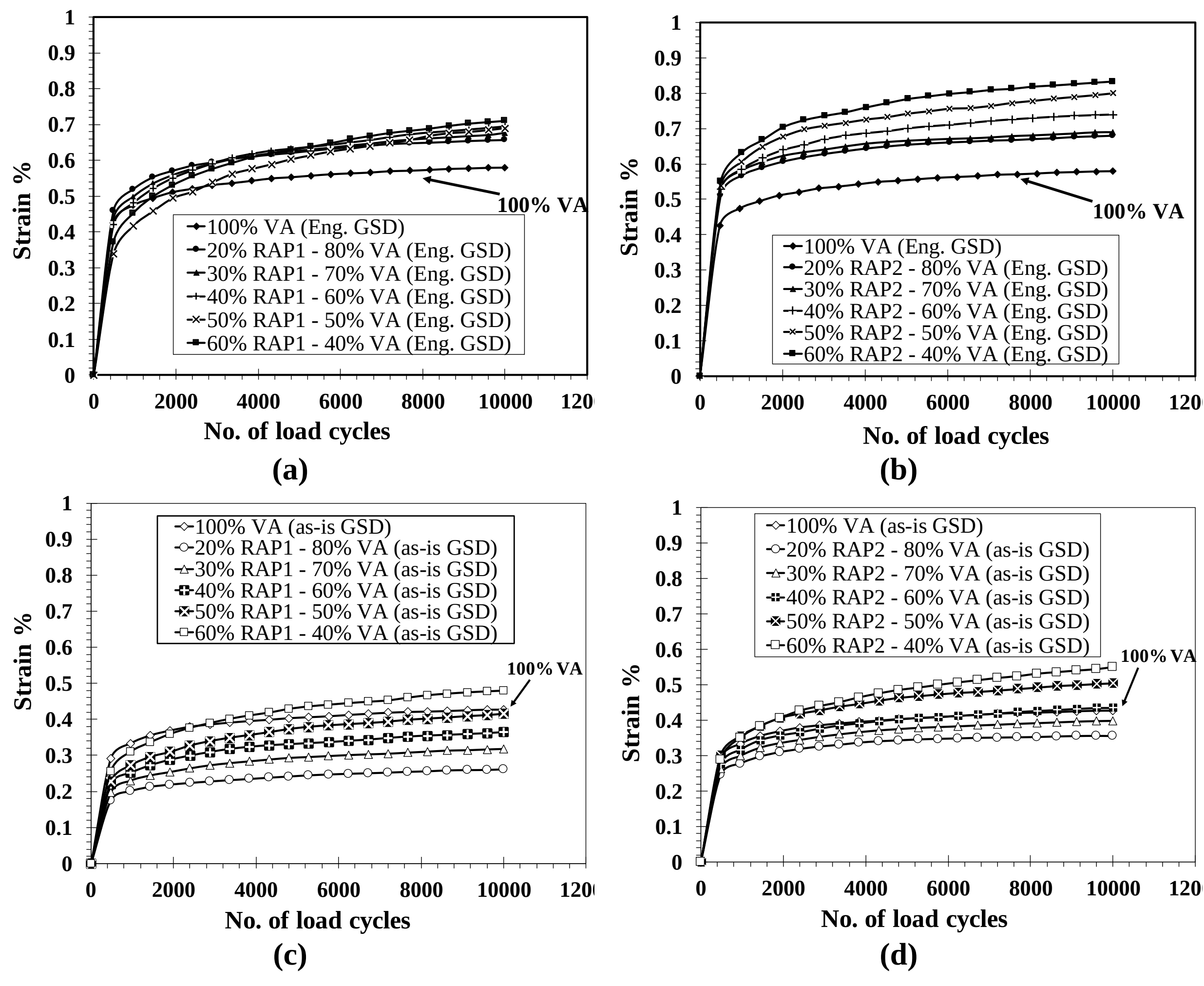
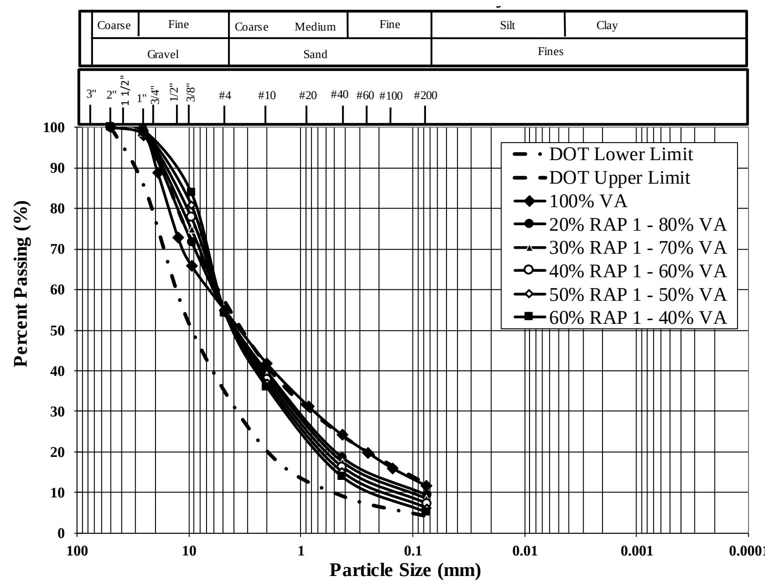


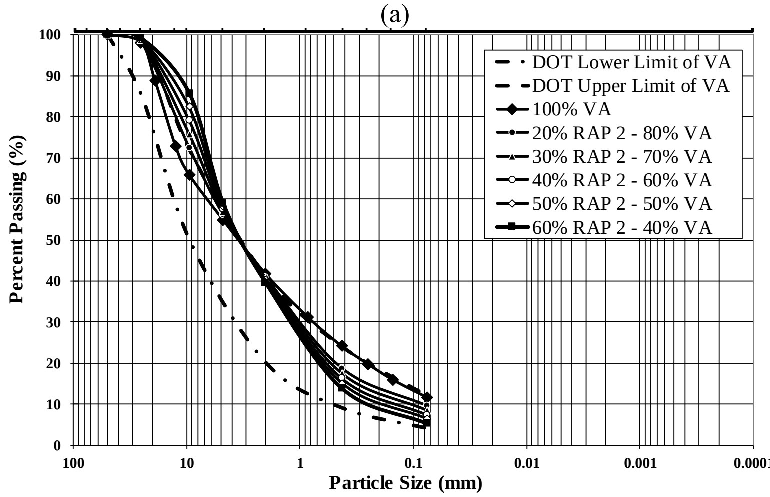
Figure 16: PD Test Results of Reclaimed Asphalt Pavements (RAP) 1 and 2 Blends with Virgin Aggregate (VA) Based on Blends Created with (a) and (b) Engineered (Eng. GSD) and (c) and (d) As-Is (as-is GSD) Mixture Design Approaches

Results from As-Is Mixture Design Gradation (Blends Based on Percent Weight)

RAP-VA blends created based on as-is gradations approach were created by mixing targeted percent of RAP with targeted percent of VA by weight. For example, to obtain 20% RAP and 80% VA, 20% of the whole mass of RAP as determined by weight was blended with 80% VA also as determined by weight. This approach resulted in grain size distributions that are slightly different for each RAP-VA blend as shown in Figure 17. When compared with VDOT's 21A aggregate grain size range, the fraction of the particles at sand size and smaller were within the VDOT's range. Particles coarser than sand size were outside of the VDOT's range. It should be noted that all RAP samples shown in Figure 17 contained aggregate pieces of diabase origin.



(a)



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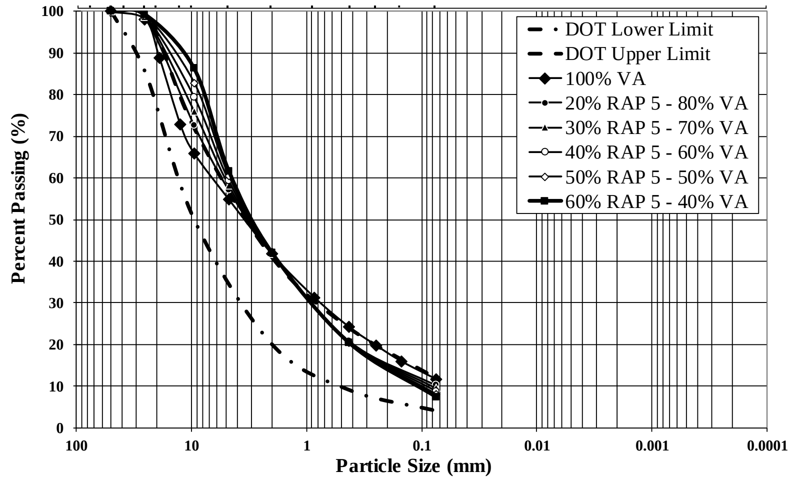


(c)

Figure 17: Grain Size Distribution of Blends with (a) RAP 1, (b) RAP 2, and (c) RAP 5 (Mixed by Weight)

Tables 6 and 7 summarizes the compaction characteristics of the RAP-VA blends evaluated in this study based on Proctor and vibratory compaction methods. RAP-VA samples were created with 100% relative compaction and at optimum moisture content. This approach is consistent with VDOT's specifications to construct unbound base aggregate (VDOT 2016).

Table 6: Compaction Characteristics of Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) Blends Used in This Study Based on Vibratory Compaction Tests (ASTM D7382)

RAP Sample Location	20%RAP 80%VA		30%RAP 70%VA		40%RAP 60%VA		50%RAP 50%VA		60% RAP 40% VA	
	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)
RAP 1	6.15	147	6.25	145	6.30	142	6.10	140	5.85	138
RAP 5	6.20	148	6.20	145	6.10	143	6.00	142	6.00	137
RAP 2	6.20	149	6.10	146	6.20	143	6.10	141	5.90	138

Note: MDD: maximum dry density and OMC: optimum moisture content, 1pcf = 16.02 kg/m^3

Table 7: Compaction Characteristics of Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) Blends Used in This Study Based on Proctor Tests (ASTM D698)

RAP Sample Location	20%RAP 80%VA		30%RAP 70%VA		40%RAP 60%VA		50%RAP 50%VA		60% RAP 40% VA	
	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)
RAP 1	7.50	145	7.80	142	7.40	141	7.80	139	7.20	134
RAP 5	7.50	145	7.60	141	7.35	141	7.60	138	7.40	132
RAP 2	7.30	144	7.44	142	8.00	141	7.80	136	7.50	132

Note: MDD: maximum dry density and OMC: optimum moisture content, 1pcf = 16.02 kg/m^3

Figures 16c and d show the accumulated strains with increased number of load cycles for RAP1 and RAP2 blends and 100% VA. The results show that up to a certain percentage of RAP, the accumulated strains are less than the accumulated strains from VA although such behavior is more pronounced for the RAP with higher binder content (RAP1) (Figure 16c) compared to the lower binder content (RAP2) (Figure 16d). This comparison could directly be made because both 100% RAP samples contained the same diabase origin aggregate particles. This observation provides an opportunity to determine a threshold where the addition of RAP improves the performance up to a certain percentage of RAP blended with VA. Therefore, unlike the results from the engineered mixture design approach (Figures 16a and b), the results from mixing RAP and VA by weight produces an opportunity to create blends that up to a certain threshold perform better than the VA under same repeated load cycles.

Summary of Comparison of Two Different Blending Approaches

The comparison of the PD test results of the RAP-VA blends created based on engineered gradation versus as-is gradation indicated that blending the material with as-is gradation approach (mixtures achieved based on percent weight) results having more favorable outcomes in terms of the performance of RAP-VA blends. Based on these observations the remaining of all evaluations were conducted in this study by blending RAP and VA based on weight (here in referred as-is gradation method).

Results Associated with Task 1 – Developing Optimized Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) Blends

Task 1 involved in developing an evaluation method to relate the performance of a typical VA base course with RAP/VA blended base material. The goal was to determine a threshold of how much RAP may be added to VA before the addition of RAP starts to show signs of adverse effects in terms of the performance of the final product compared to VA. To achieve task 1, M_R, PD, and CBR tests have been conducted with RAP-VA blends that were created based on mixing the material by weight. Results from each of these tests are presented below.

Resilient Modulus (M_R)

Table 8 presents the parameters for Equation 2 that are obtained from the M_R tests of 100%VA and blends of RAP-VA. In almost all cases, the increase in RAP content increases the k_1 parameter, which directly results in increasing the estimated resilient modulus of the material. Using the information provided in Table 8, a specific M_R value can be computed for variety of roadway profiles. In this study, to be consistent with PD results, M_R values have been computed for the scenario that corresponds to σ_1 = 30 psi (206 kPa) and σ_3 = 5 psi (35 kPa). Two different parameters were evaluated: (1) effects of RAP percentage and (2) effects of binder content. The binder content of each RAP-VA blend was determined from the known binder content of the 100% RAP on each sample and the information of specific RAP percentage (e.g., 20, 30, 40, 50, and 60%) in a given blend. Based on these information, the binder content of each RAP-VA blends could be estimated and determined from burning the asphalt content.

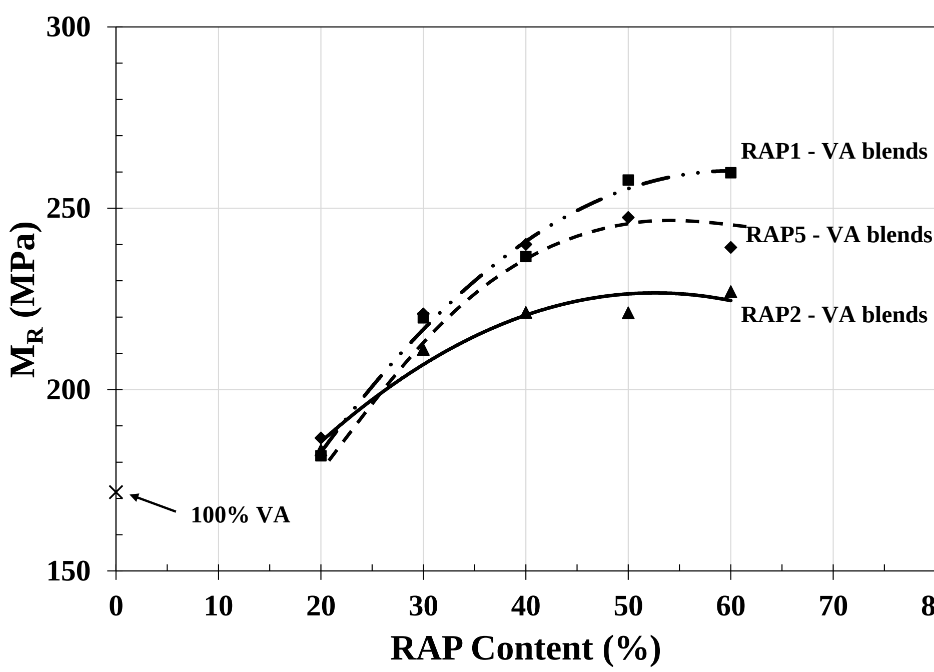
Table 8: Regression Coefficients Obtained from Resilient Modulus (M_R) Test

Sample	MEPDG model			
	k_1	k_2	k_3	R^2
	Virgin Aggregate (VA)			
100% VA	845	0.695	0.000	0.990
	RAP 1 (Binder = 5.6 – 5.8%) Parent Rock is Diabase			
80%VA – 20%RAP	880	0.755	-0.158	0.993
70%VA – 30%RAP	1183	0.676	-0.252	0.994
60%VA – 40%RAP	1320	0.587	-0.061	0.986
50%VA – 50%RAP	1483	0.601	-0.225	0.930
40%VA – 60%RAP	1481	0.616	-0.245	0.935
	RAP 2 (Binder = 4.5 – 4.7%) Parent Rock is Diabase			
80%VA – 20%RAP	933	0.662	0.000	0.989
70%VA – 30%RAP	1065	0.800	-0.467	0.983
60%VA – 40%RAP	1177	0.714	-0.351	0.975
50%VA – 50%RAP	1136	0.711	-0.213	0.977
40%VA – 60%RAP	1290	0.667	-0.419	0.943
	RAP 5 (Binder = 5.1 – 5.3%) Parent Rock is Diabase			
80%VA – 20%RAP	977	0.652	-0.070	0.983
70%VA – 30%RAP	1196	0.715	-0.416	0.981
60%VA – 40%RAP	1338	0.683	-0.407	0.991
50%VA – 50%RAP	1427	0.641	-0.378	0.978
40%VA – 60%RAP	1409	0.557	-0.152	0.948

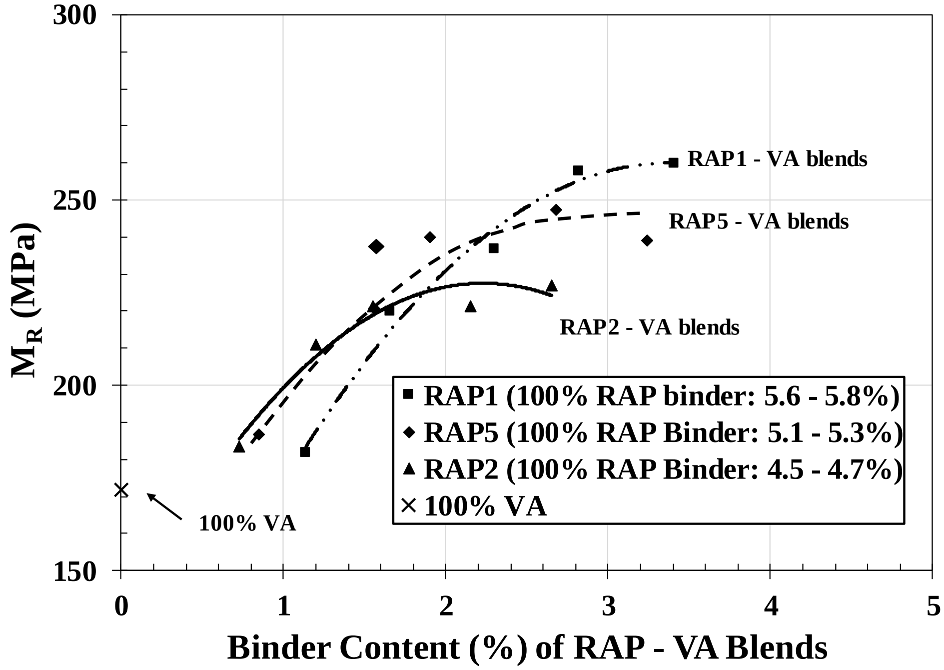
Figure 18 shows the M_R results obtained from 100%VA and RAP-VA blends created with RAP1, RAP2, and RAP5. All these RAP samples contained the aggregate pieces of the diabase rock but different binder contents. Therefore, the results shown in Figure 18a are used to evaluate the effects of RAP percentage in a given RAP-VA blend and results from Figure 18b to evaluate the effects of binder content on the modulus of the RAP-VA blends.

Figure 18a shows that for a given RAP-VA blend (such as RAP1-VA blend), the increase in RAP percentage in the blend (adding more RAP into the RAP-VA blend) increases the M_R value. When the same evaluation is made based on the binder content of the RAP-VA blend for the specific RAP (i.e., RAP 1, 2, and 5), the results show a more complex relationship as can be seen in Figure 18b. For example, in the case of 50% RAP-VA blends shown in Figure 18a, the corresponding binder contents as shown in Figure 18b are ~2.2% for RAP2, ~2.65% for RAP5, and 2.7% for RAP1. Considering that the grain size distributions of these blends are not drastically different from each other (as shown in Figure 17), the difference in M_R values is believed to be due to the difference in binder contents (~230 MPa for RAP2, 250 MPa for RAP5, and 260 MPa for RAP1) as all RAP blends contain the same 50% RAP in this example. However, Figure 18b also raises a question that the explanation based on binder content alone does not answer. For example, for the binder content of 2.8%, the M_R value of RAP1 blend is 260 MPa but the M_R value of RAP5 is 240 MPa. Also, when the binder content increases to ~ 3.2%, the M_R values either stay the same or slightly decrease. One would expect that with the increase in binder content, there should be a linear and similar change on all RAP-VA blends but the results show a more complex relationship. The reason for this complexity is not known but could be due to the differences in the age of the RAP samples. Investigating such aspect was not part of the scope of this study but most likely require further investigation.

Figure 18 also shows that when the M_R values of RAP-VA blends are compared against the M_R value of the 100%VA, in all cases, the M_R of RAP-VA blends are higher than of VA. This indicates that adding RAP increases the modulus of the VA as has previously been observed in the literature by others. However, as also previously been stated in the literature, resilient modulus alone cannot be used to evaluate the long-term performance of the RAP-VA blends as most often blends with high modulus also show high permanent deformations under repeated load cycles. To further evaluate performance and to assess long-term deformations, RAP-VA blends were also tested with permanent deformation (PD) tests. Results of these tests have been depicted in the following section.



(a)



(b)

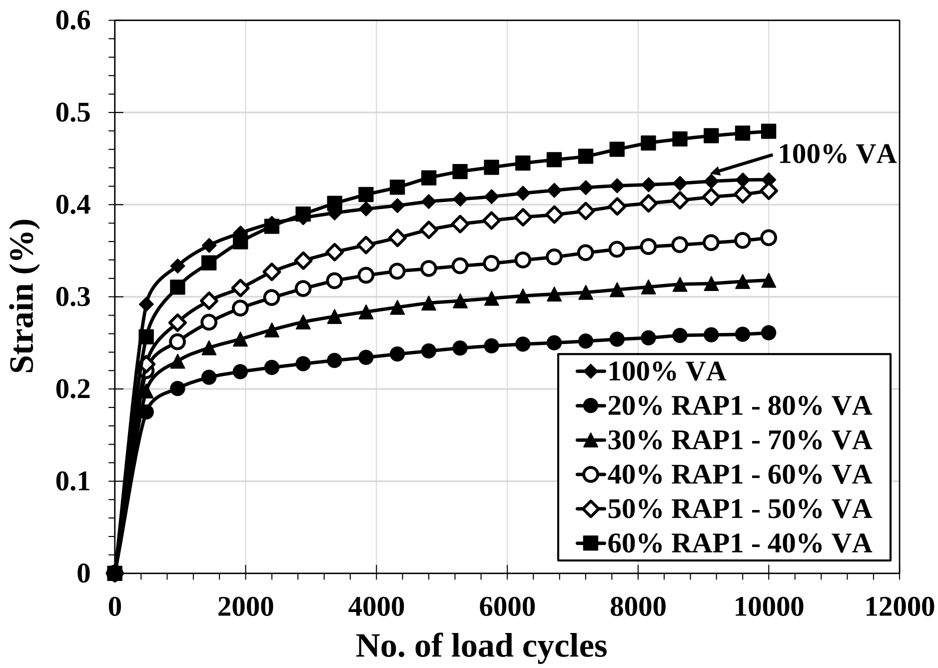
Note 1 MPa = 145 psi

Figure 18: Evaluation of Resilient Modulus (M_R) of Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) Blends of RAP with Same Parent Rock but Different Binder Content (a) RAP Percentage and (b) Binder Content within Each Blend

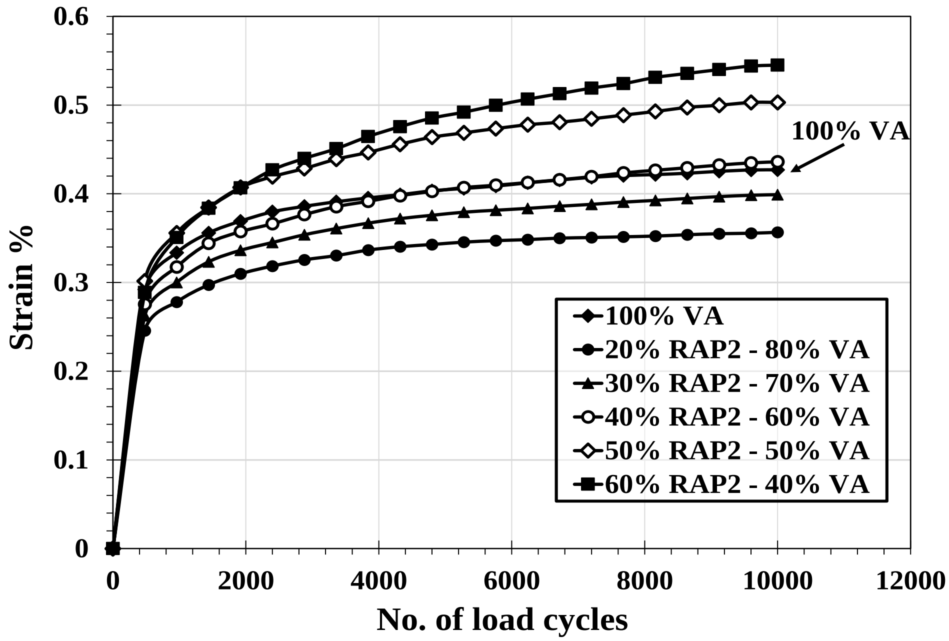
Permanent Deformation (PD)

Figure 19 shows the PD results obtained from 100%VA and RAP-VA blends created with RAP1, RAP2, and RAP5. All of these RAP samples contained the same type of diabase aggregate pieces but different binder content. Therefore, the results shown in Figure 19 are used to evaluate the effects of binder content on the percent strains of the RAP-VA blends. The trends observed in all cases of RAP-VA blends regardless of the binder content appear to be consistent with each other, that is, the addition of RAP to VA increases the total accumulated permanent strains (this is an adverse effect). This observation is consistent with previous findings by others (Bennert et al. 2000, Kim et al. 2007, Abdelrahman et al. 2010, Schaertl et al. 2010, Edil et al. 2012, Dong and Huang 2014, Ullah et al. 2018). However, Figure 19 also shows that at lower percentages, addition of RAP improves the accumulated strains when compared to 100% VA as the accumulated strains are less than of 100%VA.

