BURNS COOLEY DENNIS, INC.
GEOTECHNICAL AND MATERIALS ENGINEERING CONSULTANTS

Characterization of Asphalt Drainage Course Layers

Prepared for
Mississippi Department of Transportation

State Study No. 181
FHWA/MS-DOT-RD-10-181

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Technical Report Documentation Page

1.Report No. FHWA/MS-DOT-RD-10-181	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Characterization of Asphalt Drainage Course Layers		5. Report Date August 26, 2010
		6. Performing Organization Code BCD No. 060122-1
7. Author(s) L. Allen Cooley, Jr. and Robert S. James		8. Performing Organization Report No.
9. Performing Organization Name and Address Burns Cooley Dennis, Inc. Post Office Box 12828 Jackson, Mississippi 39236		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. SS-205
12. Sponsoring Agency Name and Address Mississippi Department of Transportation P.O. Box 1850 Jackson, MS 39215-1850		13. Type Report and Period Covered Final Report
		14. Sponsoring Agency Code

15. Supplementary Notes
MDOT State Study
Project No.

16. Abstract: Asphalt Drainage Courses (ADCs) have generally been required under all four-lane flexible pavements in Mississippi. Asphalt drainage courses are designed in Mississippi using No. 57 limestone, sandstone or granite combined with 2.5 percent asphalt binder. This research study had two primary objectives related to ADCs. First, the research was to characterize ADC layers for default input values into Mississippi's M-E pavement design system. Secondly, the research was to characterize ADC layers in the field to provide inputs for ADC layers in ELMOD5.

In order to investigate the first objective, six different aggregates were obtained and tested. The exact test method selected for testing these materials was based upon a literature review and discussions with industry experts on the testing of these materials. Based upon the testing of these materials, a default modulus value of 60,000 psi was recommended for use with the new M-E pavement design system.

In order to investigate the second objective, falling weight deflectometer (FWD) testing was conducted on six in-place pavements in which an ADC layer was included. Data from these six pavements were analyzed and recommendations provided for inclusion into ELMOD5. These recommendations were based upon the conditions of the edge drains for the pavements and had input modulus values ranging from 60 ksi to 200 ksi.

17. Key Words Asphalt Drainage Course, ADC, Modulus, Edge Drains, Pavements	18. Distribution Statement Unclassified

19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 42	22. Price

Form DOT F 1700.7 (8-72)

Reproduction of completed page authorized

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CHAPTER 1 - INTRODUCTION

1.1 BACKGROUND

Asphalt Drainage Courses (ADCs) have generally been required under all four-lane flexible pavements in Mississippi. Asphalt drainage courses are designed in Mississippi using No. 57 limestone, sandstone or granite combined with 2.5 percent asphalt binder. Within typical pavement sections, ADCs are placed over a stabilized granular base layer.

Within Mississippi, ADCs are not included when calculating the structural capacity of a pavement. In other words, no structural value (layer coefficient) is assigned to ADCs. The Mississippi Standard Specifications for Road and Bridge Construction (2004) states that ADCs are to be placed at a thickness of 4 inches. Therefore, 4 inches of asphalt binder stabilized aggregates is omitted within the structural capacity of flexible pavements.

Recently, the Mississippi Department of Transportation initiated a large and very important research study with Applied Research Associates, Inc. to implement the new mechanistic-empirical (M-E) design guide. Within this new M-E pavement design guide, all layers of the pavement structure are characterized and included within the pavement design methods. Fundamental properties of each layer are used along with anticipated traffic loadings to estimate the amount of damage that occurs during the life of the pavement. For proper estimation of the amount of damage that occurs within a flexible pavement, ADCs need to be characterized.

Proper characterization of the ADCs will allow a pavement designer to make economically sound decisions on the pavement structure when ADCs are included within the pavement structure. With the proper characterization and incorporation of ADCs within the M-E design guide, the need for various stabilized granular base layers may not be warranted or the thickness requirements may be reduced. If the ADCs do in fact provide structural integrity, then the exclusion of a stabilized layer would reduce the overall cost of new construction or reconstruction.

Since the ADCs were mandated under flexible and rigid pavements for some time, there are many lane miles of roads in Mississippi that include ADCs. Currently, MDOT utilizes a falling weight deflectometer (FWD) to characterize the structural capacity of in-place pavements. Data developed from FWD testing is then input into ELMOD 5 for evaluating layer moduli and overlay designs. Currently, there are not methods to characterize the ADC layers during FWD testing. Input modulus values are needed to properly include ADC layers into ELMOD 5.