Figure 20 represents the total accumulated strains from the data shown in Figure 19 that correspond to the condition at the end of 10,000 load cycles for each of the RAP-VA blend prepared with RAP1, RAP2 and RAP5. The results provide two important general information: (1) the higher the RAP percentage within the RAP-VA blend for a given RAP binder content, the higher the percent strains and (2) the higher the RAP binder content, for a given RAP-VA blend, the less the accumulated percent strains. The reasons why tests conducted with RAP that has higher binder content results in less accumulated strains for a given RAP-VA blend percentage is not known. Similar type of improvement was also observed after the M_R tests (See Figure 18). Considering that the grain size distributions of the samples were not significantly different from each other and all RAP samples contained diabase aggregates, one potential reason could be due to the age of the RAP. Understanding this phenomenon requires further evaluation. Although Figure 20 is useful to depict the percentage of RAP-VA blends that may result in similar total strains as 100%VA, this figure cannot be used to optimize the RAP-VA blends. This is because, the data shown in Figure 20 was generated based on the data from Figure 19 and Figure 19 shows that the permanent strain – loading cycle curves of RAP-VA do not reach an asymptote at the end of 10,000 cycles. In terms of stability, this type of behavior is known to result in creep shakedown, a type of response where material continues to accumulate permanent strain with increased load cycles. In ideal conditions for material considered as unbound base course, the material is expected to exhibit resilient behavior after completion of post compaction period. This would result in what is known as plastic shakedown behavior, a type of response where the material becomes entirely resilient and no or very little permanent strain occurs after a finite number of load application (Werkmeister et al. 2001). To estimate the conditions that is more aligned with plastic shakedown behavior, as suggested by Bennert et al. (2000), the data shown in Figure 19 are extended based on logarithmic model. This approach allowed the predictions of the total accumulated strains at the end of 100,000 loading cycles (extrapolated by one log cycle) as shown in Figure 21. When Figures 20 and 21 are compared, the results show that as the loading cycles increased from 10,000 to 100,000 cycles, although the 100% VA sample showed only an increase in 13% of total strain, the RAP-VA blends had significant increase in total permanent strains.



(a)



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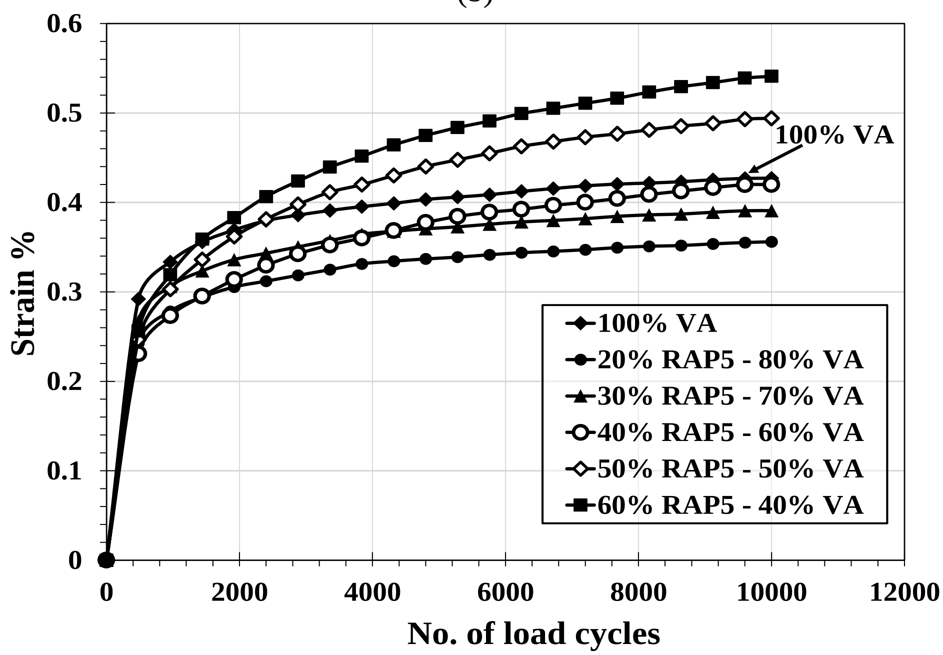
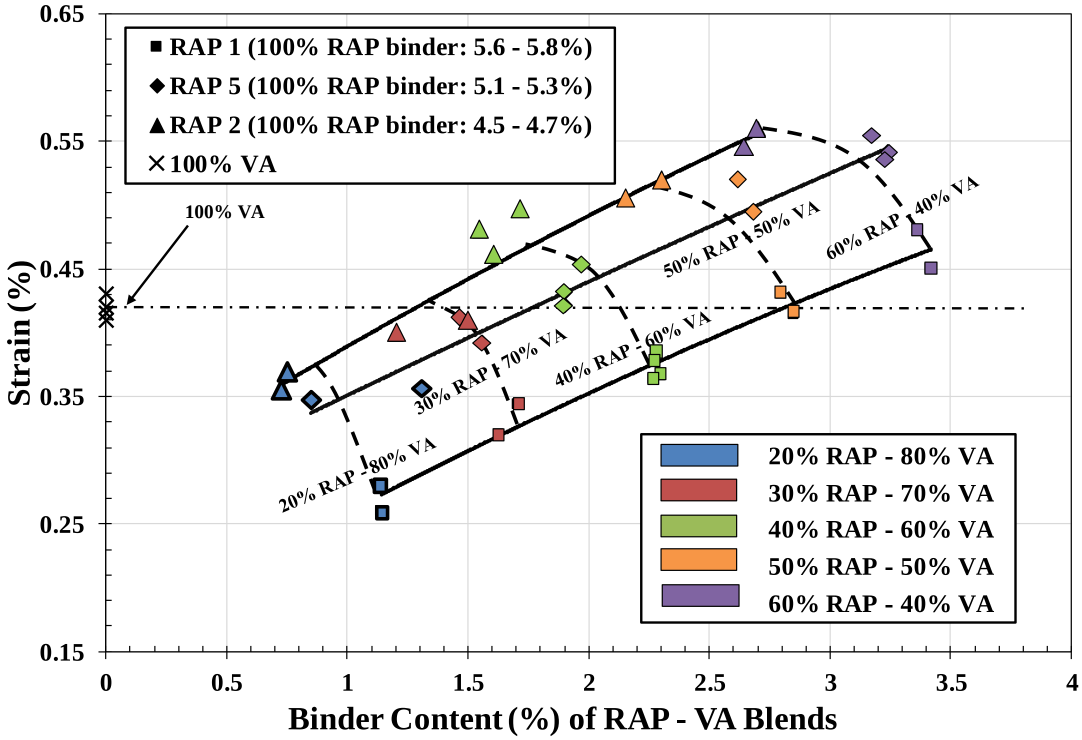


(c)

Note: All samples contained diabase aggregate pieces

Figure 19: Evaluation of the Effect of Binder Content of RAP on Permanent Deformation of Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) Blends with (a) High (5.6 – 5.8%), (b) Low (4.5 – 4.7%), and (c) Medium (5.0 – 5.2%) Binder RAP.



Note: The dashed lines shown as are drawn to indicate theoretical boundaries between the RAP-VA blends that resulted in the presented data.

Figure 20: Relationship Between Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) Blend (Binder Content) and Percent Strain at the End of 10,000 Load Cycles in a Permanent Deformation Test

Figure 21 reflects the conditions where the behavior of RAP-VA blends is closer to plastic shakedown limits (meaning that the permanent strain – loading cycle curves of RAP-VA reach to an asymptote at the end of 100,000 cycles) rather than creep shakedown limits (meaning that RAP-VA do not reach an asymptote at the end of 10,000 cycles and continue to deform with additional load). Figure 21 reflects targeted conditions that are closer to the ideal when selecting a coarse aggregate for structures that undergo cyclic loading. Therefore, when it comes to performance evaluation, Figure 21 is believed to be the appropriate characterization rather than Figure 20. Based on this characterization, the following observations are noted from Figure 21:

- The total accumulated permanent strain of the 100% virgin aggregate (VA) tested in this study was around 0.45%.
- The shaded area below the 0.45% strain line indicates that results that lie within this zone would indicate a better performing material than VA alone.
- Blends of 20%RAP2 and 80%VA and blends of 30%RAP1 and 70%VA show similar total accumulated strains (~ 0.44 to 0.45% strain) as observed from 100%VA (0.45% strain). 100% RAP2 had lower binder content (i.e., avg. 4.6%) and 100% RAP1 had higher binder content (i.e., avg. 5.7%).
- Blends of RAP2 greater than 20% and blends of RAP1 greater than 30% show total accumulated strains greater than 100%VA (more than 0.45% strain).

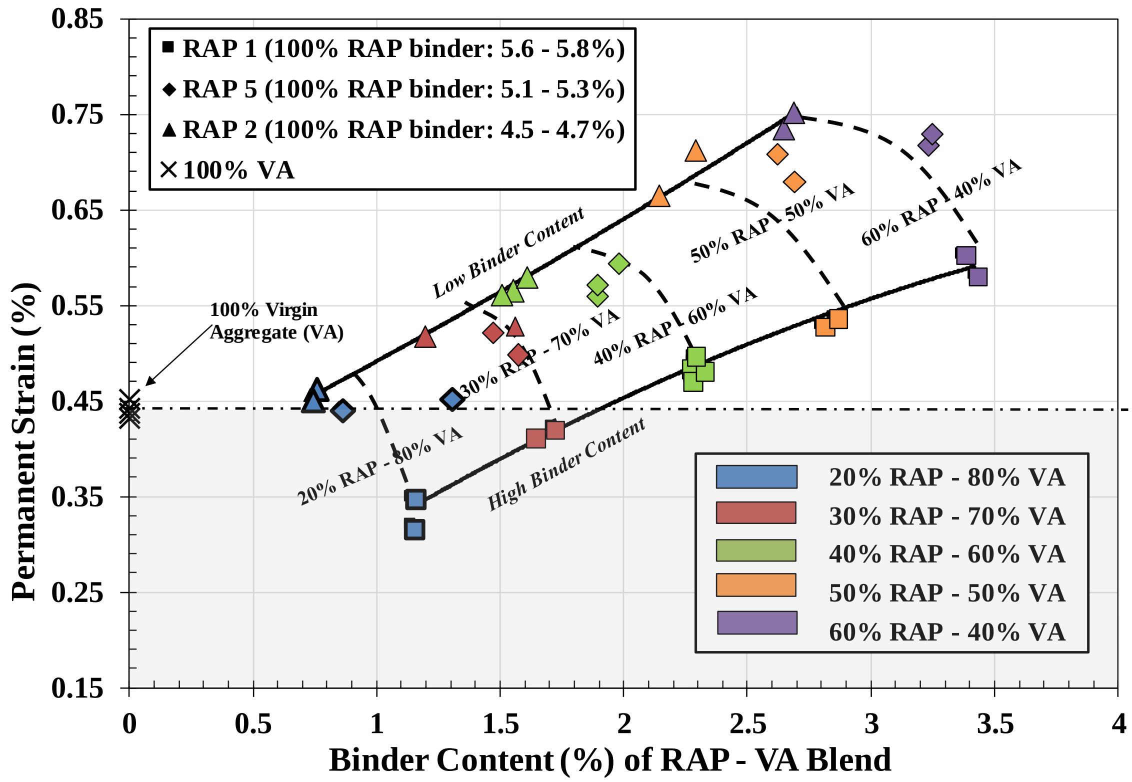


Figure 21: Relationship Between Binder Content of Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) and Percent Strain at the End of 100,000 Load Cycles (Predicted Using Logarithmic Models)

Based on the noted observations, a threshold can be identified to limit the percentage of added RAP into VA as a function of the binder content characteristics of the 100%RAP that is used in the blending.

California Bearing Ratio (CBR)

Figure 22 shows the CBR results obtained from 100%VA and RAP-VA blends created with RAP1, RAP2 and RAP5. The results shown in Figure 22 are used to evaluate the effects of binder content of RAP on the CBR value of the RAP-VA blends. Each bar presented in this figure represents the average results obtained from each of the triplicate tests conducted for each RAP-VA blend. The triplicate results showed repeatability with a coefficient of variation of less than 5%. Unlike the M_R and PD tests, CBR test results show that it is more difficult to differentiate the difference between the performance of different RAP samples based on their binder content as the CBR values of all RAP-VA blends are very similar to each other. However, overall there is a trend that shows an addition of up to 30% RAP (regardless of the binder content) could be considered acceptable as the CBR values of these blends results in similar or slightly better CBR values than the 100%VA.

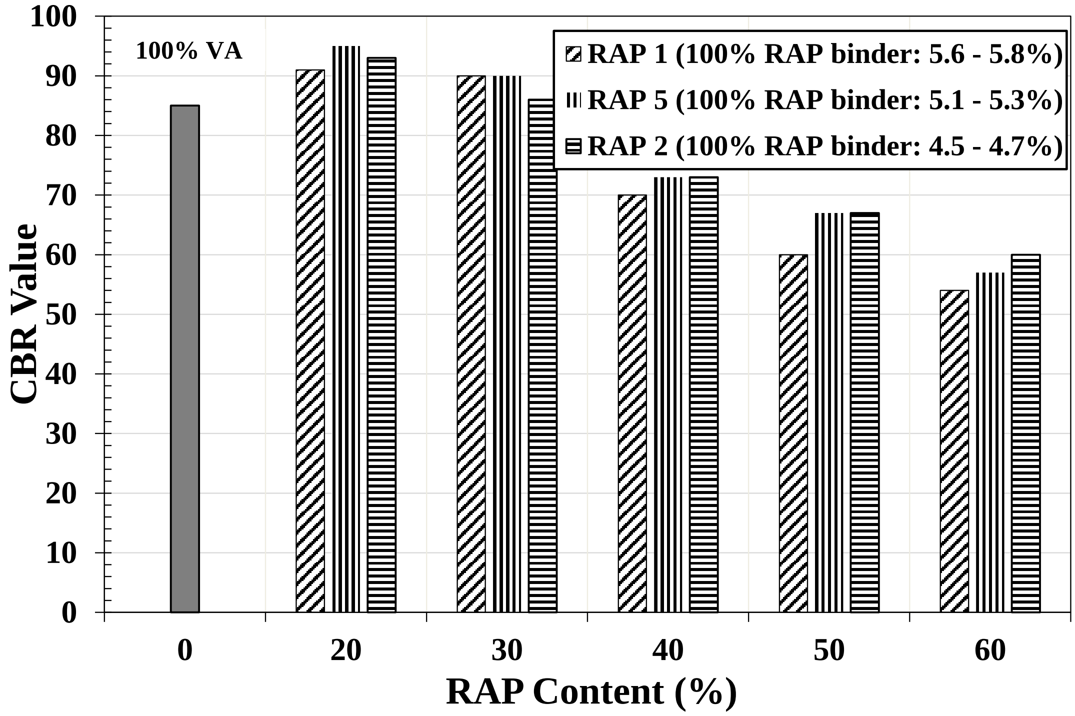


Figure 22: Evaluation of California Bearing Ratio (CBR) of Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) Blends Created with RAP1, 2 and 5 (Parent Rock Diabase)

Summary of Resilient Modulus (M_R), Permanent Deformation (PD), and California Bearing Ratio (CBR) Results

In this study, the performance evaluations have been conducted based on M_R, PD, and CBR tests. The results showed that some test methods are more effective than others to understand the differences in performance based on the differences in binder content. The overall observations noted from the tests conducted in this study are summarized below:

The M_R test results showed that both the percentage of RAP in a given blend and the binder content of the 100% RAP used in the blends effect the modulus of the RAP-VA blends. However, in all cases, the RAP-VA blends result in higher modulus than the modulus of the VA alone.

The PD test results showed that addition of RAP to VA increases the permanent strains (adverse effect) yet there is a threshold RAP content below which the permanent deformation of RAP-VA blends is less or comparable to 100% VA. This threshold RAP content varies depending on the amount of binder present in the 100%RAP mixed with VA and depending on the total number of loading cycles applied during the PD test. In this study, this threshold was estimated under 100,000 load cycles and found to be 30% for RAP with high binder content (i.e., between 5.6 and 5.8%) and 20% for RAP with low binder content (i.e., between 4.6 and 4.8%) (Figure 21).

The CBR test results showed a trend in terms of the effect of RAP percentage within a VA blend. However, this test method is not very effective to evaluate the effects of the binder content within RAP. Overall, the results indicate that the addition of RAP to VA enhances the CBR to a certain threshold (i.e., 30% RAP) but beyond that decreases the CBR value of the RAP-VA blend.

Based on the results summarized herein, the remainder of the study focused on conducting the evaluations with RAP-VA blends created with up to 30% RAP.

Results Associated with Task 2 –Suitability of Quality Control Tools

Task 2 involved in developing an evaluation to relate the use of LWD and speedy moisture content tests to perform quality control measures in the field for the RAP-VA blends. To achieve task 2, LSME, bench scale laboratory evaluation, and FSME tests have been conducted in addition to a separate evaluation of the validity of speedy moisture content test. Results from each of these tests are presented below.

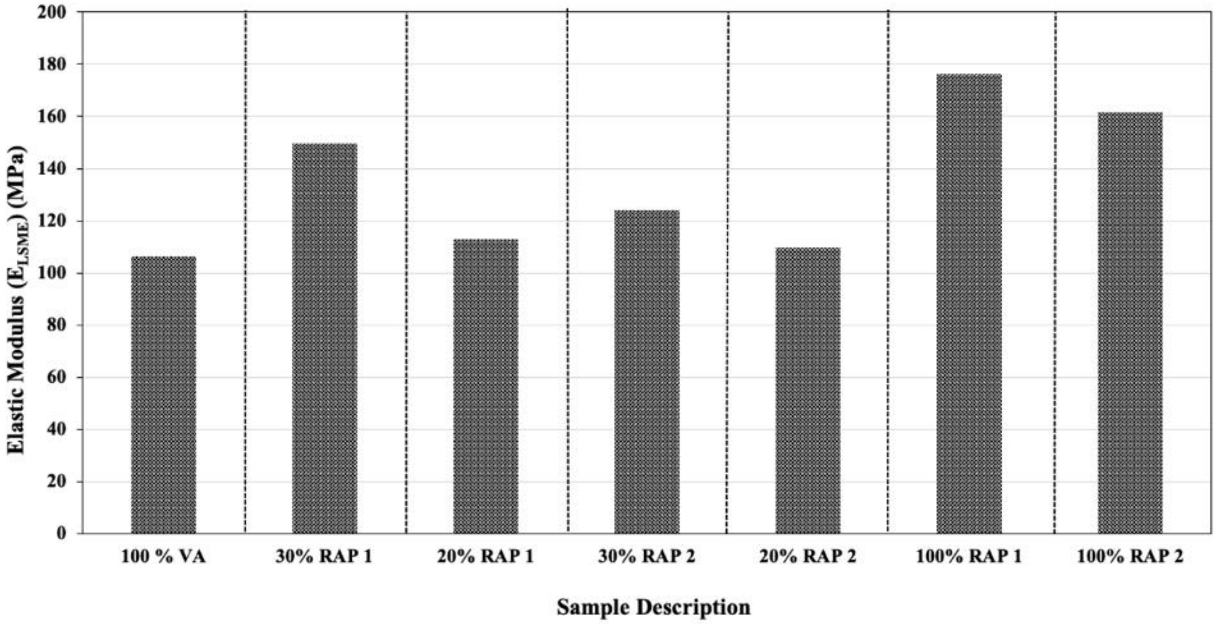
Light Weight Deflectometer (LWD) Test Results from Large Scale Model Evaluation (LSME)

The results of LWD tests performed on the test pit that was used as part of the LSME are shown in Figure 23. These results represent the values obtained from the top of the base course layer that was constructed in the LSME set-up (i.e., top of the 18-in (0.45 m) layer). The modulus obtained from these LWD test results represent the modulus of the materials without the interference of any boundary conditions. Although the modulus values from the LWD cannot be directly compared against the modulus values from the laboratory M_R tests, the trends observed from each of the RAP samples (herein RAP1 represents the RAP with high binder content and RAP2 with low binder content) (Figure 23) are in agreement with what has been observed in the laboratory tests (Figure 18). This observation validates the results from LWD equipment in terms of being able to capture the RAP-VA behavior appropriately. The COV values for the LWD tests ranged between 0.76 and 1.56. Keeping in mind that COV less than 15% is considered acceptable, the COV values determined from the LSME tests indicate good repeatability.

Light Weight Deflectometer (LWD) Test Results from Bench Scale Laboratory Evaluation

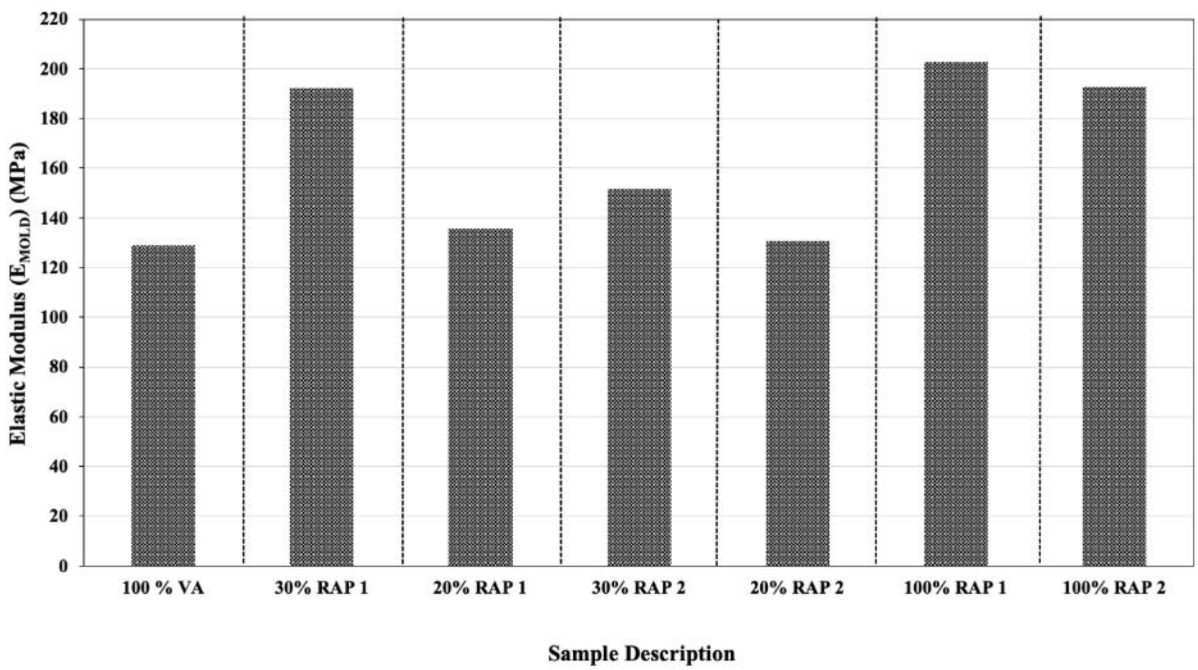
LWD tests conducted in LSME reflect the modulus of the materials without the interference of any multi-layer effects. Therefore, these values could be used as target values in the field to check the degree of compaction of the RAP-VA blends constructed. However, it is not expected that VDOT will construct a LSME set-up to establish target values prior to field construction. Therefore, LWD tests were also conducted with the same material using a standard 6-in (0.15 m) compaction mold. Figure 24 shows the results of the modulus values obtained from these tests. These modulus values were not directly obtained from the LWD equipment. Instead the load and deflection measurements from the instrument were used in Equation 5 to compute the modulus values (herein referred as E_{Mold}). The COV from the tests ranged between 0.75 and

6.82 (within the range of being less than 15%). These values validate the repeatability of the results.



Note: 1 MPa = 145 psi

Figure 23: Light Weight Deflectometer (LWD) Test Results Performed in Large Scale Model Evaluation (LSME) Test Pit

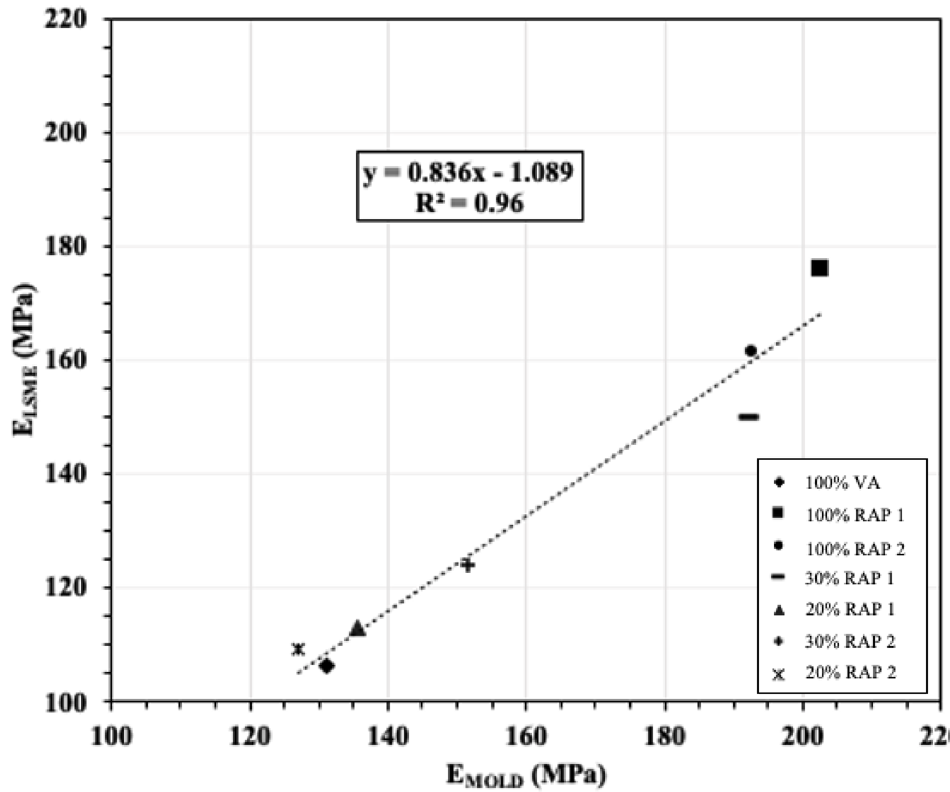


Note: 1 MPa = 145 psi

Figure 24: Light Weight Deflectometer (LWD) Test Results Performed on Compaction Mold

The comparison of LWD test results obtained from the tests conducted in compaction mold and LSME showed difference in terms of magnitude, although the trends between the LWD test results from two different testing procedures were almost the same. The moduli values obtained from LWD test results from the compaction mold were greater in magnitude than the moduli obtained from LSME LWD tests for respective samples (comparison of Figures 23 and 24). This observation is expected because of confinement provided by side walls of the steel compaction mold. Thus, in order to make use of the data from the compaction mold, a correlation between LWD tests conducted in LSME and compaction mold was developed. Figure 25 shows such correlation. The relationship between modulus values obtained from these methods were shown as in Equation 8.

$$E_{LSME} = 0.836\ E_{Mold} - 1.089 \quad (R^2 = 0.96) \qquad \text{Equation 8}$$



Note: 1 MPa = 145 psi

Figure 25: Scale Effects Between Large Scale Model Evaluation (LSME) - Light Weight Deflectometer (LWD) and 6-in Mold - Light Weight Deflectometer (LWD) Tests

Equation 8 that is derived from Figure 25 is valid for both 100%VA, 100% RAP (including both with high and low binder contents), and for the RAP-VA blends created based on different RAP percentages and binder contents. E_{LSME} computed with this approach is intended to be used as the target modulus value in the field (E_{Target}) for the LWD tests conducted for quality control. All of these test results were obtained from the materials that were compacted to 95% relative compaction and at optimum moisture contents. Therefore, in the field, if the modulus values from LWD equipment were determined to be equal or greater than the E_{LSME} (a.k.a. E_{Target} in this case) value of that particular material, the compaction criterion is considered satisfactory. However, such conclusion requires that the material is at optimum moisture content as determined from the speedy moisture content tests.

Light Weight Deflectometer (LWD) Test Results from Field Scale Model Evaluation (FSME)

To put it into perspective on how to implement the given methodology of compaction quality control in the field, results of the FSME test with 30% RAP1 have been evaluated. In this evaluation the modulus of the base course had to be back calculated from the measured LWD deflections because the thickness of the base course was smaller than the depth of influence of the LWD. Therefore, the modulus determined from the LWD measurements were affected by both the modulus of the base course and the subgrade underneath. Once the back calculated base course modulus ($E_{ComputedBase}$) was determined, it was then compared against the previously established E_{Target} value as described above.

The results of the LWD tests on top of the base course was 65.2 MPa (herein referred as $E_{Multilayer}$). The COV in these tests were calculated as 0.63, which is well below the maximum threshold, confirming the repeatability of the LWD tests. The back calculation of the base course modulus was established with Equation 7 (i.e., resulted in $E_{ComputedBase}$=163.6 MPa). The LWD results from the mold set-up for the 30% RAP 1 is 192.2 MPa (see Figure 23). When converted to E_{LSME} (using Equation 8), this value becomes 159.6 MPa, which was used as E_{Target}. When $E_{ComputedBase}$ is compared against the E_{Target}, the difference between these two values is very small (i.e., approximately 2%). This shows that the proposed methodology based on laboratory evaluation stays valid in the field application where the bottom layer consists of subgrade. The similarity of the E_{Target} and $E_{ComputedBase}$ was the expected outcome because the field test site was constructed to match the degree of relative compaction and moisture content that was followed during the mold and LSME tests.

These findings showed that the proposed methodology could be implemented for quality control in the field. This validation was important before initiating the actual full-scale implementation in the field (Task 3).

Speedy Moisture Content Test Results

Figure 26 shows the laboratory results obtained from the speedy moisture content tests for all materials tested in this study based on the modified testing protocol. The figure also compares these values against the moisture contents determined from air-drying method. In the case of RAP, determining the moisture content with oven drying method is not reliable as the binder content of the material affects the results. Therefore, air-drying is considered as the conventional (traditional) method to determine moisture contents. The results shown in Figure 26 indicate that the modified protocol of the speedy moisture content test in this study correlates well with the results obtained from the air-drying results. This validates the proposed method in this study.

Summary of Large Scale Model Evaluation (LSME), Bench Scale Laboratory Evaluation, Field Scale Model Evaluation (FSME), and Speedy Moisture Content Results

LSME tests presented the evidence that LWD as an equipment provides repeatable results and works well with RAP-VA blends. The values obtained from the LSME may serve as the target modulus for the field applications. The bench scale laboratory evaluation results showed that LWD equipment could directly be used over a 6-in (0.15 m) compaction mold and the values obtained from these tests could be correlated with the results observed from the LSME. This relationship allows the users to determine a target modulus value in the field. FSME test results showed the validation of how to handle the scenarios in the field where due to the thickness of the base course, a multi-layer scenario may exists when LWD is placed directly over the base course.

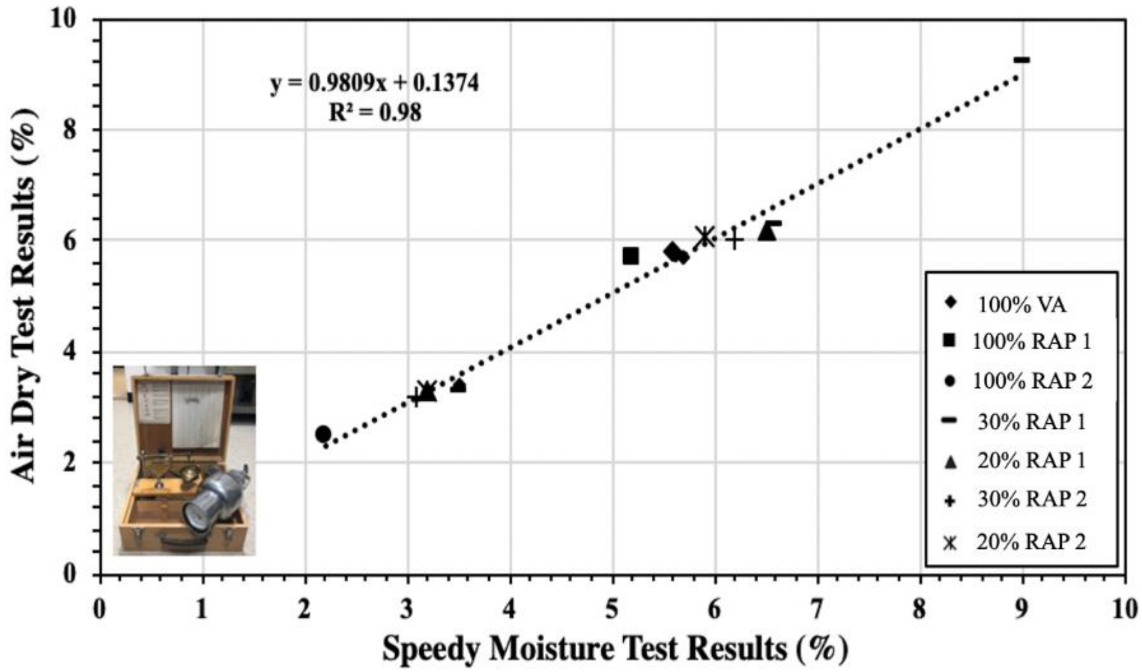


Figure 26: Results of Correlation Tests on Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) Blends with Speedy Moisture Test.

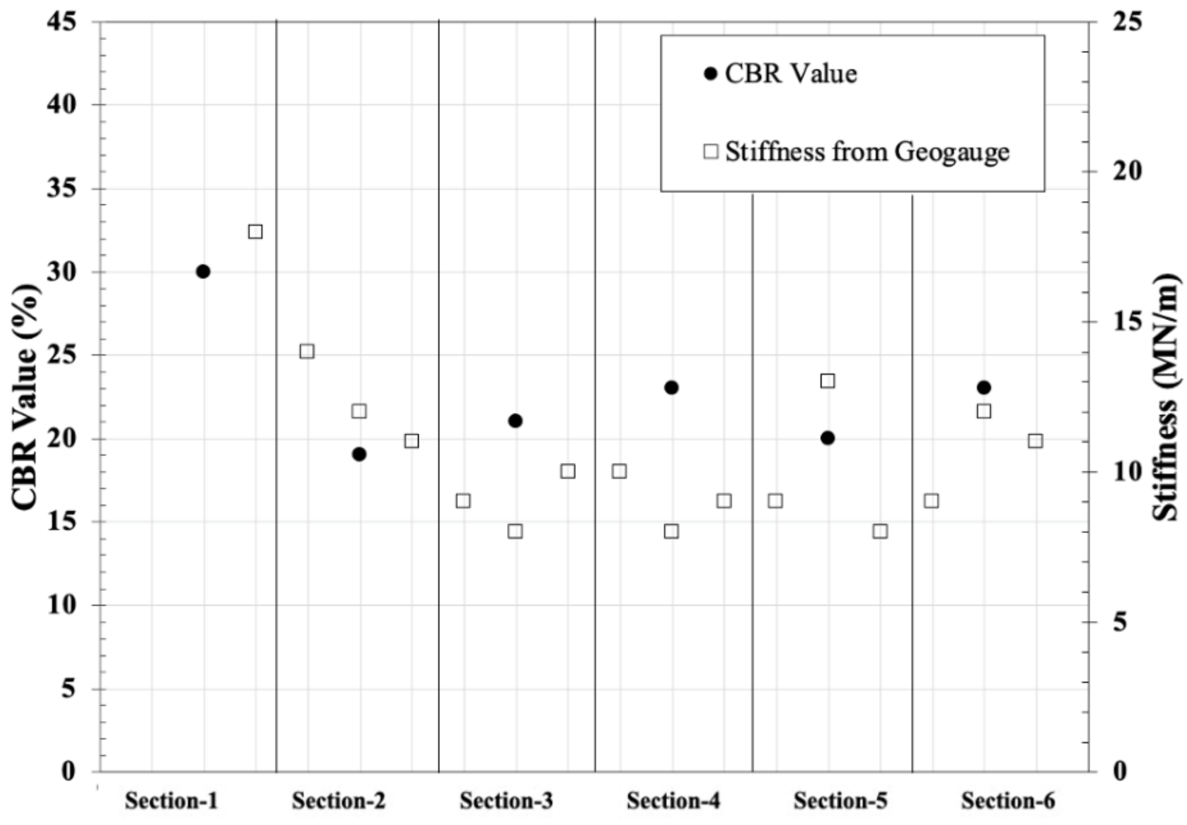
FSME results also confirmed that the target modulus values from the laboratory could be used to confirm the suitability of the compaction in the field granted the moisture content of the material is also known. The results obtained from the modified speedy moisture content testing procedure confirmed that this device can successfully be used in the field to determine the moisture content of the RAP-VA blends.

Results Associated with Task 3 – Evaluation of the In-Situ Field Performance

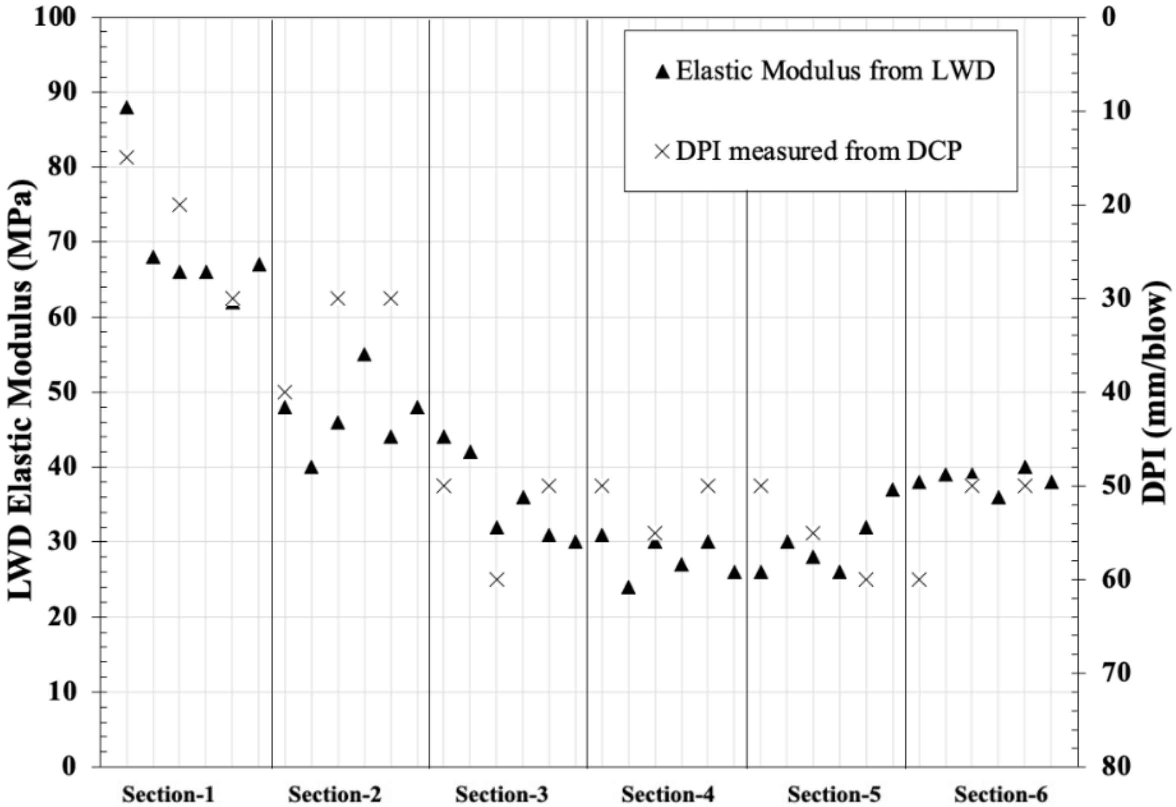
Results from the Quality Control Demonstration in the Field – Minnieville, Virginia

At the beginning of the construction, subgrade was characterized to confirm the uniformity within the site. A combination of the grain size distribution data and Atterberg Limit results were then used to classify the soil based on UCSC.
The average Atterberg Limits of the subgrade was determined as LL=46 and PL=34 and CBR of as 20. The subgrade soil found at the site was classified as low plastic sandy silt (ML) according to the USCS soil classification system. The water content of the subgrade soil at the time of construction was found to be between 20 – 25%, which is lower than the plastic limit of the soil.

Figure 27 shows the consistency of the compacted ground in terms of laboratory determined CBR in unsoaked condition and in-situ tests (LWD, DCP, and SSG survey results). The magnitude of the values determined from each instrument are expected to be different. The intention of the data shown in Figure 27 is to present the trend lines observed based on different methods. All of the results indicate a slightly stronger ground condition right underneath the section 1 and fairly uniform ground conditions underneath all of the other sections. The average CBR values were 20, which classifies the subgrade as good. These results confirm that the test sections were constructed above a competent ground that is fairly uniform.



(a)



(b)

Note: DPI = DCP Penetration Index 1 MPa to 145 psi

Figure 27: In-situ Characterization of Compacted Subgrade

Once the subgrade was characterized, this information was used as part of the quality control measures for the base course layer. Therefore, following the placement of the base course, additional LWD tests were conducted at the same locations where the previous LWD tests were conducted on the subgrade. These test results are shown in Table 9. The measured thickness of the base course at each of these locations were known (Table 9). Considering the thickness of the base course layers and the depth of influence of the LWD equipment (i.e., 0.3 m), the average modulus obtained from the top of the base course indicates the modulus of the multilayer system consisting of both the base course and the subgrade, hence depicted in Table 9 as $E_{Multilayer}$. In all tests, the calculated COV values were within the acceptable threshold (Table 9) indicating the acceptability of the LWD test results.

In order to determine the modulus of the base course (without the influence of the subgrade) ($E_{ComputedBase}$), the modulus values obtained from the field were used in Equation 7 along with the thicknesses of the base course determined in the field. The computed values are shown in Table 9.

Table 9: Modulus Values Obtained from Field Implementation and Computed Modulus for the Base Course

Field Sections	$E_{Subgrade}$ (MPa)	COV (%)	$E_{Multilayer}$ (MPa)	COV (%)	Thickness of base course (m)	$E_{ComputedBase}$ (MPa)
30% RAP 1	40.0	4.47	83.8	12.61	0.16	173.1
	40.0	4.47	97.5	6.89	0.18	171.5
	54.2	5.01	101.7	5.55	0.15	167.9
	43.5	5.58	95.7	4.66	0.17	168.3
20% RAP 1	27.8	1.47	61.5	3.04	0.18	113.7
	29.9	1.37	64.7	3.62	0.18	118.1
	29.8	1.37	61.8	2.79	0.17	116.8
	24.2	3.11	55.7	3.70	0.17	118.9
30% RAP 2	27.3	2.99	48.5	5.01	0.14	139.1
	29.7	8.70	53.5	2.83	0.18	131.0
	23.3	3.50	52.8	3.86	0.18	141.3
	23.5	3.56	50.3	4.65	0.15	128.5
20% RAP 2	25.3	7.76	55.5	2.48	0.17	114.4
	28.0	2.26	61.5	3.53	0.17	112.9
	36.3	1.42	74.5	4.86	0.17	118.1
	36.9	1.52	73.0	2.45	0.17	114.7
100% VA	68.8	3.10	76.8	5.99	0.19	113.8
	86.8	7.65	76.3	3.94	0.18	108.6
	67.7	3.11	75.6	4.88	0.19	111.9
	62.0	2.11	72.3	3.23	0.19	108.8

Notes: $E_{Subgrade}$: Average Subgrade Layer Elastic Modulus, $E_{Multilayer}$: Average Multilayer Elastic Modulus, $E_{ComputedBase}$: Average Base Course Layer Elastic Modulus

When the overall methodology developed in this study is put together, in order to determine the acceptability of the constructed base course layers, the computed base course modulus needs to be compared with the target modulus (which in this study has been determined previously in the laboratory). This comparison is presented in Table 10. When compared, the results show that at each location in the field within each section, the LWD modulus values for the base course are greater than the target elastic modulus (E_{Target}). This indicates that the sections were constructed to a compaction level that is considered satisfactory. On the contrary, when the actual nuclear density gauge results obtained from each section were compared against the target relative compaction that was set forth in this project (i.e., 95% or greater), the results showed that only the section that was constructed with VA satisfies this criterion (Table 10). This observation indicates that the predicted relative densities computed from the NDG tests for sections constructed with RAP blends were not accurate. Figure 28 presents the summary of this comparison.