Based upon this discussion of ADC layers within MDOT pavement structures, research is needed to properly characterize the stiffness (modulus) of ADC layers. This research is needed in order to include ADC layers within the new M-E design guide as well as characterizing the stiffness of existing ADC layers during FWD testing.

1.2 OBJECTIVES

This research project had two primary objectives which include:

1) Characterize ADC layers for default input values into Mississippi's M-E pavement design system; and
2) Characterize ADC layers in the field to provide inputs for ADC layers in ELMOD5.

CHAPTER 2 – RESEARCH APPROACH

2.1 INTRODUCTION

In order to accomplish the objectives of this study, the research approach involved four tasks. The first task was to conduct a brief literature review. This literature review was conducted to establish methods that have been used to properly characterize asphalt drainage course layers. The second task included a laboratory experiment to characterize ADC layers comprised of various materials. Results from these characterization tests will be useful during both pavement design and FWD “back calculations.” The third task involved developing the proper input values and transfer functions for inclusion of the ADC layers within ELMOD 5 analyses using field data. The final task was to produce a final report that documents the findings, conclusions and recommendations from the research effort. The following sections describe the research approach within each of the four tasks.

Task 1 – Literature Review

During Task 1, a literature review was conducted to evaluate relevant research that has been conducted on the characterization of ADC materials for mechanistic analyses. In addition to reviewing current literature, the researchers also interviewed other experts with intimate knowledge of both the new M-E design guide and ELMOD 5. The characterization method(s) should be applicable to both M-E pavement designs as well as for inclusion within ELMOD 5 analyses and provide stiffness (modulus) values for a range of temperatures and frequencies, if required.

Task 2 – Laboratory Characterization of Typical ADC Materials

Currently, the most common method of characterizing hot mix asphalt is to conduct dynamic modulus testing. However, the current method is an unconfined triaxial test which is most likely not applicable to the very coarse aggregate gradations used in ADCs. Based upon the literature review, the best method of characterizing ADC materials was used during Task 2.

During Task 2, six ADC materials were utilized. Formal mix designs were not conducted as the design of ADC materials is a recipe type design. Within Mississippi, the most common aggregate type used within ADCs is limestone. Therefore, three sources of limestone were identified and used to fabricate ADC samples. With assistance from MDOT, three other sources of materials common in Mississippi were used to fabricate ADC samples including: two sandstone aggregates and one granite aggregate. The final aggregate was a native chert gravel.

Using each of the six ADC materials, samples were prepared in the laboratory and characterized using the method(s) identified during the Task 1 literature review. As stated previously, the selected method(s) must provide stiffness measurements at various temperatures and frequencies, if required, from which master curves can be developed. Therefore, results from this characterization testing will be applicable for both the M-E pavement design method and ELMOD 5 analyses.

Task 3 – Incorporation into ELMOD 5

ELMOD 5 is used by MDOT for calculating stresses and strains at critical points within a pavement section using FWD data. Results from the ELMOD 5 analysis program are used to make recommendations for rehabilitation and maintenance techniques for pavements. Without the proper inputs and transfer functions for ADC layers, analyses with ELMOD 5 will be inaccurate.

Work during Task 3 involved determining the proper inputs for ELMOD 5 using field data obtained with FWD. Results from Task 2 will assist in identifying typical modulus values for inclusion within ELMOD 5. The researchers worked with Dynatest to identify and define other inputs that are required for inclusion of ADC layers.

Task 4 – Final Report

A final report was compiled according to MDOT guidelines and will present a clear and concise summary of the findings, conclusions and recommendations generated from this study.

CHAPTER 3 – LITERATURE REVIEW

3.1 INTRODUCTION

A brief literature review was conducted in order to determine the current state of practice with respect to ADCs. Following are the results of the brief literature review.

3.2 ASPHALT DRAINAGE COURSES

- Alexander, M.L. and R.L. Moore *Structural Value of Asphalt Treated Permeable Base and Open Graded Asphalt, Concrete,* Division of New Technology, Transportation Materials and Research, California Department of Transportation, October 1989.

This study evaluated the structural properties of asphalt treated permeable base (ATPB). California uses a gravel equivalency factor (G_f) instead of a structural number for the design of their pavement structure. In 1989 when this paper was written, California used a G_f of 1.4 for ATPB layers. This is compared to a G_f of 1.0 for aggregate subbase and 2.5 for dense graded HMA. Therefore, it was believed that the ATPB was approximately 40 percent stiffer than an aggregate subbase. Alexander and Moore came to the conclusion that when the ATPB was overlain with dense graded HMA pavement, an ATPB layer is as effective in reducing deflection as an equal thickness of HMA. They do, however, recommend a resilient modulus of 141,000 psi for ATPB as compared to 300,000 psi for HMA.

The specifications for an ATPB layer in California require coarse aggregates to have a minimum Los Angeles Abrasion of 45 percent loss, 90 percent crushed coarse aggregate particles, 1.5 to 2.0 percent asphalt and the following gradation:

Sieve Size (US)	Percent Passing
1 inch	100
¾ inch	90-100
½ inch	35-65
3/8 inch	20-45
No. 4	0-10
No. 8	0-5
No. 200	0-2

- Bejarano, M.O. and J.T. Harvey *Accelerated Pavement Testing of Drained and Undrained Pavements under Wet Base Conditions* Transportation Research Record 1816.

In this paper the researchers loaded pavements that contained an ATPB layer in their structure and found that the ATPB layer increases the fatigue cracking life of the pavement. They also showed evidence that ATPB has a tendency to strip and lose its structural capacity when saturated, which suggests that there may be a great deal of variance in the performance of the drained pavement sections in the field. They also found that once the ATPB failed it was no longer able to prevent a decrease in stiffness of the unbound layers.

- Cook, M. and S. Dykins Treated *Permeable Base Offers Drainage, Stability,* Roads and Bridges, Vol 29 No. 5, Scranton Gillette Communications, Inc., May 1991.

ATPB was used under a Portland Cement Concrete Pavement at Reno Airport. Life cycle cost calculations presented in this paper indicate a 33 percent longer life from rigid pavement that have APTB.

- Diefenderfer, B.K., K. Galal and D.W. Mokarem.(*Effect of Subsurface Drainage on the Structural Capacity of Flexible Pavement.*) Virginia Transportation Research Council 05-R35 June 2005

This paper involved a literature review and investigation of subsurface drainage systems to determine the effectiveness of a drainage layer in Virginia pavements. The literature almost uniformly stated that an improperly maintained drainage system is worse than no drainage system at all. FWD testing was performed on two sites in Virginia that had drained and undrained sections of pavement. The effective structural number calculated from this testing indicated that there was a slight improvement with the drained sections of pavement. The Virginia DOT expects to get 4 additional years of performance from their pavements constructed with subsurface drains; this is a 44 percent increase in the life of the pavement.

- NCHRP Report 583, *Effects of Subsurface Drainage on Pavement Performance, Analysis of the SPS-1 and the SPS-2 Field Sections*,

This paper evaluated the performance of drained and undrained pavements in the Long Term Pavement Performance (LTPP) sites SPS-1 and SPS-2. The authors cite a lack of information that convincingly demonstrates the benefits of subsurface drainage, therefore this project was undertaken. The conclusions indicate that the strongest and the weakest pavement structures evaluated were those that were undrained, while the drained pavements fell in between the undrained pavements. The authors concluded that subsurface drainage systems may still be needed to achieve good performance in some places, but it appears to be less true than it was 20 years ago because of improvements in materials, paving structures and paving practices.

- Harvey, J., B. Tsai, F. Long and D. Hung *CAL/APT Program – Asphalt Treated Permeable Base (ATPB) Laboratory Testing, Performance, Predictions, and Evaluation of the Experience of Caltrans and other Agencies,* Division of New

Technology, Transportation Materials and Research, California Department of Transportation, July 1999.

This study was performed to evaluate the experience that Caltrans and other agencies have had with asphalt treated permeable base (ATPB), as well as perform lab testing and quantify the affects of ATPB on pavement performance. Harvey et al., recommended increasing the G_f from 1.4 to 2.0 for ATPB unless water damage had occured. If water damage occured, it will lead to a reduction in stiffness and a G_f of 1.4 to 1.7 should be used. The paper states that edge and transverse drains must be adequately designed and maintained to prevent prolonged entrapment of water. If lack of maintenance is determined to be a widespread issue, the authors suggest evaluating the ATPB in a saturated condition as well as considering not using ATPB in the pavement structure. The results of laboratory testing showed significant reduction in resilient modulus and increased permanent deformation rates after soaking in water for 10 days at 20° C, as well as a loss of cohesion and binder stripping when subjected to repeated loading while saturated.