Table 11 shows the results from the field determined moisture contents at each section based on modified speedy moisture content tests and air drying of the same field samples in the laboratory. The results show very similar values from both methods. Also, it is important to note that when these moisture content values were compared against the target moisture content values that were established for the project, the results obtained from the field confirms the intent of the design.

Table 10: Comparison of the Computed Base Course Modulus with Target Values and Relative Compaction from Nuclear Density Gauge

Field Sections	$E_{ComputedBase}$ (MPa)	E_{Target} (MPa)	Ratio of $E_{ComputedBase}$ and E_{Target}	Average Ratio of $E_{ComputedBase}$ and E_{Target}	Compaction from NDG (%)
30% RAP 1	173.1	159.6	1.08	1.07	89.3
	171.5		1.07		
	167.9		1.05		
	168.3		1.05		
20% RAP 1	113.7	112.3	1.01	1.04	90.4
	118.1		1.05		
	116.8		1.04		
	118.9		1.06		
30% RAP 2	139.1	125.7	1.11	1.07	91.8
	131.0		1.04		
	141.3		1.12		
	128.5		1.02		
20% RAP 2	114.4	105.2	1.09	1.09	93.4
	112.9		1.07		
	118.1		1.12		
	114.7		1.09		
100% VA	113.8	108.5	1.05	1.02	96.0
	108.6		1.00		
	111.9		1.03		
	108.8		1.00		

Notes: $E_{ComputedBase}$: Average base course layer elastic modulus, E_{Target}: Target elastic modulus from Equation 4.

Results Obtained from Field Instrumentation Embedded into Base Course

Specific locations of the instruments installed in each section were previously shown in Figure 8. Data was collected from these instruments starting right after construction in September 2018 until the end of March 2020. The subsequent sections provide the results associated with each instrument/feature.

Changes in Temperature and Moisture Contents

Figures 29 summarizes the changes in temperature observed from the base course layer at the test site at each different section. These measurements were obtained from the embedded thermistors within the earth pressure cells. The results showed that there was not any noticeable difference in ground temperatures from one section to another and between the top and bottom of the base course.

The previous researchers Dong and Huang (2014) and Soleimanbeigi and Edil (2015) pointed out the effects of temperature on the performance of 100% RAP. The results of their study indicated that if the temperature within the ground exceeds 25 or 22 degrees Celsius (77 and 71.6 degrees Fahrenheit) respectively, the use of 100% RAP as an unbound base aggregate was not recommended. The results in Figure 29 show that both on top and bottom of the base course, the temperatures observed in the field have exceeded these thresholds. These observed temperature ranges also confirm that in Virginia the visco-elastic properties of RAP may affect the overall performance of the base course. Dong and Huang (2014) and Soleimanbeigi and Edil

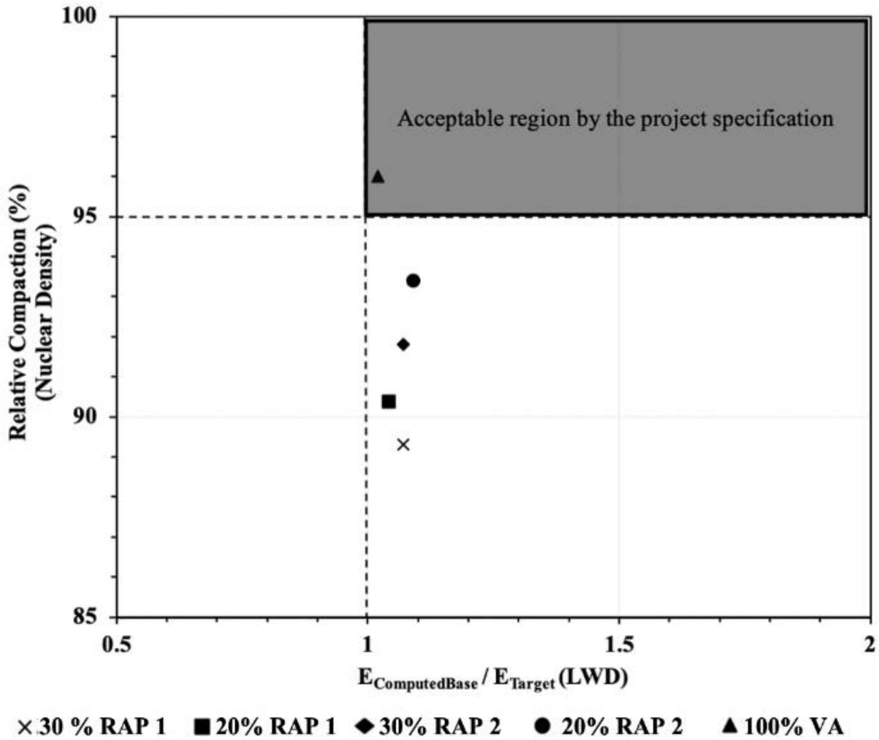


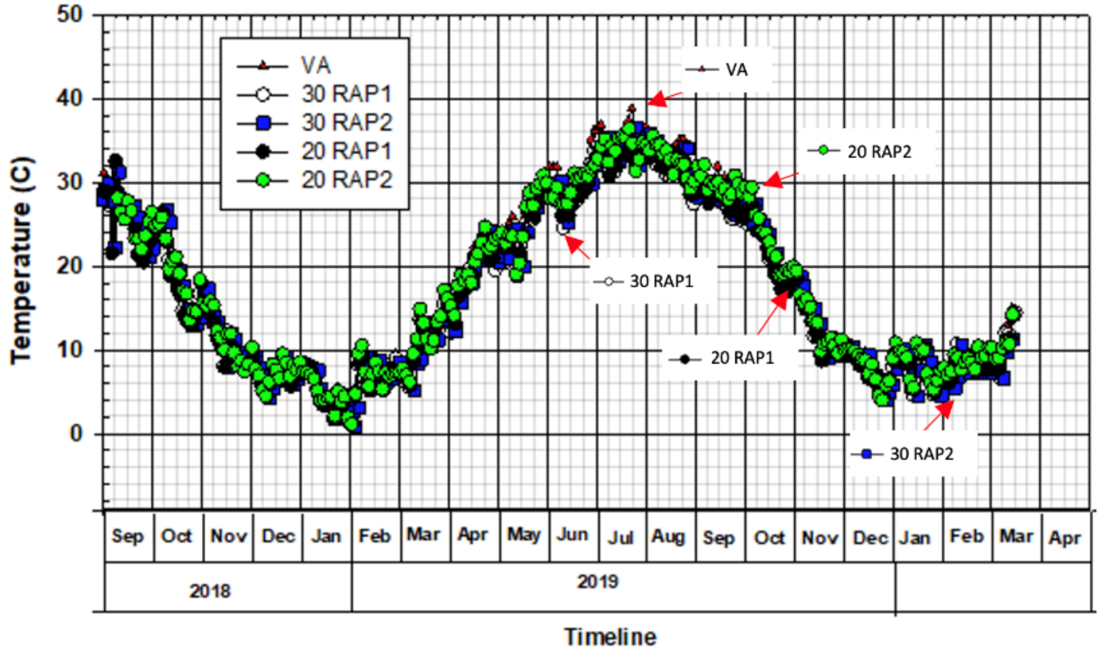
Figure 28: Comparison of Average Ratio of Computed Base Course and Target Modulus Values from Light Weight Deflectometer (LWD) Test Versus Relative Compaction Values Obtained from Nuclear Density Gauge

Table 11: Comparison of Measured Moisture Content Values of Base Course Aggregates from the Field

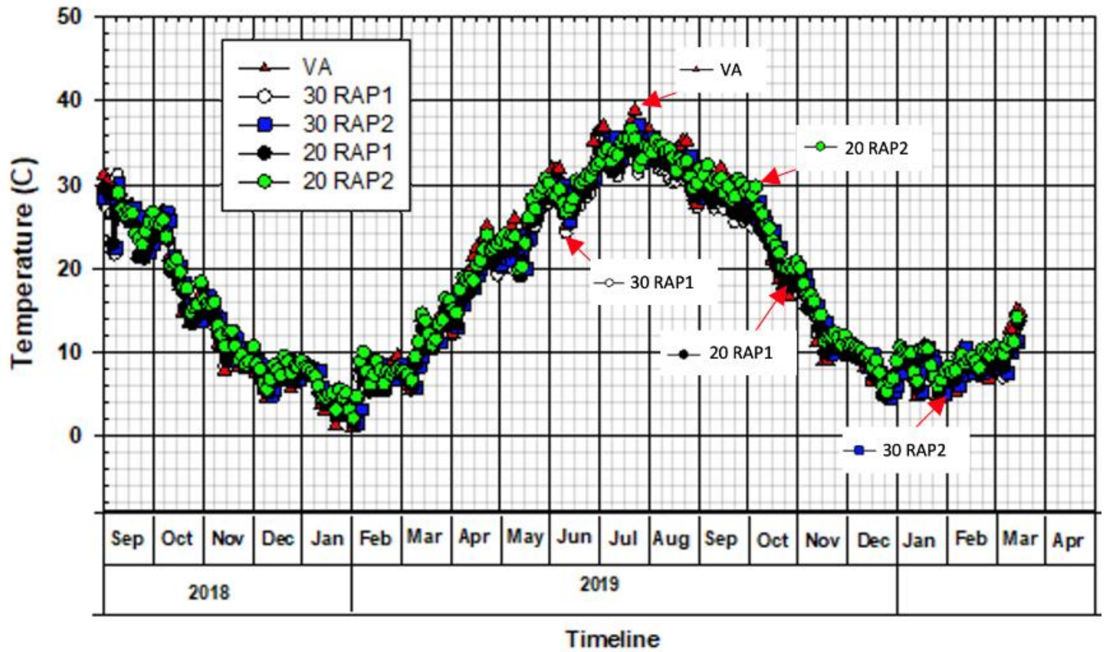
Field Sections	**Air Dry Tests (%)**	**Speedy Moisture Tests (%)**
30% RAP 1	3.61	3.47
20% RAP 1	3.49	3.56
30% RAP 2	2.80	2.64
20% RAP 2	4.65	4.75
100% VA	3.60	3.20

(2015) only tested 100% RAP in their study, therefore a specific threshold for RAP percentages cannot be determined from their observations. However, at the site constructed in this study, the maximum RAP percentage was limited to 30%, therefore the overall observations as presented in the subsequent figures confirm the suitability of this threshold.

Figure 30 presents the changes in precipitation near the site and recorded moisture contents within the base course of each section. These measurements were obtained from the



(a)



(b)

Note: 1 degree Celsius =33.8 degrees Fahrenheit

Figure 29: Changes in Field Temperature within Base Course (a) at the Top and (b) at the Bottom (Right above the Subgrade)

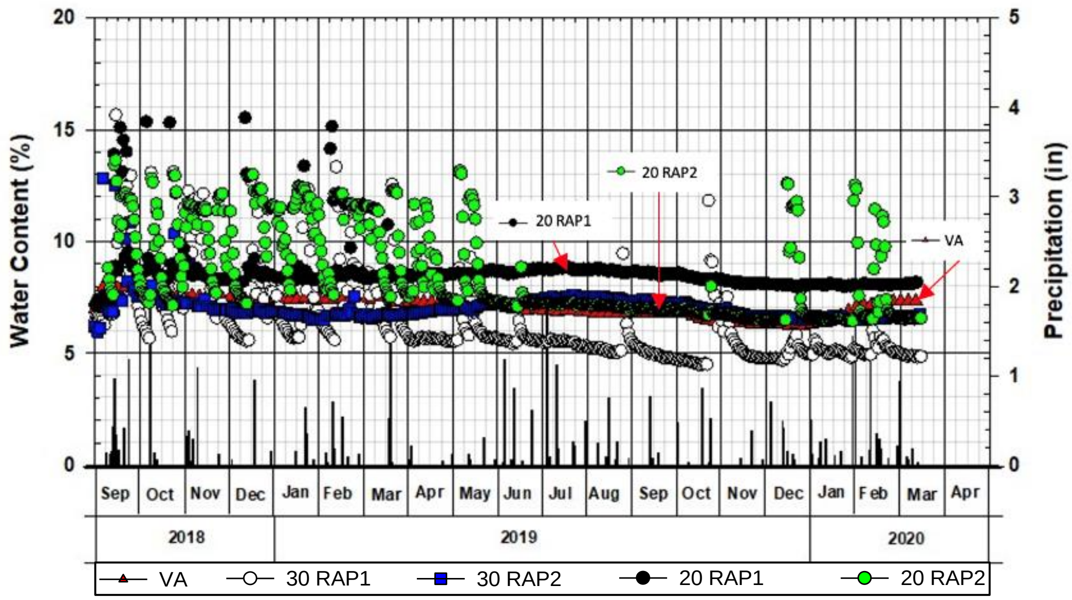
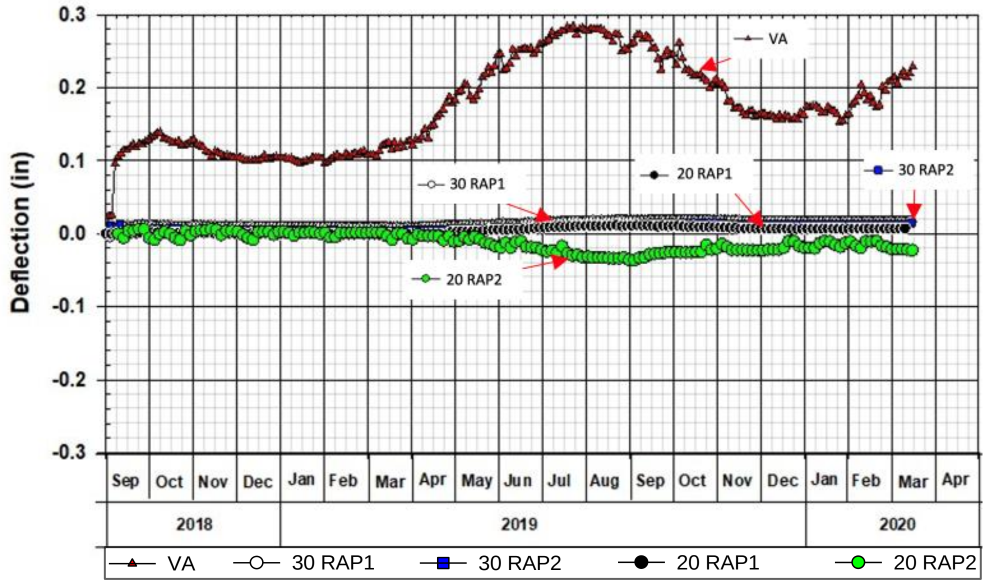


Figure 30: Changes in Field Precipitation and Moisture Contents within the Bottom of Base Course

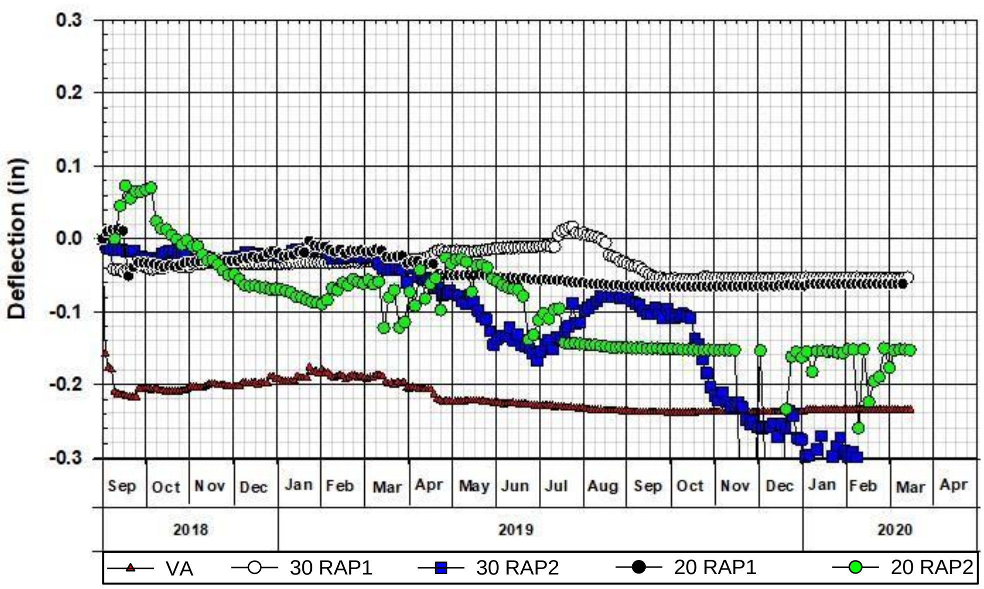
water content reflectometers embedded at the bottom of the base course. The precipitation data was obtained from National Weather Service Forecast Office operated by National Oceanic and Atmosphere Administration (NOAA) located in Manassas (Lat: 38.72°N, Lon: 77.52°W), 14 miles (22.5 km) north-west of test pavement section constructed in Woodbridge, VA. The results show that overall, in all sections the moisture content of the material on average stayed between 5 and 10 percent. In most cases the instruments showed the changes in moisture content were due to precipitation. This is interpreted as a confirmation that the RAP sections stayed unbound because the water was able to infiltrate and change the results. However, such sensitivity was not observed from all instruments as in the case of 100% VA. In that section, it is believed that the installed instrument was not within the path of the infiltrated water from precipitation. Water content reflectometers installed in unbound aggregate media are known to be less sensitive and accurate with their measurements. However, overall data (with some fluctuations) indicate that once the material was compacted, the water contents in the field stayed close to their optimum moisture contents determined from the laboratory (i.e., + 2%, - 1%).

Changes in Deflections Observed within Base Course

Figure 31 shows the deflections measured from horizontal and vertical soil strain gauges installed within the base course. Data from Figure 31a show the results of lateral extension that are measured from the horizontal soil strain gauges. In all sections constructed with RAP-VA blends, the horizontal movements almost do not exist. However, for the 100%VA section, lateral deflections increase over time since the instrument was installed during construction. The fluctuations noted within the data indicate the seasonal effects when the data is evaluated between Sept. 2018 and March 2019 and between April 2019 and March 2020. However, the trend of increasing deflections indicates that at a nearby location some compression is occurring within the layer. Based on the data on Figure 31a, sections constructed with RAP-VA blends perform much better than the section constructed with 100% VA.



(a)



(b)

Figure 31: Changes in (a) Lateral Extension and (b) Vertical Compression within Base Course

Data from Figure 31b show the results of vertical compression that are measured from the vertical soil strain gauges. Sections constructed with RAP1(RAP with high binder content) indicate very little movement compare to the other sections. More importantly, the trend of the data indicate that the compression has taken place but appear to be stable (not increasing over time). This indicates that sections with RAP1 perform better than all other sections. This observation supports the threshold of RAP percentage and binder content implemented based on the laboratory study. When the same data is evaluated for RAP2 sections, the section with 30% RAP shows a trend of increasing deflections that are higher than any other sections. Also, the trend shows an increase in deflections with time that has not yet stabilized. In a long run, if not stabilized, section constructed with 30% RAP2 may start to show issues related to pavement performance. As can be seen from Figure 21, this behavior was also noted from the laboratory evaluations, however this section was constructed because the intention was to find the maximum threshold where RAP could be blended with VA. The field behavior confirms the laboratory observations. This indicates that for RAP with low binder contents (i.e., 4.5 to 4.7%), it is best to keep the percentage of RAP threshold within 20%. Section with 20% RAP2 performs slightly worse than RAP1 sections but slightly better than 100% VA.

Changes in Earth Pressures Observed at the Bottom of Base Course

Figure 32 shows the data from the earth pressure cells installed at the bottom of the base course (right above the top of the subgrade). The data shows that in all sections except for 20% RAP1, the range of pressure fluctuation follows the expected seasonal trends and are within the similar values. In summer the pressures increase and in winter they decrease. This could be due to the precipitated water in summer and near freezing conditions in winter. Given the fact that the density of all of these materials were close to each other, the observed magnitudes are acceptable although these values should not be directly compared against what is expected from theoretical vertical stress computations. This is because in field applications, most often arching of the soil occurs over the instrument, which affects the magnitude of these measurements. Also, the saturation conditions in the field may fluctuate over time, which results in changes in the soil pressure over the instrument. Therefore, trends convey a story more than the exact magnitudes.

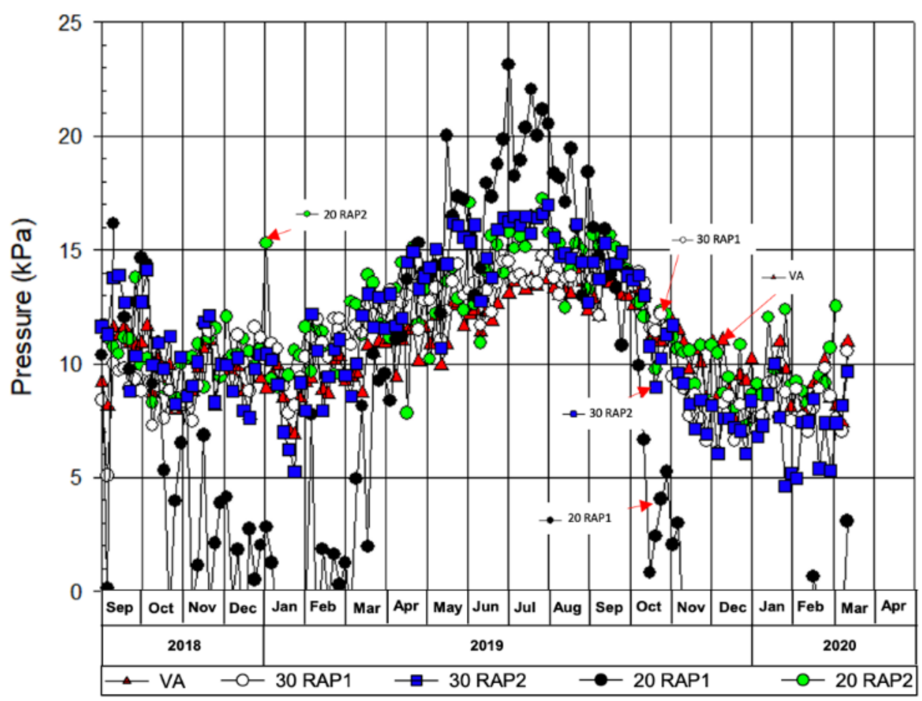


Figure 32: Changes in Vertical Earth Pressures Estimated at the Bottom of Base Course

Based on these observations, it is stated that for the purposes of stress distribution, all layers behaved similarly even though the observations from 20% RAP1 show a different trend. This indicates that sections with RAP stayed as unbound aggregates and not bonded as could happen under high temperatures with asphalt material. The earth pressure cells installed in the field are known to be prone to point loading effect due to the individual grains pushing on the overall plate where the measurements are obtained from. As a result, the common practice is to lay out a thin layer of sand below and over the top of the plate as can be seen from Figure 9c. The results obtained from 20% RAP1 section indicate that even with this practice, some amount of aggregate pieces have created this so-called point load effect as the results show a significantly different trend than others. However, this data in itself also support the concept that the aggregates stayed as unbound.

Changes in Earth Pressures and Tensile Strains Observed at the Bottom of Asphalt Layer

The focus of this study was to evaluate the performance of the base course layers, however pressure distributions underneath and tensile strains within the asphalt layer were also evaluated to confirm the uniformity of the constructed HMA.