To decrease water damage to the ATPB, the authors suggest ensuring that edge and transverse drains for the ATPB layer are adequately designed and maintained to prevent prolonged entrapment of water in the ATPB and decrease the flow of surface water through the HMA surface layer by decreasing its permeability. The author also recommended that a filter layer should be used to prevent clogging of the ATPB layer.

3.3 ELMOD5

The acronym ELMOD stands for Evaluation of Layer Moduli and Overlay Design. ELMOD is a software package supplied by the Dynatest Group to analyze deflection basins created by falling weight deflectometers (FWD) or heavy weight deflectometers (HWD). According to the User's Manual, ELMOD5 performs three major tasks using deflection basin data. First, the program calculates modulus values for each layer within the pavement structure. Next, the calculated modulus values are adjusted to reflect conditions representative of each season specified for the pavement's design period. Finally, the seasonal modulus data is used to determine the expected remaining life within the pavement structure and the needed overlay thickness required to maintain the pavement structure.

Up to five pavement layers can be included within the ELMOD5 analysis. The User's Manual cites several considerations when analyzing these layers. First, the pavement structure should only include one stiff pavement layer. A stiff layer is considered one in which the modulus is at least five times higher than the modulus of the subgrade. If multiple "stiff" layers exist within the structure, the layers should be combined into a single layer. A second consideration is that layer modulus should be decreasing with depth. Overlying layers should be at least twice as stiff, as measured by modulus, as an underlying layer.

At the time of this study, ELMOD5 was used by MDOT for the design of asphalt concrete overlays. Also at the time this study was initiated, MDOT had three typical pavement structures that were being constructed. All three consisted of five pavement layers. Table 1 shows the typical pavement structures. Therefore, the ELMOD5 requirement that a maximum of five pavement layers was met.

Table 1: Typical Pavement Structures in Mississippi

Pavement Layer	Structure 1	Structure 2	Structure 3
1	6.5 to 10.5 in. HMA	6.5 to 10.5 in. HMA	6.5 to 10.5 in. HMA
2	4 in ADC	4 in ADC	4 in. ADC
3	6 to 8 in LFA or CT Granular Soil Base	6 in. Crushed Stone	6 to 9 in. Crushed Stone
4	6 to 8 in. LFA, L or CT Subgrade	6 to 8 in. LFA, L or CT subgrade	Geotextile
5	Untreated Subgrade	Untreated Subgrade	Untreated Subgrade

NOTE:

LFA – Lime/Fly Ash stabilized
L – Lime stabilized
CT – Cement stabilized
HMA – Hot mix asphalt
ADC – Asphalt drainage course

CHAPTER 4 – METHODS AND MATERIALS

4.1 TEST METHODS

A number of tests were conducted on the selected materials to characterize their properties. The following paragraphs describe the test methods to characterize the materials.

Aggregates used in the laboratory study were tested to evaluate gradation, particle angularity, particle shape, toughness, unit weight and absorptive characteristics. Two tests were used to characterize the angularity of the aggregates selected for the laboratory study: percent fractured faces and the uncompacted voids in coarse aggregate. The percent fractured faces test is run by first selecting a representative sample of aggregate having a specified minimum mass and drying to a constant mass. Individual aggregate particles are then visually inspected to determine whether a particle has a fractured face. ASTM D5821 defines a fractured face as "… an angular, rough or broken surface of an aggregate particle created by crushing, by artificial means, or by nature." Further, "… a face will be considered a 'fractured face' only if it has a projected area at least as large as one quarter of the maximum projected area of the particle." Once visually inspected, the aggregate particle was placed within one of two categories: 1) particles with at least one fractured face and 2) particles not meeting the fractured face requirement. The mass of fractured particles was then compared to the total mass of the sample to determine the percent of fractured face particles within the sample.

The second test method used to characterize the angularity of the aggregate materials was the uncompacted void content in coarse aggregates (UVCA). This test method is outlined within AASHTO T326. In addition to particle angularity, this test method also provides an indication of particle shape and surface texture. AASTHO T326 provides three different methods for conducting this test. Method A was utilized during this study. The test property of interest is the voids between uncompacted aggregate particles. Loose aggregates are allowed to flow through an orifice located at the bottom of a specified funnel and fall freely into a calibrated cylinder. The excess material is struck off and the aggregate in the cylinder is weighed. The uncompacted void content is calculated using the mass of aggregate within the calibrated cylinder, the bulk specific gravity of the aggregate and the volume of the cylinder.

Particle shape was measured using ASTM D4791, Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregates. This test method entails measuring the thickness and length of individual aggregate particles. The test method begins by reducing a sample to a minimum test sample mass that is based upon the maximum aggregate size of the gradation. The sample is oven dried and then sieved to determine the gradation. For size fractions with at least 10 percent retained, 100 particles of each fraction are split out for testing. Each particle is then measured to determine length and width. This is conducted with a proportional caliper in which the length (maximum dimension) is used to set the caliper. Then the thickness of the particle is compared to the desired ratio by determining if the particle will pass between the other end of the caliper and a fixed post. For a particle to be considered flat and elongated, the length to thickness ratio must be more than 5:1. Flat and elongated particles are placed in one pile and the particles that are not flat and elongated are placed in a separate pile. The percentage of flat and elongated particles by mass are then calculated based on a weighted average determined from the sample gradation.

The unit weight of the aggregate stockpile materials was determined in accordance with AASHTO T19. This test method covers determining the bulk density of an aggregate in a compacted condition. A specified cylindrical container of known volume and a tamping rod of specified diameter and length are needed for this test. The calibrated cylinder was initially filled to one-third full and the aggregate surface leveled. The layer was rodded with 25 strokes of the tamping rod. Similarly, the cylinder was filled to two-thirds full and again rodded. Finally, the entire cylinder was filled and rodded 25 times. The mass of rodded aggregates within the calibrated cylinder is determined and used to calculate the unit weight of the aggregate.

The toughness of the different aggregates was evaluated using the Los Angeles Abrasion and Impact test (AASHTO T96). This test entails placing a graded sample of aggregate into a large steel drum. Six to twelve steel charges (depending upon the materials gradation) are placed within the drum in addition to the aggregate sample. Both the aggregates and steel spheres are rotated within the drum which subjects the aggregates to impact and abrasion by the steel spheres. Results from this test are a percent loss, which represents the mass percentage loss during the test due to degradation.

Absorption characteristics of the different aggregate materials were established by determining the specific gravities of the materials in accordance with AASHTO T85. This method begins by oven drying the aggregates and obtaining a representative sample. Next, the sample is immersed in water for approximately 15 hours to allow the water to absorb into the aggregates. Following immersion for the 15 hours, excess water is removed from the surface of the particles creating a saturated surface dry condition. Next, the saturated surface dry materials are weighed while submerged in water. Finally, the sample is oven-dried again. The masses at the various stages of the test are then used to calculate different estimates of specific gravities and the absorption characteristics of the material.

Laboratory testing of ADC mixtures involved determining the modulus of mixtures. At the onset of the project, it was unclear whether to conduct modulus testing for ADC mixtures in a manner similar to hot mix asphalt or a granular material. As such, the researchers reached out to experts in the field to determine the type of test needed to measure the laboratory modulus of ADC mixtures. Harold Von Quintus (1) of Applied Research Associates recommended considering ADC as a high quality crushed stone layer. He suggested testing using a resilient modulus test with 10 psi confinement stress and 10 to 15 psi deviator stresses. Test temperatures were suggested as 40, 60, and 80°F.

Using the suggestions of Von Quintus (1), the researchers conducted an analysis utilizing WESLEA v3.0. WESLEA is a mechanistic pavement analysis program that calculates pavement responses to user input loadings and specified pavement structures. The researchers obtained typical pavement structures used within Mississippi in which ADC layers were incorporated within the pavement structure.

Based upon conversations with MDOT, typical pavement sections which include ADC layers were determined. Figure 1 illustrates the range of typical pavement structures encountered that include ADC layers. The hot mix asphalt layer will generally be between 6.5 in and 12 inches. Immediately underlying the hot mix asphalt layer will be 4 in of ADC. ADC layers are generally underlain by either 6 to 8 in of stabilized soil base (typically called stabilized topping materials) or 6 in of crushed stone base material (typically No. 610 limestone). The top 6 in of subgrade materials are also generally stabilized. Finally, natural or fill subgrade materials are the lower most pavement layer.