Figure 33 shows the data from the earth pressure cells located right underneath the asphalt layer (at the top of the base course layer). The data shows that on average the pressure at the bottom of the asphalt course fluctuate depending on the seasonal changes and resonates around 0.73 psi (5 kPa) on average in sections with 100% VA, 20% RAP1, and 30% RAP2. The data from the asphalt layer in 30% RAP1 and 20% RAP2 sections indicate some problems with the instrument readings. The overall variations in pressure changes at the bottom of asphalt course can be explained based on anisotropic thermal expansion and contraction of the asphalt course. Zeng et al. (1999) explains that due to vertical expansion and contraction of asphalt course as a result of seasonal changes, miniscule gaps are created between base and asphalt courses resulting in arching effect during winter season. Al-Qadi et al. (2004) also reported an increase in the vertical stresses with increasing weather temperature. As supported by the previous studies, it is believed that this may be the cause of reduction in stresses during winter season as opposed to summer when surface temperatures start to rise. However, the consistency of the results from the instruments that are in good working order shows the consistency of the asphalt behavior in all sections.

The data obtained from asphalt strain gauges installed at the bottom of the asphalt course are presented in Figure 34. Asphalt layers in sections of 100% VA and 20% RAP2 had strain gauges that were installed at a location along the paved sections that were away from the traffic wheel path. However, the instrument installed at the 20% RAP2 section did not survive during construction. Therefore, the only data that indicates the movement of pavement due to creep without the effect of the traffic is shown in Figure 34a. The results indicate that there was a small increase initially however, the accumulated strains have stabilized and stayed steady for the remainder of the time period. The total accumulated tensile strain from the HMA layer was approximately 2,000 micro strains (Figure 34a). When compared with literature the total accumulated magnitude of strain seems reasonably (Al-Qadi et al. 2004). The data in Figure 34a shows that changes in season did not affect these results. This particular instrument was installed at a location very close to where the curb and gutters were installed (not in the middle of the base course). Therefore, the impact of traffic to both the base course and the asphalt layers were minimal if any.

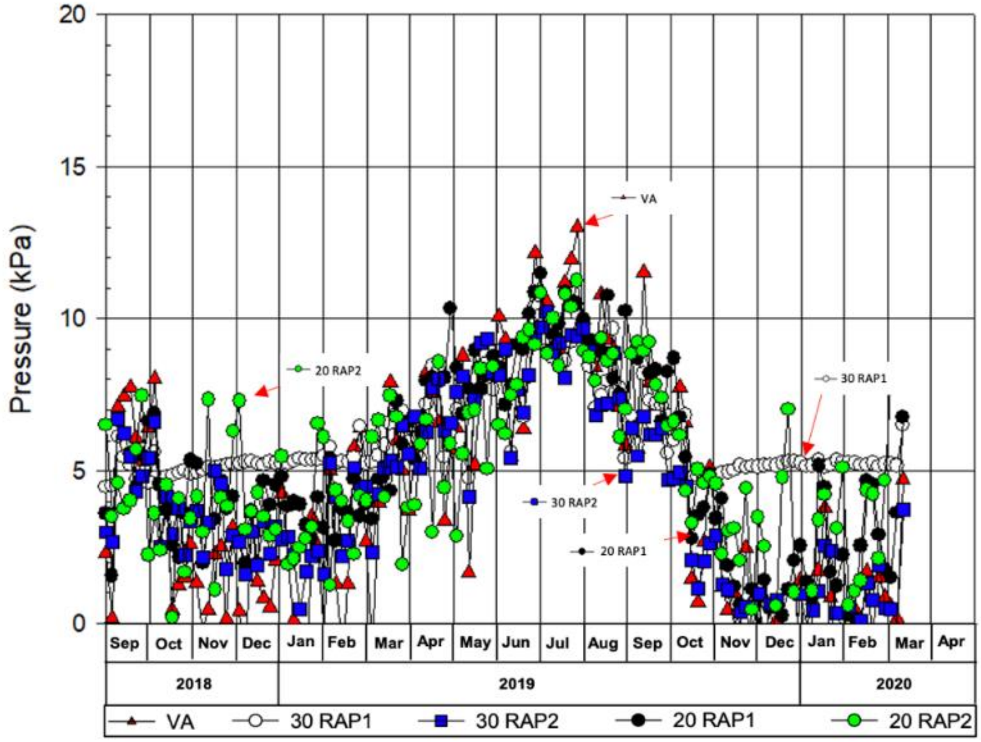
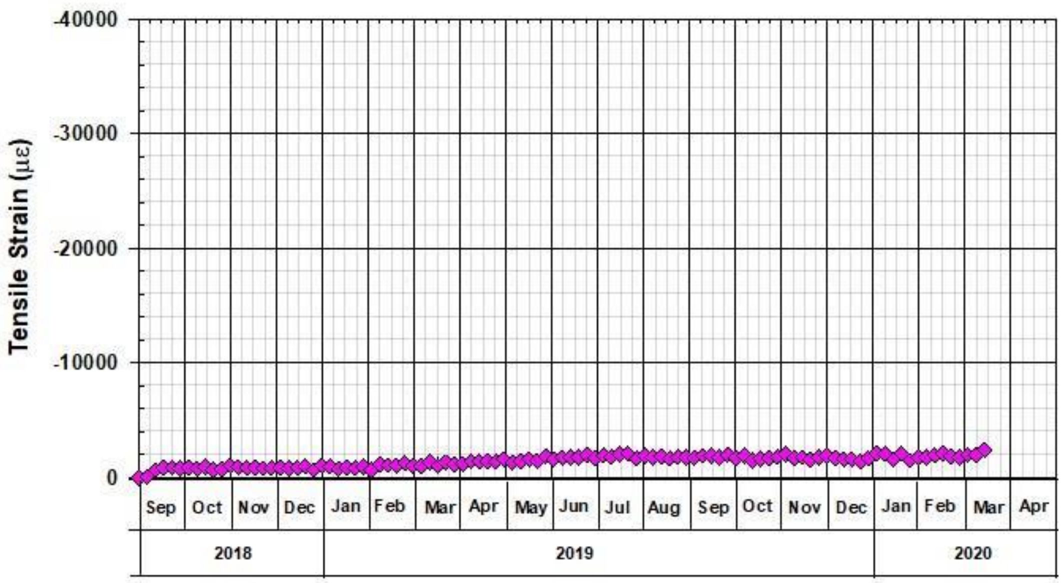
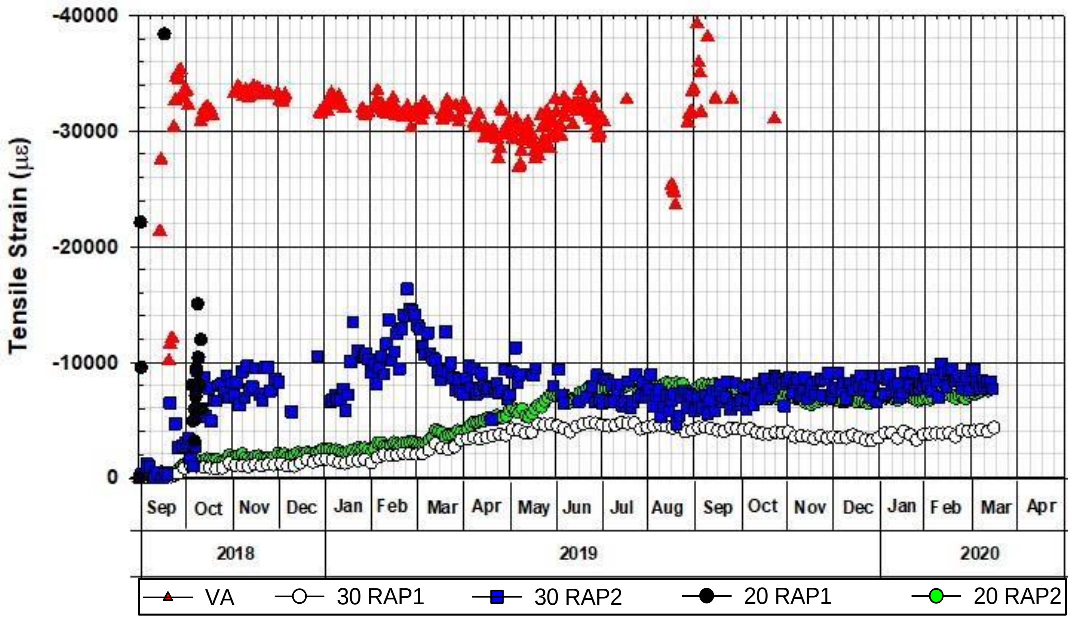


Figure 33: Changes in Vertical Earth Pressures Estimated at the Bottom of Asphalt Layer



(a)



(b)

Figure 34: Changes in Asphalt Strain Gauges At the Bottom of Asphalt Layer Cumulative Strains (a) Due to Pavement Creep Only and (b) Due to Pavement Creep and Traffic

Asphalt layers in all sections were also instrumented with tensile strain gauges installed along the wheel path of the traffic. The data obtained from these instruments are shown in Figure 34b. The instrument installed in section with 20% RAP1 stopped working by mid-October 2018 right after construction. However, data from 30% RAP1, 20% RAP2, and 30 % RAP2 all shows very similar trends where the tensile strains initially increase and then stabilizes over time. The magnitude of the strains measured due to traffic (Figure 34b) are order of magnitude higher than the strains observed from the creep of the pavement itself without the effects of traffic (Figure 34a). This is an expected outcome as the traffic results in higher strains on the surface of the asphalt. When the results of the magnitude of the tensile strains from all sections compared, it can be seen that the results from 100% VA is significantly higher than others followed by data from the asphalt layer of 30% RAP2 section, 20% RAP2 section, and 30% RAP1 section respectively. These differences can be explained when the behavior is compared with what has been shown in Figure 31b from the strain gauges installed within the base course. As the base course compresses due to traffic movement, the asphalt layer has to resist the flexural moments due to self-weight and traffic loads. The induced flexural moments cause the asphalt layer to accumulate tensile strains. The asphalt layer in 100% VA shows a drastic increase in tensile strain right from the beginning of the time period right after construction (Figure 34b). This behavior is also consistent with the explanation above as the base course about the same time also drastically show vertical compression (Figure 31b). The strain gauge in asphalt layer of the 100% VA stopped working after July of 2019.

All together seven strain gauges were installed within the asphalt layer, however, three of them ultimately stopped working. The loss of these instruments is not a surprise as other researchers have also noticed the extreme sensitivity of these instruments causing them to fail prematurely (Tim et al. 2004).

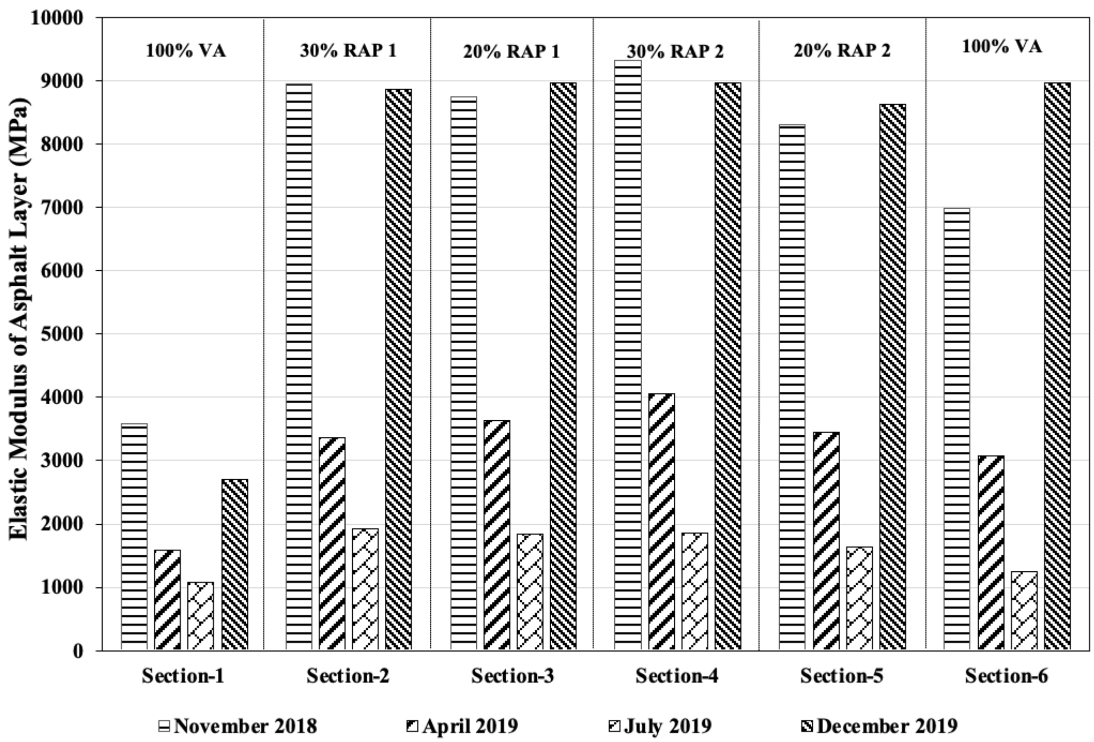
Results From Testing Conducted After Construction

Figure 35 shows the photo of the site after the construction. Testing using LWD, FWD, and IRI equipment were conducted along the entire length of the constructed test site at the beginning of November 2018 (Fall), April 2019 (Spring), July 2019 (Summer), and end of December 2019 (Winter). All tests were conducted only along the right wheel path of the far-right lane that was constructed with RAP-VA and VA base course layers.

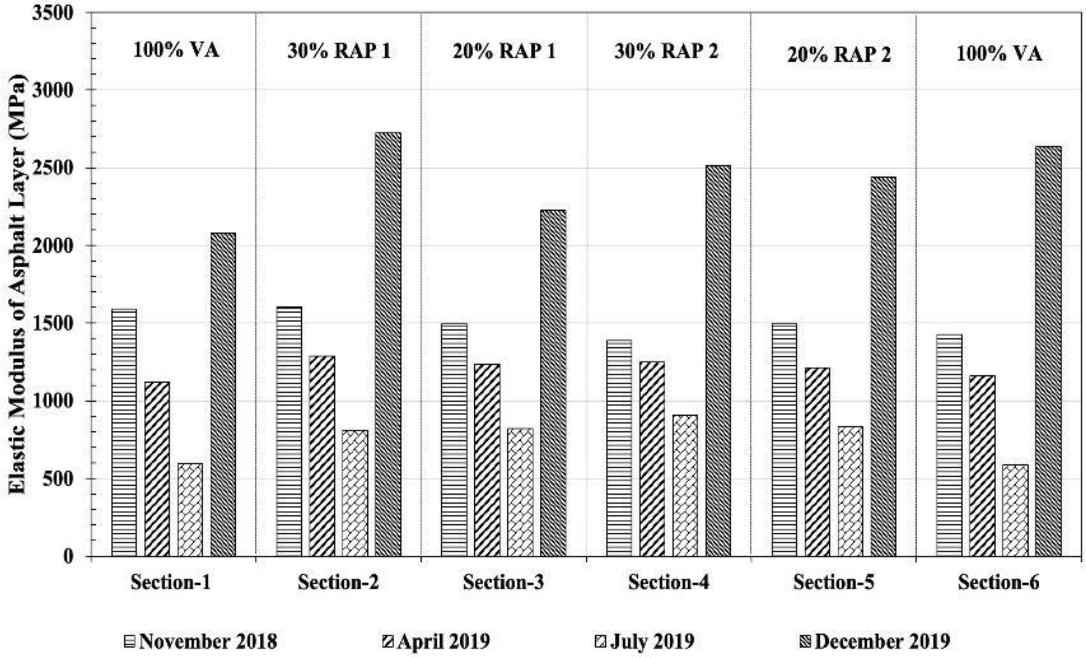
Figure 35: Overview of the Constructed Test Site

Characterization of the Asphalt Layer

Figure 36 shows the results from FWD, LWD, and IRI to characterize the asphalt layer and rideability of the surface course. The intention of evaluating all of this data was to confirm the uniformity of the asphalt layer throughout the site and also to evaluate the changes due to seasonal effects. The duration of the surveys was limited to only one-year time frame, therefore discussing about the longevity of the pavement structure would be premature at this stage. Both the FWD and LWD equipment were successful in capturing the seasonal effects on the pavement layer in terms of modulus values. In colder periods, the modulus values were higher than the warmer times (as expected). With the exception of data from 100% VA section (section 1), for a given time within the season, the modulus of the asphalt layer in each section was consistent with each other (Figure 36a). The discrepancy observed in section 1 was not noted in the data from LWD (Figure 36b), where data from all sections were fairly uniform with each other for a given time in the season. It should be pointed out that the modulus from the FWD equipment was a result of back-calculation, however the data from LWD was a direct measurement without the need to utilize the multi-layer back calculation. Although the magnitude of the modulus values cannot be directly compared with each other, the trends observed from FWD and LWD surveys (with the exception of section 1) were both in agreement with each other. This showed the consistency within the asphalt layer of the test site. The survey results from the IRI tests showed the differences in ride quality from one section to another but did not show significant difference from one season to another (Figure 36c). The data shown in Figure 36c for each location is the average data from both runs for each location. When the results seen in Figure 36c are compared against the IRI classification ratings (Table 12), the section constructed with 30% RAP1 was rated as “excellent” in terms of ride quality whereas the rating of all other sections classified as “good”. The differences in these ratings are believed to be due to the differences in the performance of the base course layers (not the differences of the subgrade or HMA layers). The subsequent section describes the reasoning for this observation.



(a)



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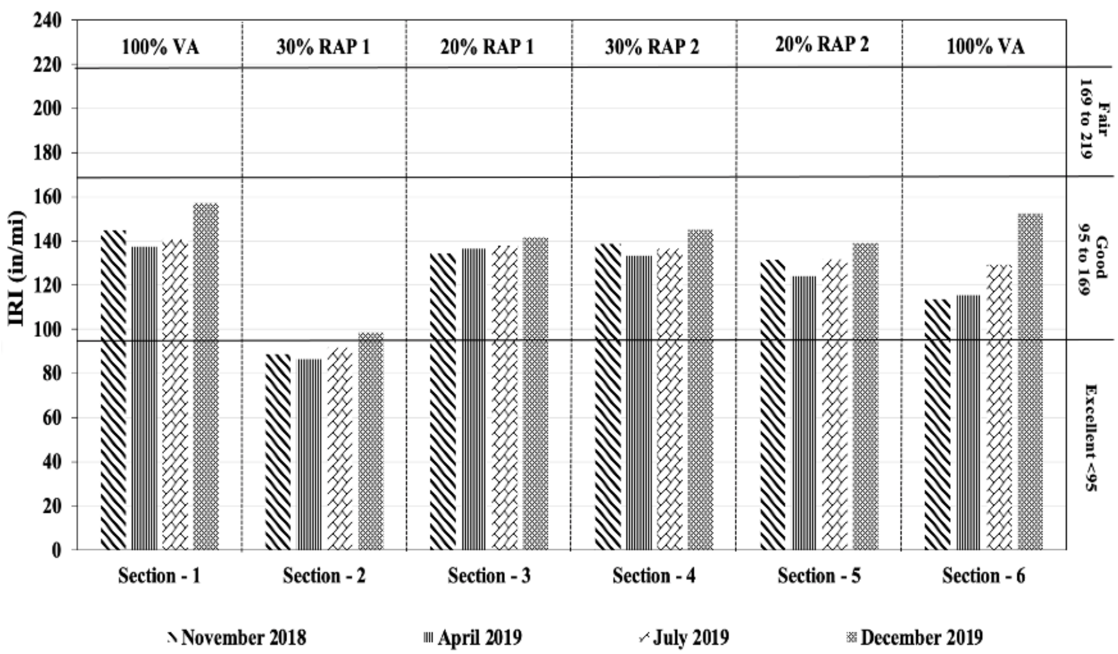

(c)

Figure 36: Falling Weight Deflectometer (FWD), Light Weight Deflectometer (LWD), and International Roughness Index (IRI) Results to Characterize the Asphalt Layer at the Test Site

Table 12: Explanation of the International Roughness Index (IRI) Ratings on Pavements (VDOT State of Pavement Report, 2016)

Ride Quality	IRI Rating (inch/mile)	
	Interstate & Primary	Secondary Roads
Excellent	<60	<95
Good	60 to 99	95 to 169
Fair	100 to 139	170 to 219
Poor	140 to 199	220 to 279
Very Poor	≥ 200	≥ 280

Characterization of the Subgrade

Subgrade has been characterized extensively during the construction of the site. The results (as discussed earlier) showed competent and uniform ground conditions (see Figure 27). FWD surveys conducted after construction were used to back-calculate the modulus of the subgrade as it relates to measurements at different times within the season. Figure 37 shows these back-calculated modulus values.

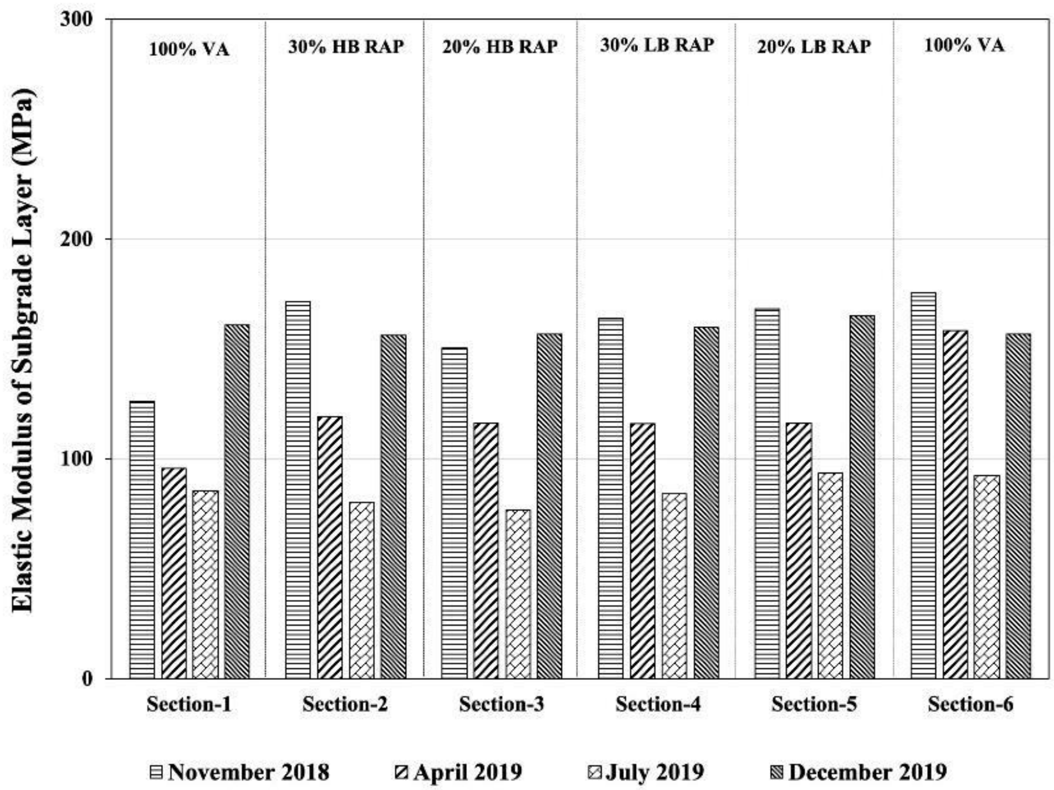

Figure 37: Average Falling Weight Deflectometer (FWD) Elastic Modulus Values on Subgrade Layer (Four Different Testing Period Time)

Results shown in Figure 37 show that the modulus of the subgrade varies with seasonal changes. In colder and less rainy time periods (i.e., November and December), the modulus values increase and in warmer and more precipitation time periods (i.e., April and July) the modulus values decrease. This trend is in agreement with what would be an expected behavior. What is more important is that, for a given time period within the season, the overall back-calculated modulus values are in agreement with each other (with the exception of Section 1). The discrepancy in Section 1 is not completely known although the survey results during construction also shows the subgrade underneath Section 1 to be slightly different than subgrade underneath other sections (Figure 27). Although during construction this layer shows a stronger behavior, the after-construction data indicates the opposite. Determining the exact reason for this is impossible however as can be seen in the comparison of data from July 2019, there has been a time period where the data from Section 1 is in agreement with the data from other Sections. Therefore, it is concluded that subgrade maintained its uniformity even after construction, however seasonal conditions effect the magnitude of the back-calculated modulus values.

Characterization of the Base Course Layer

Results discussed above related to asphalt layer and subgrade showed that in all sections, these layers showed somewhat uniform behavior. This means, if there are any discrepancies in the FWD, LWD, and IRI from one section to another, it is most likely that such differences would be related to the performance of the base course sections.

Figure 38 shows the back-calculated modulus values from the LWD surveys for the base course layers in each of the section. These values were determined based on the multi-layer back calculation as explained in Equation 7. The modulus of the asphalt layer and the combined modulus of the asphalt layer and base course (referred as multilayer modulus) were used as inputs to the Equation 7. The modulus of the asphalt layer used in Equation 7 was directly obtained with the LWD equipment with a set-up of having a 6-inch (0.15 m) diameter plate. The multilayer modulus that was used in Equation 7 was determined with the same LWD equipment with 12-inch (0.3 m) diameter plate. The results show that LWD equipment was not very effective in determining the effects of seasonal changes. However overall, the modulus values of all sections were very close to each other except in 30% RAP 1 section. This section showed the highest modulus values compared to the other sections. This observation is consistent with what has been observed during the laboratory M_r tests (see Figure 18a) as well as what would have been expected based on the laboratory evaluations from the PD tests (see Figure 21). Also, the better performance of the 30% RAP 1 section can be seen from the IRI data (Figure 36c), where that section has been rated as "excellent" for ride quality.

Figure 39 shows the back-calculated modulus values of the base course. The effects of seasonal changes can be seen in this data set. As observed in the survey results from the asphalt layer and subgrade, in colder and less rainy time periods, the modulus values are greater than the times of warmer and more rainy periods. Unlike the LWD results, Figure 39 shows the modulus of the base course in the 100%VA section (Section 1) to be much less than modulus values from other sections. However, when the comparison was made without this data, the modulus values from all sections appear to be relatively close to each other. The clear trend that was observed from LWD data as it relates to the performance of the 30% RAP base course layer is not seen as clearly from the FWD data. However, the data shown from FWD also does not contradict the trend observed from LWD. Overall RAP-VA blends appear to have performed similarly or better than the 100% VA base course.

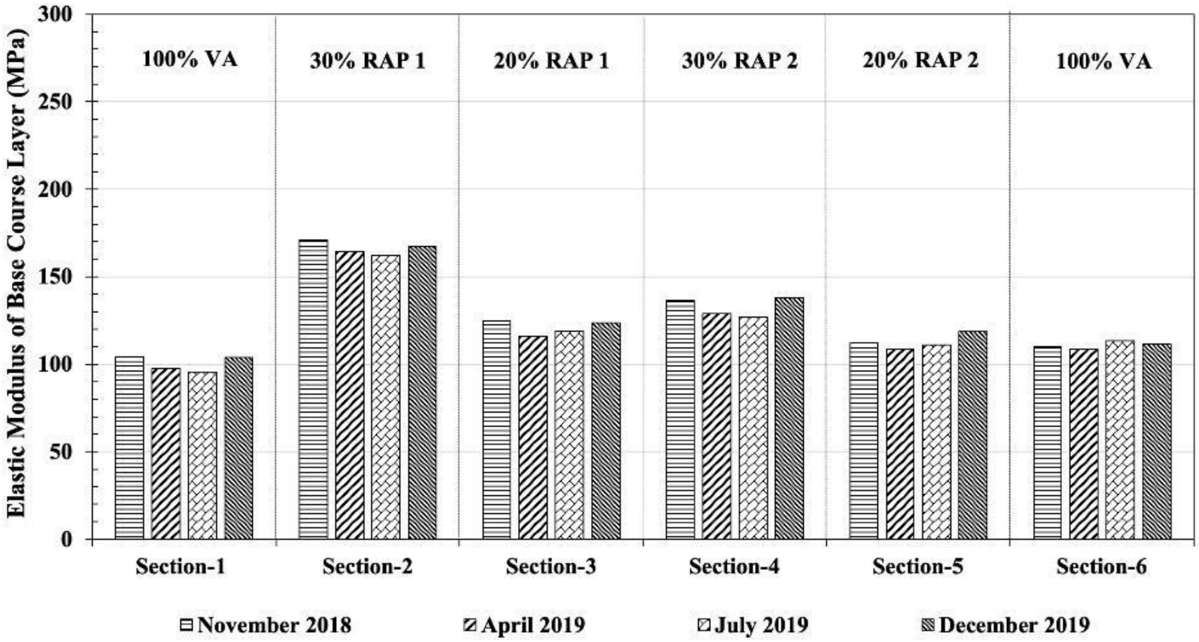

Figure 38: Average Light Weight Deflectometer (LWD) Elastic Modulus Values on Base Course Layer (Four Different Testing Period Time)

The discrepancy observed in Section 1 from the FWD modulus back calculations (Figure 39) appeared to be similarly reflected in the data for the asphalt layer (Figure 36a) and subgrade (Figure 37). Unlike LWD, the back calculations from FWD require the raw data from different depths to be processed all at once (not independent to each other). Considering that asphalt layer was constructed with a material that was manufactured at a plant and placed uniformly with the same equipment throughout the site, it is believed the cause of the observations from Section 1 is either due to the base course or subgrade. Evaluation of the subgrade indicate that if anything, the subgrade underneath the section 1 is stronger than the subgrade in other sections. This leaves the reasoning of the discrepancy in Section 1 to be primarily due to the potential issues with the base course. Considering that the base course in sections 1 and 6 are both constructed with the same material (100% VA), it may be possible that there were issues associated with the compaction during the field application (not an issue related to the properties of the material). The LWD tests that were reported in Table 10 during construction of the base course were conducted on the material in Section 6 and not Section 1 (both having 100% VA). This is because of the reasons that were beyond the control of the research team at the time of the construction. Therefore, the exact relative compaction of Section 1 is not known. However, Section 1 had embedded instruments and the results from these instruments have been discussed in the previous subsection above. As can be seen from Figure 31b, the base course in Section 1 has gone through initial vertical deflection that is larger than any RAP-VA blends. Combining all of these observations together, it is believed that the back-calculated FWD modulus values reflect the issues associated with the 100% VA base course in Section 1 accurately. However, the data for the same section for asphalt layers (Figure 36a) and subgrade (Figure 37) do not capture the behavior accurately and the results are associated with the reflection of the issues observed within the base course in Section 1. LWD did not show the same problem that was observed in FWD data (comparison of Figures 38 and 39). It is believed that the reason for this is associated with the significant differences in the magnitude of the dropped weight from these two types of equipment. In the case of FWD, the dropped weight was approximately 4 times more than the dropped weight in LWD.

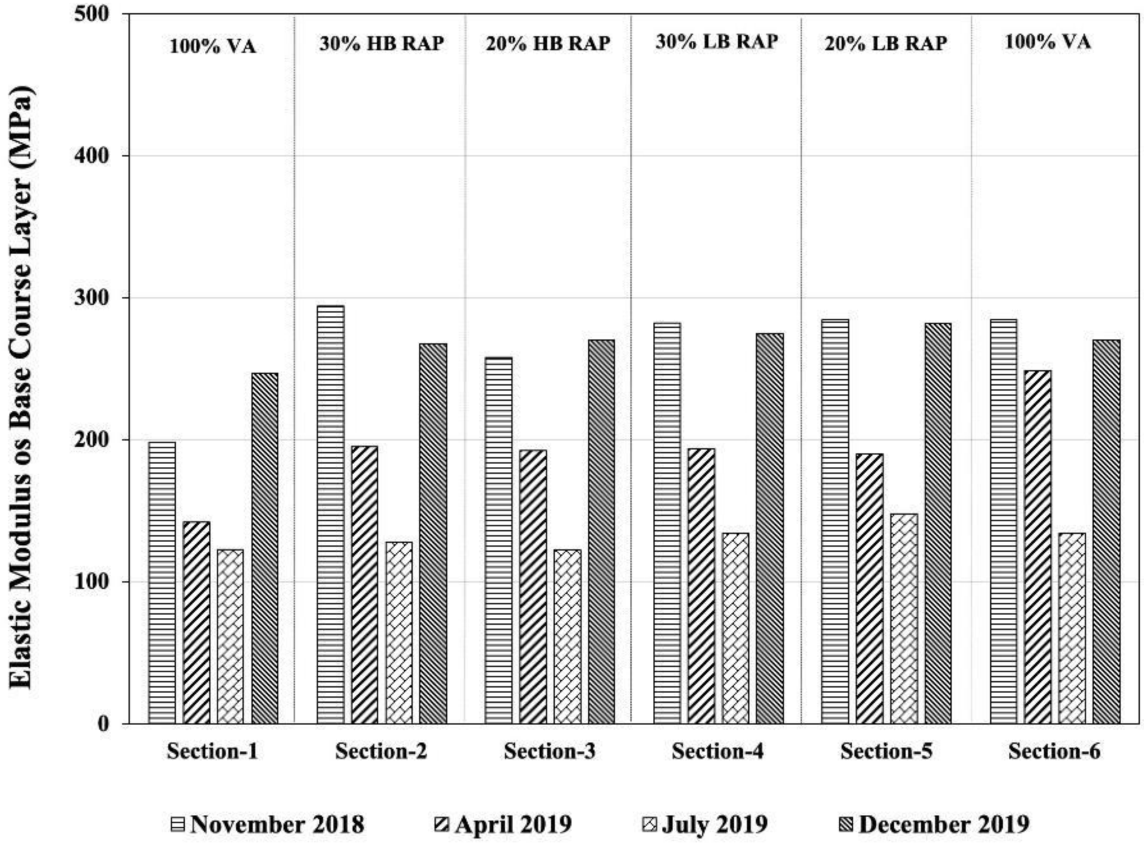

Figure 39: Average Falling Weight Deflectometer (FWD) Elastic Modulus Values of Base Course Layer (Four Different Testing Period Time)

Summary of Results From During and After Construction Field Demonstration

The findings show the suitability of using LWD and modified speedy moisture content test to conduct quality control tests during construction for the sections with RAP-VA blends. These findings are confirmed both from the laboratory investigation and field demonstration.

The results obtained from the field tests indicated that limiting the threshold to 30% RAP for high binder material (RAP 1) and 20% RAP for low binder material (RAP 2) will result in performances similar or slightly better than the 100% VA. The performances referred herein include both the observations during construction as well as after construction for a period of one year under different seasonal conditions.

The field instrumentation data showed that water penetrates into the base course from the asphalt layer as was evident from the water content changes in the base course after precipitation. Although it was not the focus of this study, the effects of changes in water content of the base course after the initial compaction of the layer to optimum moisture content has also been investigated. The findings have been presented at the ASCE Geocongress 2020 and accompanied by a conference article by Ullah, S. and Tanyu, B. F. (2020), ***doi: 10.1061/9780784482810.063***.

DISCUSSION

As presented in the introduction section, previous researchers have indicated that addition of RAP improves the M_R of RAP-VA blends regardless of the binder content. The research conducted herein agrees with these previous findings as can be seen from the laboratory tests (Figure 18, 23, and 24) and as observed from the field measurements (Figure 38 and 39). However, M_R test results are not good indicators for the long-term performance as the research presented herein also showed that addition of RAP adversely affects the deformations of the RAP-VA blends. This observation is not only a function of the RAP percentage but also a function of the binder content of the selected RAP. Figures 20 and 21 show this affect from the laboratory evaluations conducted in this study. In both figures, the addition of RAP improves the performance of the RAP-VA blend to a point but then additional RAP decreases the performance. Figure 21 depicts the conditions under 100,000 repeated load cycles where the stress – strain curve for each RAP-VA blend becomes asymptotic. This means addition of repeated loads is not expected to significantly change the accumulated permanent strains. Based on this observation, with the data shown in Figure 21, a threshold for the RAP-VA blends could be established from the laboratory evaluations as 30% for RAP with average binder contents in a range from 5.6 and 5.8% (i.e., high binder content) and 20% for RAP with average binder contents in a range from 4.5 and 4.7% (i.e., low binder contents). The significance of this threshold can also be seen from Figure 22 where CBR values of the RAP-VA blends start to show trends that are below the CBR of the 100%VA based on the above-mentioned thresholds. The embedded field instrumentation data as seen in Figure 31 supports the observations from the laboratory tests. The field data show that section constructed with 30% RAP1 (high binder content) and 20% RAP2 (with low binder content) had vertical deflections better than the 100% VA. Based on the combined laboratory and field studies, the overall results support that for the RAP generated in Virginia, there should be a threshold to blend RAP with VA between 20 and 30% depending on the binder content of the 100% RAP. Figure 40 is created to depict this threshold graphically, where x-axis is used as the input by the user to determine the output in y-axis. For example, if the average binder content of the 100%RAP was determined to be as 5%, the maximum allowed RAP percentage should be kept at 24%. If the average binder content of the RAP happens to be greater than 5.6%, the maximum allowed RAP percentage in a given blend should be kept at 30%. Figure 40 does not depict a minimum RAP percentage as such threshold is not believed to pose concerns for potential adverse effects. It should be kept in mind that, Figure 40 presents the thresholds for RAP binder content based on the conditions before RAP is blended with VA. Based on the results in this study the maximum binder content of the blends could range approximately between 0.9 and 1.7%. It is important to note that this research has been conducted with RAP and VA, both of which containing aggregate pieces of diabase and with 100%RAP being processed in accordance to what is referred as fine-processed RAP in Virginia (max. particle size of 1-inch).

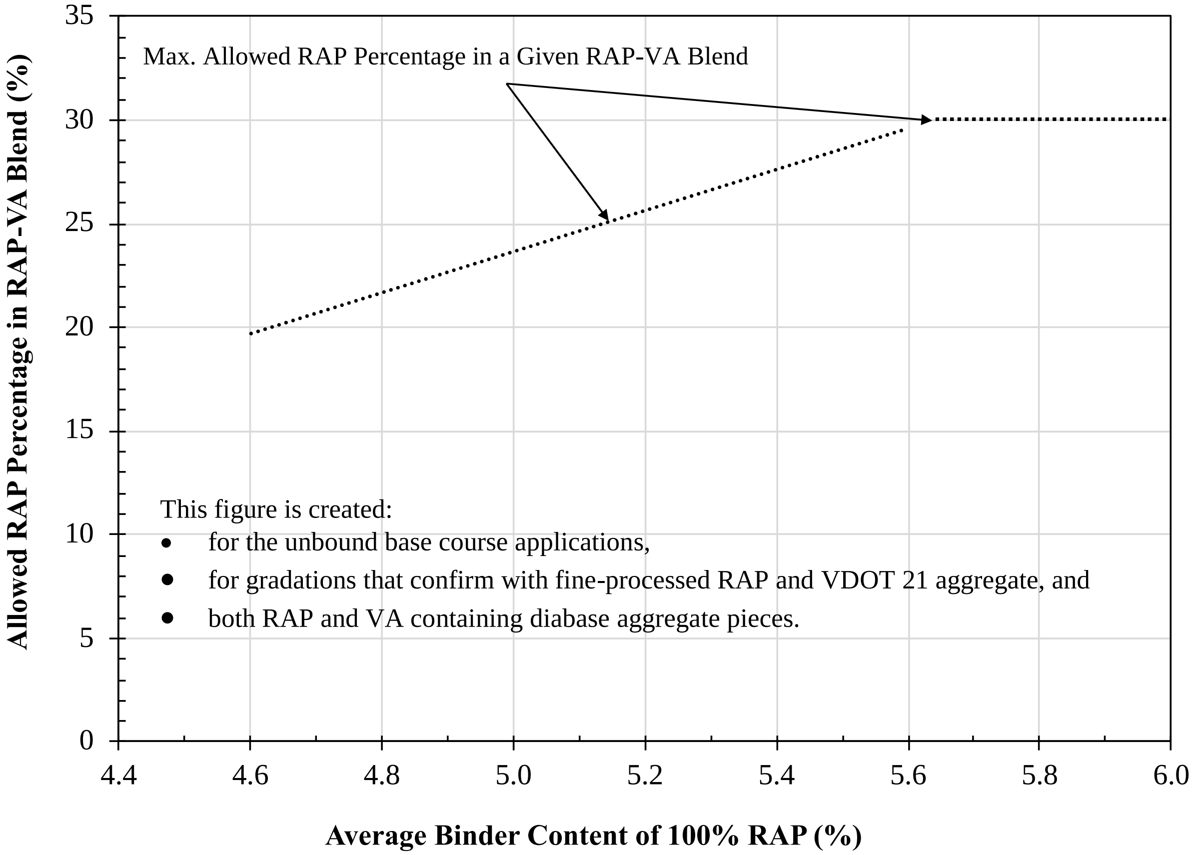

Figure 40: Relationship Between Acceptable RAP Percentage in a Given Reclaimed Asphalt Pavement and Virgin Aggregate (RAP-VA) Blends Based on Average Binder Content of RAP Used to Create the Blends

Table 1 summarizes the findings from existing DOT specifications that allow the use of RAP for unbound base course. The range of RAP percentage allowed by DOTs is between 15 and 100%. Also, some DOTs do not specify the RAP percentage but limit the binder contents. The range of binder contents allowed by DOTs is found to be between 1.5 and 4.0% (Table 1). In none of the specifications the age of the RAP is considered as part of the evaluation. The range of limitations depicted in Table 1 are primarily associated with different maximum particle sizes for RAP. Among the list, only Arkansas and New Mexico limit the maximum particle size of RAP similar to the size evaluated in this study. Arkansas has a limitation that the RAP-VA blend to have a binder content between 2.5 and 3%. New Mexico has a limitation to add up to 50%RAP (no limitation of binder content). When compared with Figure 40, the limitations set forth by New Mexico and Arkansas are different than what is determined from this study. Several factors may influence the differences between what has been developed in this study as a threshold and what the limits established by New Mexico and Arkansas. One of the factors could be related to the differences in RAP as it relates to aging and the types of aggregates that RAP contains. The other factor could be due to the differences in methods used to develop these thresholds. Unfortunately, the approach followed by other DOTs to determine their limits is unknown. It is important to note that the threshold developed in this study (as requested by VDOT) was established based on comparing the performance of RAP-VA blends with the performance of the 100% VA to result in a better or equal performance. Therefore, a direct comparison between the limits established by different DOTs should only be made if the data and the details of how these thresholds were determined by each DOT are available. This also emphasizes the importance of each DOT conducting their own evaluations and developing their specific thresholds. The study presented herein is specific to the materials produced in Virginia and relates to the performance of the RAP-VA blends to the evaluated 100% VA.

Findings from this study and variation of the thresholds from different DOTs (Table 1) indicate the importance of properly characterizing RAP that will be used in construction. As demonstrated in this study, at a minimum, the maximum particle size of RAP and the binder content of the 100%RAP need to be known to determine a threshold for the RAP percentage to be blended to VA. In addition to proper characterization, how the RAP and VA is blended also need to be documented. As can be seen from Figure 16, blending of RAP and VA to create a fixed gradation does not result in a blend where addition of RAP improves the rutting behavior of the material to a certain percentage of RAP. Therefore, RAP-VA blends should be created by addition of RAP with specific percentage as determined by weight. This approach also appears to be consistent with the approaches used by other DOTs.

As cited throughout the report, the previous literature also indicates that RAP is a visco-elastic material and when tested as 100% at temperatures greater than 72 degrees Fahrenheit (22 degrees Celsius), it shows extensive rutting potential. Considering that the RAP in this study was evaluated to be used as unbound base course for paved structures, it was important to determine the fluctuation of the temperatures within the base course as this information was not found in previous studies. Figure 30 shows that in this study even the average temperature in the field is within the range of problematic temperatures determined by the previous research. In summer months, the temperatures have also risen 1.4 times of 72 degrees Fahrenheit. This information is important that under no circumstances 100% RAP should be considered as unbound base course in Virginia if the intention is to keep the base course as unbound granular material. However, as supported by the field data in this study, when RAP in RAP-VA blends is limited to 30%, even under the temperature fluctuations observed in the field, the RAP-VA blends appeared to show vertical deformations that are reasonable compared to 100%VA. This information supports the importance of keeping the RAP-VA thresholds to the limits determined in this study (Figures 40).

Table 1 shows that many DOTs that allow the use of RAP refer to nuclear density gauge as quality control tool. However, as demonstrated in this study, nuclear density gauge does not work properly for this purpose (Figure 28). The bases of the DOTs decision to use nuclear density gauge for RAP-VA blends could also be due to the differences in RAP produced from one State to another as shown in Table 1. RAP particles greater than 1-in (25.4 mm) typically contains much less binder contents than the fine processed RAP used in this study. Therefore, perhaps in some DOTs the allowance of nuclear gauge is because the RAP used in that State does not contain as much binder content as the RAP produced in Virginia. Minnesota, New York, and North Dakota are the only States that refer to alternative means to perform quality control with RAP base course. New York appears to just rely on visual inspection and North Dakota refers to the use of traditional methods such as sand cone and rubber balloon, which takes time to implement in the field. Minnesota appears to be the only state that suggests the use of DCP and LWD. As discussed in this report, DCP may not be very suitable for materials with aggregate size particles as the presence of these particles sometime result in misleading results. This leave the consideration of LWD as the most suitable equipment as was investigated in this study.

This study demonstrated that LWD and speedy moisture content test devices could be used in the field to check the quality of the compaction and moisture content when RAP-VA blends are used to construct unbound base aggregates. Minnesota DOT also allows the use of such device for quality control of base course aggregates. However, it should be noted that sometimes due to the thickness of the base course, the readings from the LWD may not directly reflect the parameters of the base course by itself as the depth of influence of the LWD equipment may exceed the thickness of the RAP-VA blend. Understanding the existence of such scenario in the field is of paramount importance as demonstrated in this study. Incorrect interpretation of the results would have significant impacts on accepting or declining the constructed RAP-VA layer. Guidance on this topic have already been included to this report.

Accurate characterization of the RAP material and use of proper tools for quality control in the field are essential to proper use of RAP-VA blends to minimize potential adverse effects in terms of rutting failures. Once the pavement is constructed, field behavior of the overall pavement structure could be monitored using existing FWD, LWD, and IRI tests. The overall performance of the base course is not only a function of the RAP-VA blends but also functions of subgrade and the HMA layers. In this study RAP-VA blends were constructed over a competent subgrade and below a 9.5-in (0.24 m) thick HMA layer. The effects of softer subgrade have not been investigated in this study. However, laboratory PD tests were conducted based on simulating the presence of 3-in (0.08 m) HMA layer above the base course. Such condition has not been tested in the field.