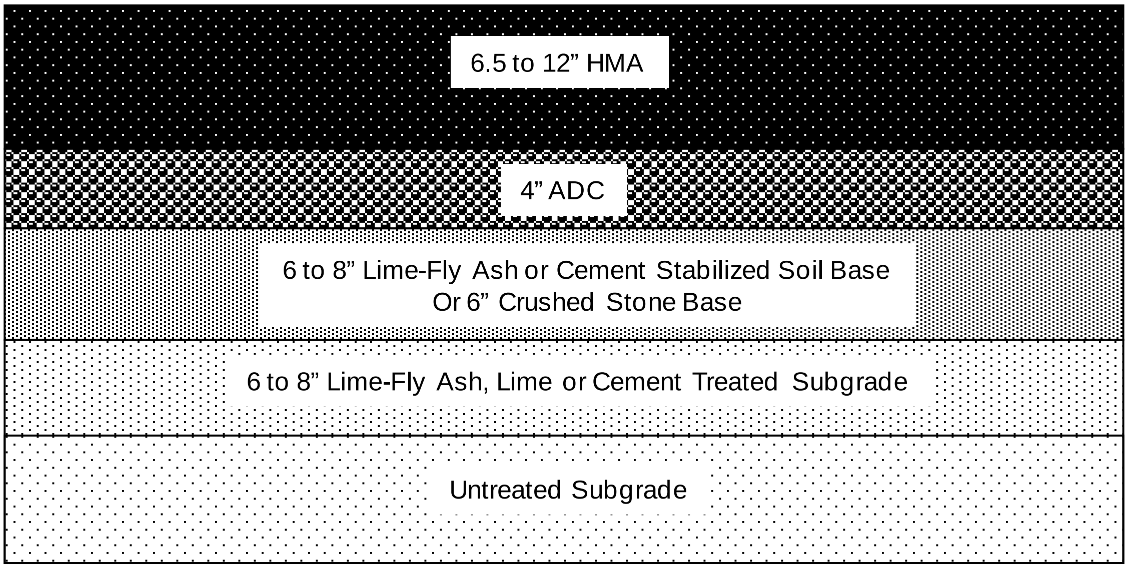


Figure 1: Typical Pavement Structure When ADCs are Utilized

Using WESLEA, the typical pavement responses (stresses and strains) were determined for the ranges of pavement structures that include ADC layers. Typical modulus values for each of the layers shown within Figure 1 were utilized within WESLEA to calculate the pavement responses assuming a tire pressure of 100 psi and wheel load of 5,000 lbs. Based upon this analysis, Table 1 presents the stresses utilized for determining the modulus values of ADC. In order to evaluate the influence of temperature on modulus values, test temperatures of 40, 60 and 80°F were used during testing as suggested by Von Quintus (1).

Table 2: Confining and Deviator Stresses for ADC Modulus Testing

Confining Stress (σ_3), psi	Deviator Stress (σ_1-σ_3), psi
5	2, 5, 10, 15
10	2, 5, 10, 15
15	2, 5, 10, 15

ADC samples used for modulus testing were proportioned in accordance with Section 306 of the MDOT Standard Specifications. A PG 67-22 asphalt binder was added at approximately 2.5±0.4 percent, by total mix mass. The samples were compacted in a 6 inch Superpave gyratory compaction mold using a static load of 10,000 lbs applied by a servo-hydraulic equipment at a temperature of 235±15°F. The mass of mix placed into the mold was calculated based upon the desired air void content and the theoretical maximum specific gravity of the specific ADC mix. The final height of the samples was approximately 8.5 in. Samples were allowed to cool overnight prior to extruding from the gyratory molds. After extrusion, the samples were weighed and measured to determine the air void content via the dimensional analysis approach. Samples were placed within an environmental chamber held at 40°F until testing to prevent the samples from becoming damaged.

This method of compaction was developed based upon a mini-experiment. The mini-experiment involved visiting an ongoing ADC field construction project (Figure 2). After rolling was completed, cores were removed from the ADC layer and the density determined through the dimensional method for bulk specific gravity. The maximum specific gravity was determined on plant produced ADC. In-place air voids ranged from 35 to 40 percent within the cores. The compaction method described above yielded similar air void contents as those produced in the field and, therefore, was utilized to produce laboratory test samples for modulus testing.