CONCLUSIONS

- *Evaluation of fine processed RAP obtained from 14 different plants in Virginia showed that the asphalt binder content is clustered in two distinct ranges; around 4.6% (low binder content) and around 5.8 % (high binder content).*

- *Virtually all RAP samples (with one exception) and VDOT 21A base aggregate obtained during this study contained diabase aggregate rock type.*

- *No measurable performance advantage was observed with blends created using engineered gradations (target grain size distribution).*

- *While the addition of RAP to virgin aggregate can improve the resilient modulus of the blend, it can also lead to problematic long-term deformations.*

- *Resilient modulus laboratory tests are not suitable for determining the maximum percentage of RAP that may be blended with virgin aggregate. However, for a given RAP percentage and binder content, resilient modulus values are indicative of the performance differential.*

- *Permanent deformation tests are the most effective approach to compare the long-term performance of a blend to that of a virgin aggregate. The most appropriate predictor of the long-term performance was determined to be the total accumulated strain at the end of 100,000 loading cycles. This is because at 100,000 load cycles, the permanent strain values reach an asymptote. Therefore, the effect of increased load cycles on accumulating additional permanent strain values are minimized.*

- *Laboratory studies indicated that the adding fine processed RAP to virgin aggregate improves the resulting blend performance, as manifested by accumulated strains, up to a certain threshold percentage of RAP. Distinctly adverse outcome was observed when this threshold level was exceeded. These laboratory findings were confirmed by the subsequent field study. Thus, adding a certain percentage of RAP to virgin aggregate can actually improve the long-term performance of unbound base aggregate.*

- *Laboratory CBR test results confirmed the evidence of RAP thresholds obtained from the permanent deformation cyclic load testing.*

- *The potential for deformation is not only a function of the RAP percentage but also a function of the binder content of the selected RAP. For the materials tested through this research, performance (as defined by permanent deformation and CBR tests) was not a problem in blends where the allowable percentage of fine processed RAP was no more than:*

  — *20% for RAP containing 4.6% or less binder content*
  — *30% for RAP containing 5.6% or more binder content.*

- *Field quality control for the compaction of RAP blends can be effectively accomplished by using LWD and speedy moisture content tests.*

RECOMMENDATIONS

1. *VDOT's Materials Division should allow the use of fine processed RAP (100% of the particles finer than 1-inch) in unbound aggregate base material.* The implementation process should be gradual, with a constant feedback on field performance. Initial projects should be constructed with blends not exceeding 20% RAP regardless of the asphalt binder content. The RAP limit threshold value may be ultimately increased up to approximately 30% if warranted by field performance.

2. *VDOT's Materials Division should require reporting the asphalt binder content (at least 3 tests per day of production) of RAP source and RAP/aggregate blends used on all construction projects.* Testing should follow the Virginia Test Method (VTM-102) requirements.

3. *VDOT's Materials Division should consider allowing the use of LWD equipment with the Speedy moisture content tester as an alternative method for field quality control of compaction of RAP/aggregate blends.*

4. *VTRC's Pavement Research Advisory Committee should consider pursuing future research to evaluate the effects of RAP aging and binder chemical additives on the performance of RAP/aggregate blends.* The outcome of this research would be used to optimize the allowable RAP percentage in an arbitrary aggregate blend.

IMPLEMENTATION AND BENEFITS

Implementation

With regard to Recommendation 1, VDOT's Materials Division, with the assistance from VTRC and GMU, will develop a Special Provision (SP) that allows the use of RAP in unbound aggregate base material for implementing this recommendation. It is important to note that development of the SP is contingent upon successful development of acceptable field quality control method for compaction of RAP/Aggregate blends (Please see the implementation plan for Recommendation 3 below).

With regard to Recommendation 2, the testing and reporting requirements for the asphalt binder content of RAP source and RAP/aggregate blends will be specified in the SP to be developed.

With regard to Recommendation 3, VTRC will conduct a study to evaluate the suitability of the Nuclear Density Gauge (NDG) to accurately determine the density of RAP/virgin aggregate blends and assess the relative merits of each tool (LWD and NDG). The suitability of LWD to perform quality control of RAP/aggregate blends during construction was evaluated in this research, but the NDG was not evaluated. The NDG is proven equipment for use in quality control of virgin aggregate base course in the field to determine the moisture content and density. However, in the presence of RAP, as demonstrated in this study, NDG has limitations concerning determination of the moisture content of the RAP/virgin aggregate. If this additional study confirms that NDG provides fast and accurate results for in-place density measurements, the use of NDG with a speedy moisture content testing device will be preferentially considered. VTRC's study is contingent on VDOT conducting further trials with RAP/virgin aggregate blends in base courses.

With regard to Recommendation 4, a possible need for further research on RAP material properties stems from the current study's finding that the binder content is not the sole determinant of optimum RAP/aggregate blend performance. VTRC and VDOT's Materials Division will discuss the need for this future research. If it is determined that additional research would be beneficial, the research need statement will be developed and submitted to the Pavement Research Advisory Committee for scoring and prioritization.

Benefits

The benefits of implementing Recommendations include the harmonized use of RAP as unbound base course in road construction across the VDOT network in a cost-effective manner with minimal adverse consequences. The revised specifications and guidelines will facilitate consistent, efficient, and systemic use of RAP blended with virgin aggregate on road projects. It is anticipated that subdivision and secondary roads in particular will benefit appreciably from implementation of these recommendations, resulting in reduced construction costs. It is envisioned that with time and demonstrated confirmed performance evaluations, VDOT will increase the maximum allowed RAP threshold.

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REFERENCES

AASHTO T217 (2014) Determination of Moisture in Soils by Means of a Calcium Carbide Gas Pressure Moisture Tester. AASHTO, Washington, D.C.

Abdelrahman, M., T. Alam, and J. Zollars. Performance of high recycled asphalt pavement (RAP) content as base layer in flexible pavement, *The Journal of Solid Waste Technology and Management*, v. 36, 2010, p. 131-142.

Arizona Department of Transportation. Standard Specification for Highway Construction. Phoenix, 2008. http://origin.azdot.gov/docs/business/2008-standards-specifications-for-road-and-bridge-construction.pdf?sfvrsn=0

Arkansas Department of Transportation. Standard Specification for Highway Construction. 2014. https://www.arkansashighways.com/standard_spec/2014/2014SpecBook.pdf

Alam, T.B., Abdelrahman, M. and Schram, S. Laboratory Characterization of Recycled Asphalt Pavement as a Base Layer, *International Journal of Pavement Engineering 11*, 2010, no. 2: 123-131.

Al-Qadi, I. L., Loulizi, A., Elseifi, M., and Lahouar, S. The Virginia Smart Road: The impact of pavement instrumentation on understanding pavement performance, *Journal of the Association of Asphalt Paving Technologists*, 73(3), 2004, pp. 427-465.

Alshibli, K., Abu-Farsakh, M., & Seyman, E. Laboratory Evaluation of the Geogauge and Light Falling Weight Deflectometer as Construction Control Tools. *Journal of Materials in Civil Engineering*, 17(5), 2005, pp. 560–569. https://doi.org/10.1061/(ASCE)0899-1561(2005)17:5(560)

ASTM International. ASTM D1883: Standard Test Method for California Bearing Ratio of Laboratory-Compacted Soils. *ASTM International*, West Conshohocken, PA, 2016.

ASTM International. ASTM E1926: Standard Practice for Computing International Roughness Index of Roads from Longitudinal Profile Measurements. *ASTM International*, West Conshohocken, PA, 2008. https://doi.org/10.1520/E1926-08

ASTM International. ASTM E2583: Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD). *ASTM International*, West Conshohocken, PA, 2015.

Attia, M. I. E. S. Characterization of the structural behavior of reclaimed asphalt pavement as pavement base layer. 2010. North Dakota State University.

Balunaini, U., Damalla, Y., and Madhav, M. Settlement Due to Uniform Circular Load: On Finite Two-Layer System. *Geotechnical and Geological Engineering*, 31(1), 2013, pp. 255–265.

Barksdale, R. D., and Itani, S. Y. Influence of aggregate shape on base behavior. *Transportation Research Record*, 1227, 1989, pp. 173–182.

Bennert, T., Papp, W. J., Maher, A., Jr., and Gucunski, N. Utilization of construction and demolition debris under traffic-type loading in base and subbase applications. *Transportation Research Record* 1714, Washington, DC, 2000, pp. 33–39

Bennert, T., and Pezeshki, D. Performance Testing for HMA Quality Assurance, FHWA-NJ-2015-010, 2015.

Bilodeau, J. P., Doré, G., and Depatie, J. Mitigation of permanent deformation in base layer containing recycled asphalt aggregates. *Canadian Journal of Civil Engineering*, 40(2), 2013, pp. 181-187.

Bozyurt, O., Tinjum, J. M., Son, Y. H., Edil, T. B., & Benson, C. H. Resilient modulus of recycled asphalt pavement and recycled concrete aggregate. In GeoCongress 2012: *State of the Art and Practice in Geotechnical Engineering*, 2012, pp. 3901-3910.

California Department of Transportation. Standard Specification. 2018. https://dot.ca.gov/manuals

Colorado Department of Transportation. Standard Specification for Road and Bridge Construction. 2019. https://www.codot.gov/business/designsupport/cdot-construction-specifications/2019-construction-specifications/2019-specs-book

Chang, G., Q. Xu, J. Rutledge, B. Horan, L. Michael, D. White, and P. Vennapusa, Accelerated implementation of intelligent compaction technology for embankment subgrade soils, aggregate base, and asphalt pavement materials. 2011. Publication FHWA-IF-12-002, FHWA, *US Department of Transportation.*

Chow, L. C. Permanent deformation behavior of unbound granular materials and rutting model development. 2014.

Connecticut Department of Transportation. Standard Specifications for Roads, Bridges, Facilities and Incidental Construction. 2016.

Copeland, A., Reclaimed asphalt pavement in asphalt mixtures: State of the practice. 2011, Report No. FHWA-HRT-11-021.

Dong, Q., and Huang, B. Laboratory evaluation on resilient modulus and rate dependencies of RAP used as unbound base material. *Journal of Materials in Civil Engineering 26*, no. 2, 2014, pp.379-383.

Dynatest, 3031 LWD instruction manual for use and maintenance. Glostrup, Denmark: The Dynatest Group, 2010.

Edil, Tuncer B., Tinjum, J.M., and Benson, C.H., Recycled unbound materials, 2012, No. MN/RC 2012-35.

Florida Department of Transportation. Florida Method of Test for Density of Soils and Bituminous Concrete Mixtures In Place By The Nuclear Method. 2007.

Florida Department of Transportation. Standard Specifications for Road and Bridge Construction. 2019. https://fdotwww.blob.core.windows.net/sitefinity/docs/default-source/programmanagement/implemented/specbooks/july-2019/719ebook.pdf?sfvrsn=bf366a48_2

Garg, N., and Thompson, M. R. Lincoln Avenue reclaimed asphalt pavement base project. *Transportation Research Record*, 1547(1), 1996, pp. 89-95.

Haider, I., Kaya, Z., Cetin, A., Hatipoglu, M., Cetin, B., and Aydilek, A. H. Drainage and mechanical behavior of highway base materials. *J. Irrig. Drain. Eng*., 2014, 10.1061/(ASCE)IR.1943-4774.0000708, 04014012.

Han, J. Principles and practice of ground improvement. Hoboken, 2014, New Jersey: Wiley.

Hoppe E. J., D. S. Lane, G. M. Fitch, and S. Shetty, Feasibility of Reclaimed Asphalt Pavement (RAP) Use as Road Base and Subbase Material, Publication VCTIR 15-R6, Charlottesville, VA: Virginia Center for Transportation Innovation and Research, 2015.

Hossain M. S., and A. K. Apeagyei, Evaluation of the Lightweight Deflectometer for In-Situ Determination of Pavement Layer Moduli. Publication FHWA/VTRC 10-R6, Charlottesville, VA: Virginia Center for Transportation Innovation and Research, 2010.

Jeon, E. J., Steven, B., and Harvey, J. Comprehensive Laboratory Testing and Performance Evaluation of Recycled Pulverized Hot-Mix Asphalt Material. Transportation Research Record: Journal of the Transportation Research Board, (2104), 2009, pp. 42-52.

Khosravifar, S., Large-scale controlled-condition experiment to evaluate light weight deflectometers for modulus determination and compaction quality assurance of unbound pavement materials. (Order No. 10129826), Available from ProQuest Dissertations & Theses Global. (1799923138), 2015.

Kim, Woosung, Joseph Labuz, and Shongtao Dai. Resilient modulus of base course containing recycled asphalt pavement. *Transportation Research Record: Journal of the Transportation Research Board 2005*, 2007, pp. 27-35.

Korkiala-Tanttu, L., Laaksonen, R., and Törnqvist, J., Effect of the spring and overload on the rutting of a low-volume road. HVS-Nordic-research. Tiehallinto, 2003.

Little, D. N., and S. Nair., NCHRP Web-Only Document 144: Recommended Practice for Stabilization of Subgrade Soils and Base Materials. *Transportation Research Board of the National Academies*, Washington, D.C., 2009.

Little, D. N., and Nair, S. Recommended practice for stabilization of subgrade soils and base materials. 2009.

Locander, Robert. Analysis of using reclaimed asphalt pavement (RAP) as a base course material. No. CDOT-2009-5. Colorado Department of Transportation, DTD Applied Research and Innovation Branch, 2009.

Loulizi, Amara, Imad Al-Qadi, Samer Lahouar, and Thomas Freeman. Measurement of vertical compressive stress pulse in flexible pavements: representation for dynamic loading tests. *Transportation Research Record: Journal of the Transportation Research Board* 1816, 2002, pp. 125-136.

MacGregor, Jennifer, William Highter, and Don DeGroot. Structural numbers for reclaimed asphalt pavement base and subbase course mixes. Transportation Research Record: Journal of the Transportation Research Board 1687, 1999, pp. 22-28.

Montana Department of Transportation. Standard Specifications for Road and Bridge Construction. 2020. https://www.mdt.mt.gov/other/webdata/external/const/specifications/2014/2014_stand_specs.pdf

Mehta, Y. and Ayman, A. Alternatives to Nuclear Density Testing, FHWA-NJ-2016-003, 2016.

Minnesota Department of Transportation. Grading and Base Manual, Pavement Engineering Section Grading and Base Unit. 2013.

Minnesota Department of Transportation. Standard Specifications for Construction. 2018. http://www.dot.state.mn.us/pre-letting/spec/2018/2018-spec-book-final.pdf

Mohammad, L. N., Puppala, A. and Alavilli, P. Effect of strain measurements on resilient modulus of sands." Dynamic Geotechnical Testing II. *ASTM International*. 1994.

Mohammad, L. N., Wu, Z., Obulareddy, S., Cooper, S., & Abadie, C. Permanent Deformation Analysis of Hot-Mix Asphalt Mixtures with Simple Performance Tests and 2002 Mechanistic–Empirical Pavement Design Software. *Transportation Research Record*, 1970(1), 2006, pp. 133-142.

Mooney, M., and P. Miller. Analysis of Lightweight Deflectometer Test Based on In Situ Stress and Strain Response. *Journal of Geotechnical and Geoenvironmental Engineering*, 135(2), 2009, pp. 199–208.

Morgan, J. R. The response of granular materials to repeated loading. *Australian Road Research Board Proc*. 1966.

Nazzal, M., Abu-Farsakh, M., Alshibli, K., and Mohammad, L. Evaluating the deflectometer light failing weight device for in situ measurement of elastic modulus of pavement layers. *Transportation Research Record: Journal of Transportation Research Board*, 2007, pp. 13–22.

North Dakota Department of Transportaion. Standard Specifications for Road and Bridge Construction. 2019. https://www.dot.nd.gov/divisions/environmental/docs/supspecs/fullsupplementalspecswith10012018.pdf

NCHRP (National Cooperative Highway Research Program). Recycling Materials for Highways. *Transportation Research Board,* National Research Council Washington, D.C. 1978.

NCHRP (National Cooperative Highway Research Program) Guide for mechanistic-empirical design of new and rehabilitated pavement structure. Draft Final Report No. NCHRP 1-37 A, Washington, DC. 2004.

New Hampshire Department of Transportation. Standard Specifications for Road and Bridge Construction. 2016. https://www.nh.gov/dot/org/projectdevelopment/highwaydesign/specifications/documents/2016NHDOTSpecBookWeb.pdf

New Jersey Department of Transportation. Standard Specifications for Road and Bridge Construction. 2019. https://www.state.nj.us/transportation/eng/specs/2019/Division.shtml

New Mexico Department of Transportation. Standard Specifications for Highway and Bridge Construction. 2019. https://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/2019_Specs.pdf

New York State Department of Transportation. Standard Specifications. 2019. https://www.dot.ny.gov/main/business-center/engineering/specifications/busi-e-standards-usc/usc-repository/2019_1_specs_usc.pdf

Pierce, L. M., Bruinsma, J. E., Smith, K. D., Wade, M. J., Chatti, K., and Vandenbossche, J. M. Using Falling Weight Deflectometer Data with Mechanistic-Empirical Design and Analysis, Volume III: Guidelines for Deflection Testing, Analysis, and Interpretation. FHWA-HRT-16-011, 2017.

Pidwerbesky, B. D. Accelerated dynamic loading of flexible pavements at the Canterbury accelerated pavement testing indoor facility. Transportation research record, 1482, 79, 1995.

Puppala, A. Estimating stiffness of subgrade and unbound materials for pavement design. National Cooperative Highway Research Program Synthesis 382, Transportation Research Board, Washington, DC. 2008.

Puppala, Anand, Chomtid, S., and Bhadriraju, V. Using repeated-load triaxial tests to evaluate plastic strain potentials in subgrade soils. *Transportation Research Record: Journal of the Transportation Research Board* 1913, 2005, pp. 86-98.

Puppala, Anand J., Mohammad, L. N. and Allen, A. Permanent deformation characterization of subgrade soils from RLT test. Journal of Materials in Civil Engineering 11, no. 4, 1999, pp. 274-282.

Schwartz, C. W., Afsharikia, Z., and Khosravifar, S. Standardizing Lightweight Deflectometer Modulus Measurements for Compaction Quality Assurance. Publication MD-17-SHA/UM/3-20, Maryland: Maryland Department of Transportation State Highway Administration. 2017.

Siekmeier, J., Pinta, C., Merth, S., Jensen, J., Davich, P., Camargo, F., Beyer, M. Using the Dynamic Cone Penetrometer and Light Weight Deflectometer for Construction Quality Assurance Minnesota Department of Transportation. 2009.

Skermer, N. A., and S. F. Hillis Gradation and Shear Characteristics of Four Cohesionless Soils. *Canadian Geotechnical Journal*, Vol. 7, No. 1, 1970, pp. 62–68.

Steinbrenner W. Tafeln zur Setzungberechnung, Dic Strasse, 1934, 1:121.

Taha, Ramzi, Galal Ali, Adnan Basma, and Omar Al-Turk Evaluation of reclaimed asphalt pavement aggregate in road bases and subbases. Transportation Research Record: Journal of the Transportation Research Board 1652, 1999, pp. 264-269.

Tanyu, B., Kim, W., Edil, T., Benson, C. Development of Methodology to Include Structural Contribution of Alternative Working Platforms in Pavement Structure. *Transportation Research Record*, 1936, 2005, pp. 70–77. http://search.proquest.com/docview/28845225/

Tanyu, B. F., Yavuz, A. B., and Ullah, S. A parametric study to improve suitability of micro-deval test to assess unbound base course aggregates. *Construction and Building Materials*, 147, 2017, pp. 328-338.

Timm, David H., Angela L. Priest, and Thomas V. McEwen. Design and instrumentation of the structural pavement experiment at the NCAT test track. No. NCAT Report 04-01. National Center for Asphalt Technology (US). 2004.

Tutumluer, E., Ozer, H., Hou, W., and Mwumvaneza, V. Sustainable aggregates production: green applications for aggregate by-products. *Illinois Center for Transportation/Illinois Department of Transportation*. 2015.

Texas Department of Transportation. Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges. 2014. ftp://ftp.dot.state.tx.us/pub/txdot-info/des/spec-book-1114.pdf

Utah Department of Transportation. Standard Specifications for Road and Bridge Construction. 2017. https://www.udot.utah.gov/main_old/uconowner.gf?n=31730316757114651

Ullah, S., Tanyu, B. F., and Hoppe, E. J. Optimizing the Gradation of Fine Processed Reclaimed Asphalt Pavement and Aggregate Blends for Unbound Base Courses. Transportation Research Record, 2672(52), 2018, pp. 57-66.

Ullah, S., and Tanyu, B. F. Methodology to develop design guidelines to construct unbound base course with reclaimed asphalt pavement (RAP). *Construction and Building Materials*, 223, 2019, pp. 463-476.

Ullah, S., and Tanyu, B. F. Effect of Variation in Moisture Content on the Mechanical Properties of Base Course Constructed with RAP-VA Blends. In Geo-Congress 2020: Geotechnical Earthquake Engineering and Special Topics, 2020, pp. 612-620, Reston, VA: American Society of Civil Engineers.

Virginia Department of Transportation. Pavement Design Guide for Subdivison and Secondary Roads in Virginia VTM-8. 2016.

Virginia Department of Transportation. Road and Bridge Standards. 2016. Document can be accessed through http://www.virginiadot.org/business/locdes/2016_road_and_bridge_standards.asp, Accessed September 02, 2017.

Vennapusa, P., White, D., Siekmeier, J., and Embacher, R. In situ mechanistic characterizations of granular pavement foundation layers. *International Journal of Pavement Engineering*, 13(1), 2012, pp. 52–67.

Vesic, A. Discussion, session III, Proc 1st international conference on structural design of asphalt pavements. University of Michigan, 1963, pp 283-290.

Vermont Agency of Transportation. Standard Specifications for Construction. 2018. https://outside.vermont.gov/agency/VTRANS/external/docs/construction/02ConstrServ/PreContract/2018SpecBook/2018%20Standard%20Specifications%20for%20Construction.pdf

Werkmeister, Sabine, Andrew Dawson, and Frohmut Wellner Permanent deformation behavior of granular materials and the shakedown concept. Transportation Research Record: Journal of the Transportation Research Board 1757, 2001, pp. 75-81.

Witczak, M. W. Harmonized test methods for laboratory determination of resilient modulus for flexible pavement design. Project No. NCHRP 1-28A. 2003.

Xiao, Y., and Tutumluer. E. Best Value Granular Material for Road Foundations. No. MN/RC 2012-01. 2012.

Zeng, Menglan, and Donald H. Shields. Nonlinear thermal expansion and contraction of asphalt concrete. Canadian Journal of Civil Engineering 26.1, 1999, pp. 26-34.

APPENDIX
SUMMARY OF POSSIBLE BLENDING OPERATIONS IN THE FIELD

Introduction

A side study was conducted to confirm whether the blending of RAP and VA can successfully be achieved at a quarry that has access to front end loader and pugmill or the operation may be performed at an asphalt plant through storage silos and shaker combination. Samples obtained from the field after blending were compared against the samples blended in GMU’s laboratory. Grain size distribution and binder content test results were used for this comparison.

Option 1 - Blending RAP and VA at a Quarry

The blending procedure at a quarry was achieved by utilizing a front-end loader with a built-in scale in its bucket. For larger quantities, quarries may utilize other means where the weight of the material can be determined. Blends were created based on percent determined by weight. For example, a 20% RAP - 80% VA blend would require 20 kilograms of RAP and 80 kilograms of VA. The respective quantities RAP and VA were dry mixed by tossing and turning the material with the help of loader. After mixing, the material was transported to a pugmill where the RAP-VA blend is mixed again and required amount of water is added. Figure A1 provides a glimpse of quarry mixing with the aid of loader and pugmill.

(a) (b)

(c) (d)

Figure A1: Blending RAP and VA at a Quarry

Option 2 - Blending RAP and VA at an Asphalt Plant
RAP and VA were placed at two separate cold-bin storage silos that had built in automated adjustable release valves that could be operated to control the rate of material released from the silos. These silos were also equipped with an external valve where water could be sprayed onto the material inside of each silo. Depending on the desired RAP-VA blend, the required amount of material was released by the silos on a conveyor belt, which carried the material from each silo to the shaker for mixing/blending. The RAP-VA blend that passed through the openings of the shaker (which were greater than the maximum particle size of the RAP and VA) were then transferred to a truck sitting underneath the shaker. Figure A2 provides a glimpse of blending RAP and VA at an asphalt plant.

(a)

(b)

(c)

Figure A2: Blending RAP and VA at an asphalt plant

Outcome of the Blending Operations

Grain size distribution and binder content tests were adopted to verify the RAP content and uniformity of the blends.

Grain size distribution test

The grain size distribution test was used to confirm that the gradation of the field blended RAP-VA blend were in agreement with the gradation of the RAP-VA blend obtained from the GMU's laboratory tests. The laboratory gradation (or lab blend) was determined based on blending RAP and VA by weight based on the target percentage of RAP to be blended with VA.

Results from quarry blending operation

Figure A3a shows the gradation of the 100% RAP (i.e., RAP 2) and 100% VA before the blending operation. Figures A3b, A3c, and A3d compare the gradation of the laboratory and field blends. The gradation of the field blends were achieved from the samples that were obtained in the field right after the quarry completed the blending operation. It should be noted that field samples may have to be air dried before any grain size analyses.

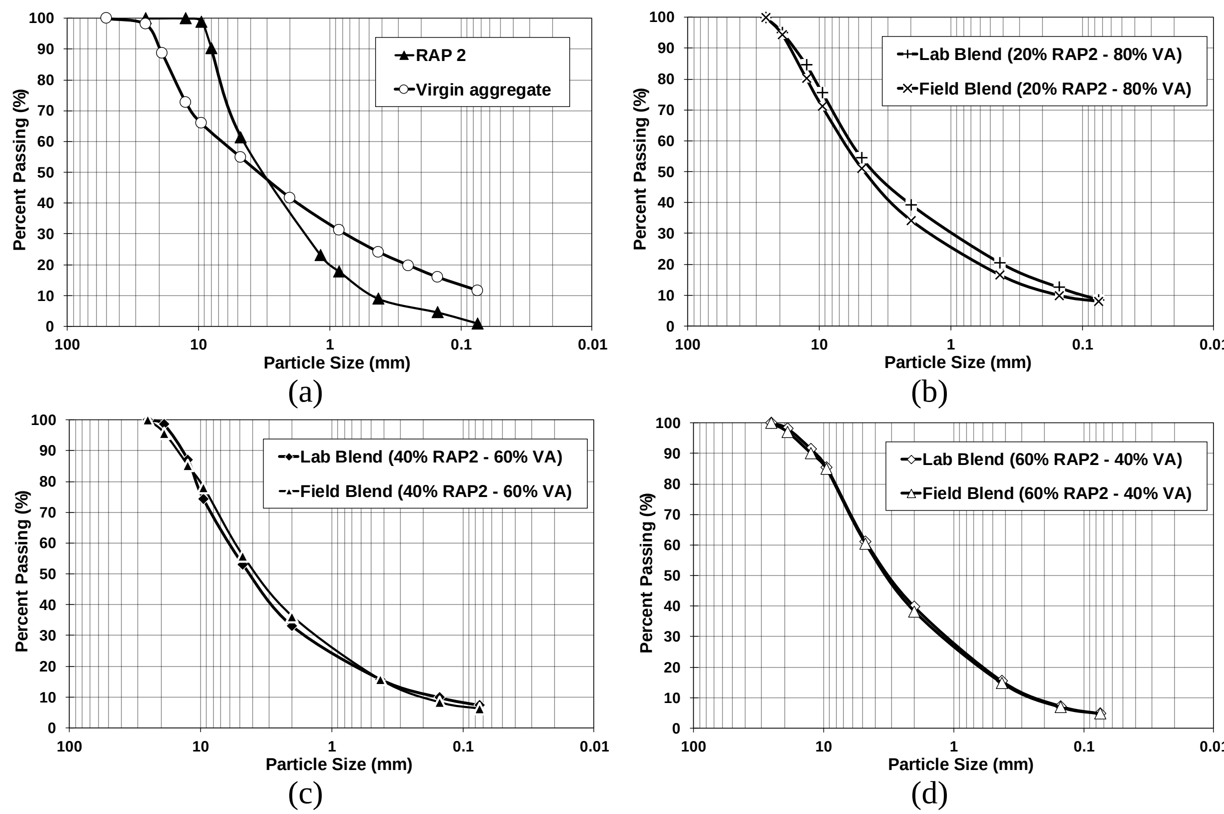

Figure A3: GSD test results for samples prepared after bucket and pugmill mixing in the field.

Results from asphalt plant blending operation

Figure A4a shows the gradation of the 100% RAP (i.e., RAP 1) and 100% VA before the blending operation. Figures A4b, A4c, and A4d compare the gradation of the laboratory and field blends. The gradation of the field blends were achieved from the samples that were obtained in the field right after the asphalt plant completed the blending operation. It should be noted that field samples may have to be air dried before any grain size analyses.

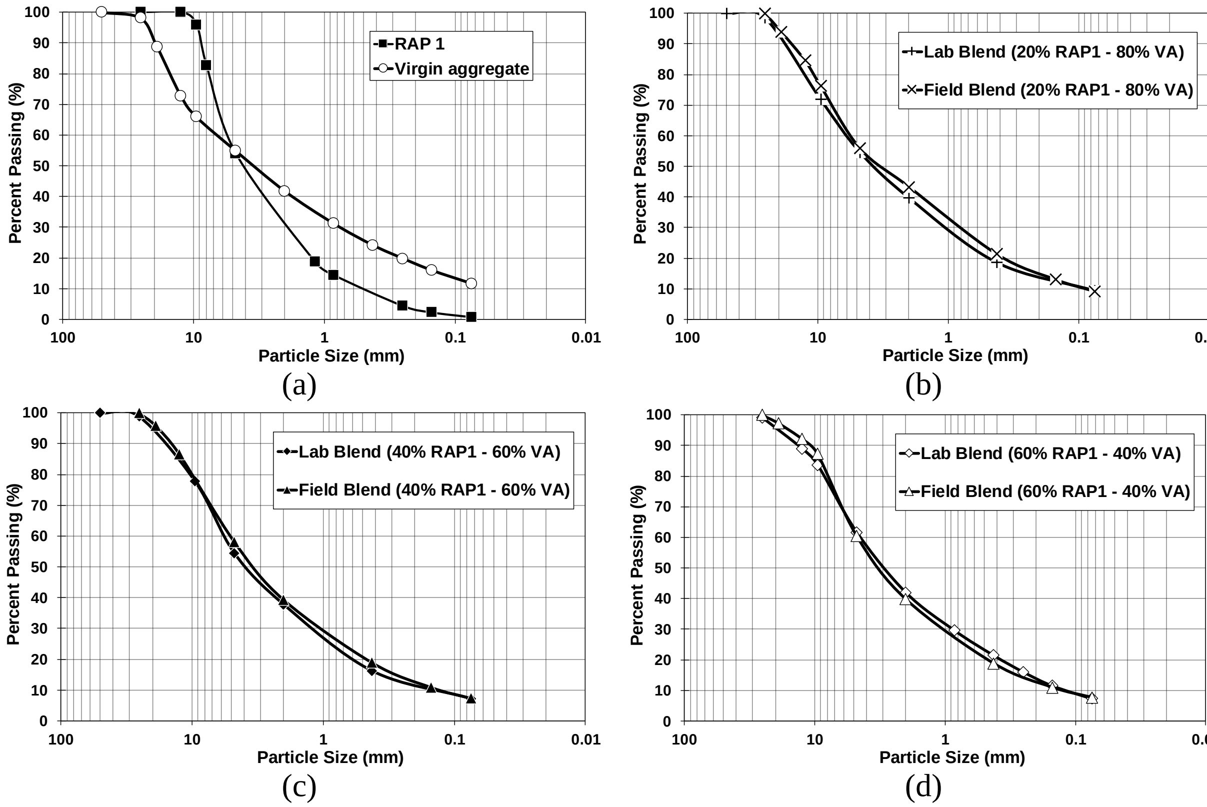

Figure A4: GSD test results for samples prepared after conveyor belt and silo mixing in the field.

<u>Conclusions</u>

Based on the consistency of the grain size distribution of the laboratory and field blended RAP-VA results, it is believed that blending of RAP and VA can be reliably obtained by the quarry and by the asphalt plant options.

Binder content tests

Binder content test results were used to confirm that the field blending resulted in creating a RAP-VA blend that the percentage of RAP blended with VA was within the target (acceptable) range. Such evaluation requires the knowledge of binder content of the 100% RAP that will be blended with VA. Being a reclaimed material, it is most likely that even at a given location at a given stockpile, the binder content of the RAP may vary. Therefore, in order to implement this approach, it is very important to first document the variation of the binder content of RAP within a stockpile that is set aside to be blended with VA. No blending should be allowed before such characterization.

As can be seen from Figures A3 and A4, the RAP used in this study by the asphalt plant is referred herein as RAP 1 and RAP used by the quarry is referred as RAP 2. During the field evaluation, RAP stockpiles that were set aside for this study were sampled and the variation of the binder content of the 100% RAP from that stockpile were documented. The binder content of the RAP may also be reported by the producers but in this study, GMU team has obtained the RAP samples from each stockpile and performed the binder content test ASTM D6307 to determine a statistical range of variation within each RAP stockpile. Fifteen samples from each stockpile was tested and the outcome of the results are shown in Figure A5.

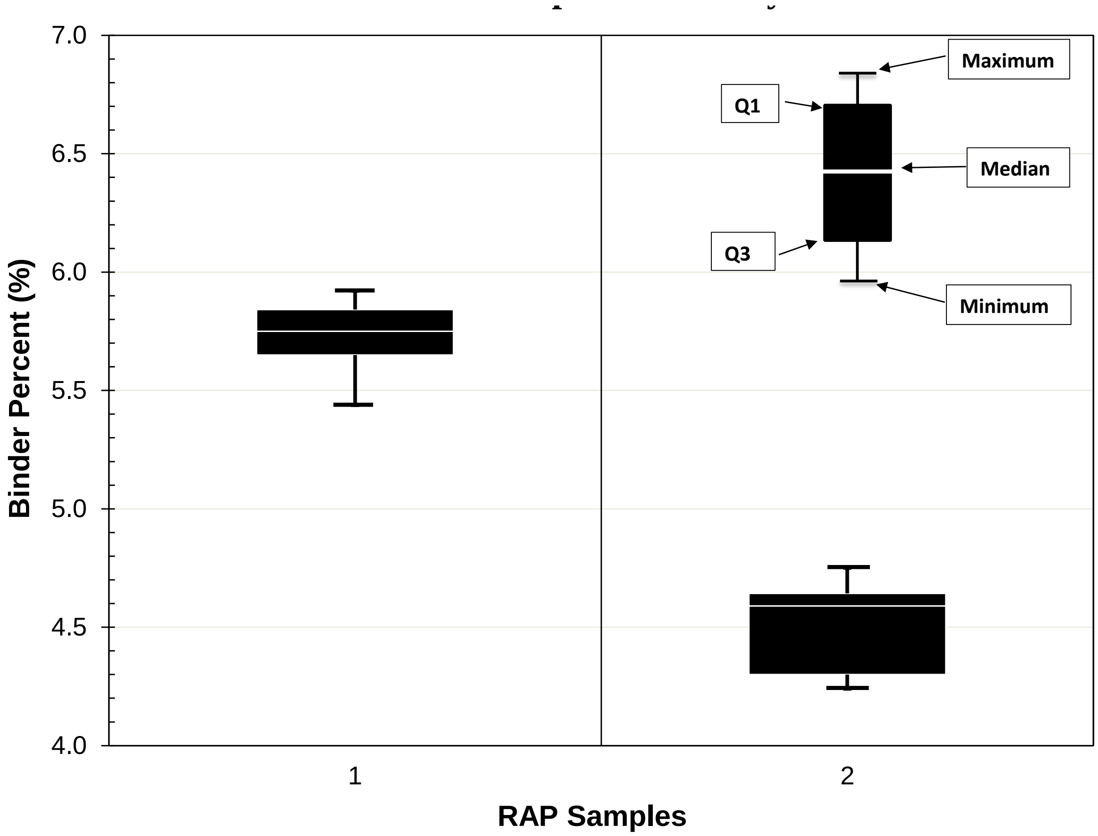

Figure A5: Statistical data on binder content for the selected RAP samples

The information provided from Figure A5 became the bases of how the percentage of RAP blended with VA be assessed. The minimum and maximum values for each RAP type is then used to identify acceptable threshold for RAP-VA blends at intended RAP content. For example, the minimum binder content observed from RAP1 stockpile was 5.44% and the maximum value was 5.92%. Therefore, an acceptable range of binder content for 30% RAP1 – 70% VA would be between 1.63% (5.44% x 0.3) and 1.78% (5.92% x 0.3). Following this approach, a range of what would be expected maximum and minimum thresholds for binder content from each different RAP-VA blend can be estimated. Figure A6 shows the theoretical min. and max. binder contents for each RAP percentage in a given blend that was determined in this study. Figure A6 also shows the actual laboratory binder content test results that were determined by the GMU from the samples that were blended in the laboratory. Comparison of the laboratory and theoretical values allows the assessment of the appropriateness of using the theoretical lines and if needed, a decision could be made for any necessary modifications. Once confirmed, these theoretical lines were then used to assess the blends obtained from the field study.

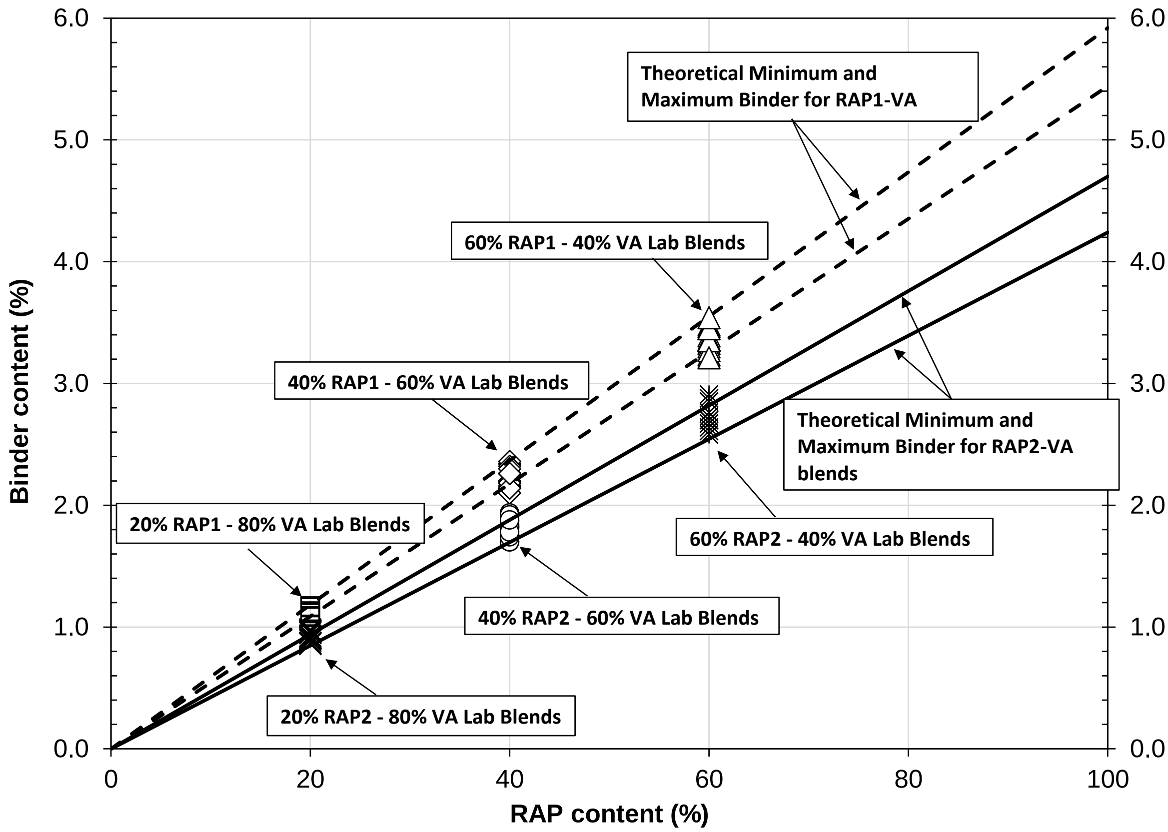

Figure A6: Blends of RAP-VA created at the GMU laboratory

<u>Results from asphalt plant blending operation in the field</u>

Figure A7 provides the results of the binder content tests from the RAP-VA blend samples obtained from the field. Ten samples for each blend were tested for this evaluation. When the binder contents of the field blends were compared against the theoretical threshold that was determined prior to the field blending, the results showed an agreement. This confirmed that in the field blends, the intended percentage of RAP to be blended with VA was within the target RAP percentage.

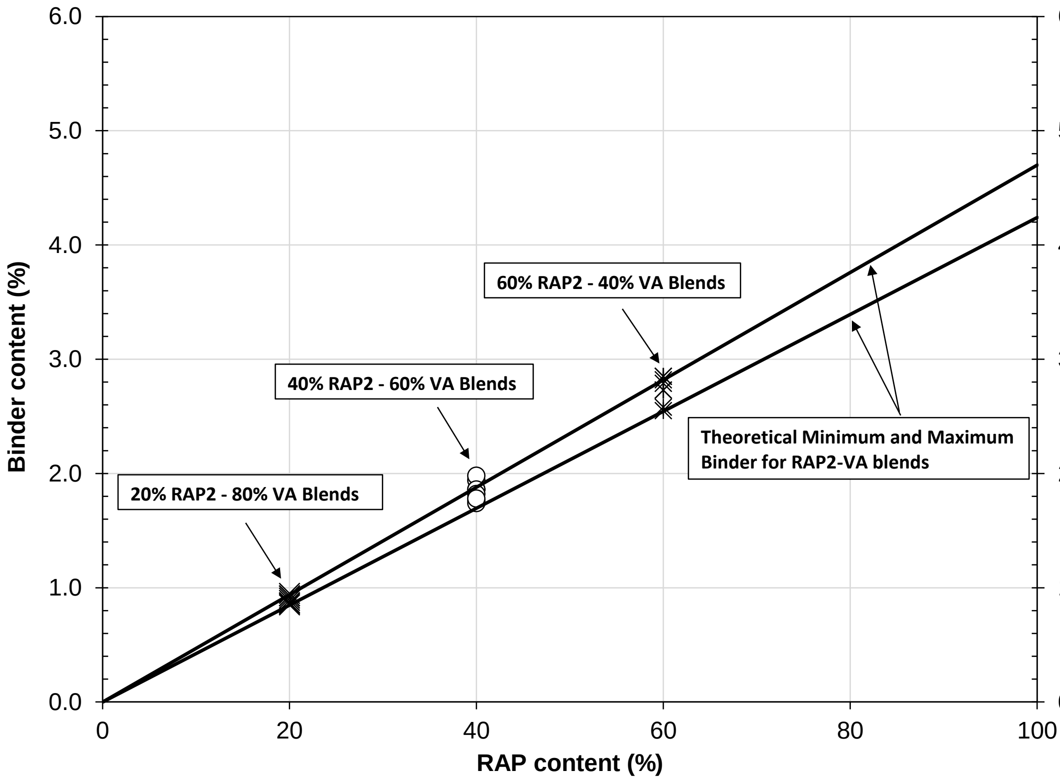

Figure A7: Binder content test results of samples prepared after conveyor belt and silo mixing in the field.

Results from quarry blending operation in the field

Figure A8 provides the results of the binder content tests from the Rap-VA blend samples obtained from the field. Ten samples for each blend were tested for this evaluation. When the binder contents of the field blends were compared against the theoretical threshold that was determined prior to the field blending, the results showed an agreement. This confirmed that in the field the intended percentage of RAP t be blended with VA was within the target RAP percentage.

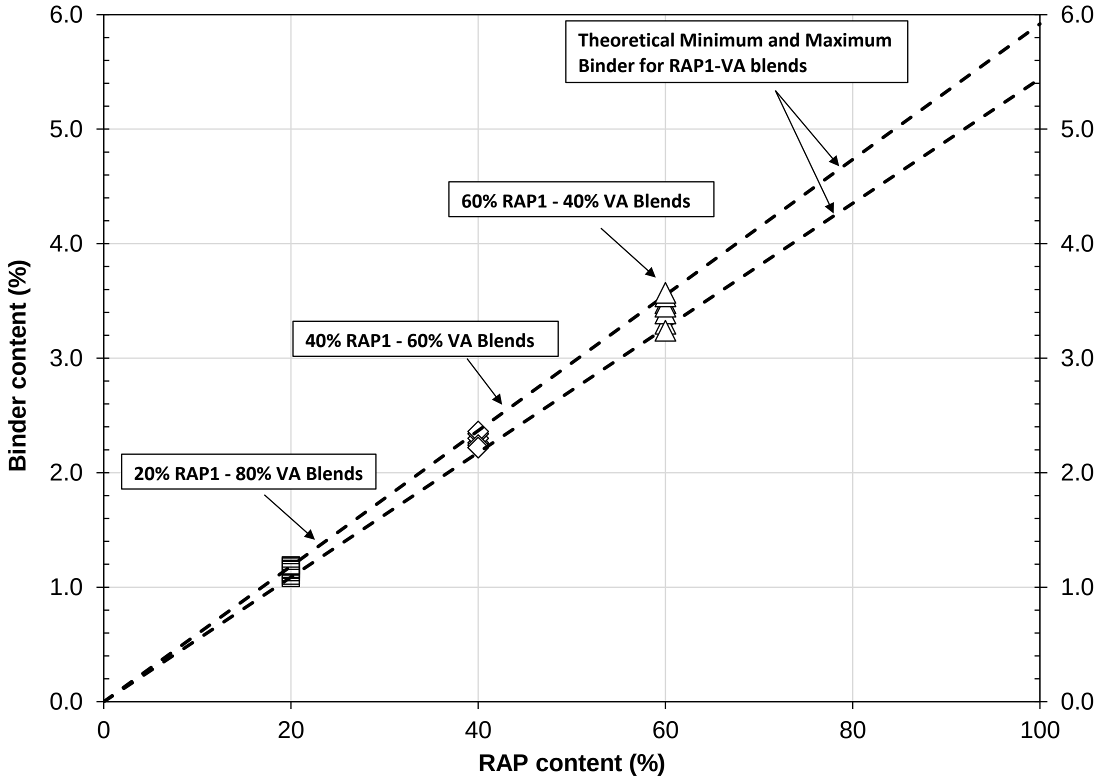

Figure A8: Binder content test results for samples prepared after bucket and pugmill mixing in the field.

Conclusions

Based on the consistency of the binder content test results from the field blended RAP-VA and the theoretically expected range based on the laboratory characterization, it is believed that the percentage of adding RAP can be controlled reliably both by the quarry and by the asphalt plant.

Overall conclusions

Based on the outcome of this study, it is believed that blending RAP and VA can be effectively achieved by the quarry or asphalt plant.