Figure 2: ADC Under Construction for Mini-Experiment

4.2 MATERIALS

Table 3 provides the properties of the six aggregate sources obtained to conduct laboratory testing on ADC materials. Of the six aggregates, three sources were limestone materials. The remaining three aggregate types were single sources of granite, sandstone and gravel.

Table 3: Properties of Selected Aggregates for Laboratory Portion of Study

Aggregate Type	LMS1	Granite	Sandstone	Gravel	LMS2	LMS3
UVCA, %	48.2	49.8	47.1	46.7	48.2	49.7
Crushed Faces, %	100	100	100	82	100	100
L.A. Abrasion, %	21.8	30.3	25.1	14.4	25.4	22.7
F&E, 5:1, %	3.5	0.0	2.8	0.0	0.3	0.0
Unit Weight, pcf	99.1	97.1	95.4	99.2	99.2	99.5
Gsb	2.740	2.692	2.565	2.509	2.630	2.655
Gsa	2.767	2.703	2.648	2.605	2.694	2.720
Abs, %	0.40	0.40	1.20	1.48	0.90	0.90
Sieve Size	%Passing	%Passing	%Passing	%Passing	%Passing	%Passing
1.5" (37.5 mm)	100	100	100	100	100	100
1" (25.0 mm)	98.1	93.0	99.5	87.3	98.5	92.7
3/4" (19.0 mm)	81.4	56.0	77.0	58.6	81.2	45.5
1/2" (12.5 mm)	38.3	13.6	38.7	36.9	46.0	7.8
3/8" (9.5 mm)	20.3	4.7	21.1	15.6	28.6	2.8
# 4 (4.75 mm)	2.9	1.0	1.6	1.0	8.6	0.8
# 8 (2.36 mm)	1.2	0.7	0.5	0.8	3.9	0.7

*Picayune,MS gravel gradation was comprised of 40% oversize material and 60% 3/4" Material

Uncompacted voids in coarse aggregate (UVCA) values ranged from a low of 46.7 percent to a high of 49.8 percent. Only the gravel source did not have 100 percent of the coarse aggregate particles with at least one fractured face. The gravel source had 82 percent with at least one fractured face, which is slightly lower than MDOT's minimum of 85 percent. Los Angeles Abrasion loss values ranged from a low of 14.4 percent for the gravel source to a high of 30.3 percent for the granite source. All six of the aggregate sources met MDOT's criteria of a maximum of 40 percent loss after Los Angles Abrasion testing. Flat and elongated particles were all below the MDOT requirement of less than 15 percent. The unit weight of the six materials were relatively similar with the lowest unit weight being 95.4 pcf for the sandstone material and the largest being 99.5 pcf for the LMS3 source. The absorption values for the six aggregate sources ranged from 0.4 to 1.48 percent. The LMS1 and granite sources both had water absorption values of 0.4 percent. The gravel source was the most absorptive.

CHAPTER 5 – FIELD PROJECTS

5.1 INTRODUCTION

While conducting FWD testing of in-place pavement structures to assist in obtaining the proper inputs for ADC layers into ELMOD5, the researchers evaluated six different pavements. These pavements were selected in cooperation with MDOT.

5.2 FIELD SECTIONS

Six in-place pavements were evaluated as part of a field study. Falling-Weight Deflectometer testing was conducted by MDOT on each of these pavement sections. After FWD testing was completed, three cores were obtained within each evaluation section. These pavements ranged in age from 1 year to 4 years. Tables 4 through 9 present specifics about each of the pavements. In most cases, more hot mix asphalt (HMA) was placed than was identified within the pavement design. Only one of the six projects averaged less HMA than the design combined layer thickness. The ADC layers were generally thicker than the 4 in design thickness. Again, only one of the six projects averaged less ADC than the design layer thickness. The thickness of stabilized layers was a mixed bag in that the actual thickness of these layers was sometimes larger than design and sometimes thinner than design.

Table 4: Field Project 1 Details

Layer	Design	Field Sample		
		1	2	3
HMA, in.	7.9	10.0	9.0	8.25
ADC, in.	4.0	4.5	4.25	3.0
Cement Treated Granular, in.	6.0	5.25	3.25	4.75
Chemically Treated Subgrade, in	6.0	3.5	6.0	4.0
Location: Highway 84, Jefferson Davis County, NH-0015-02(15) Age when Tested: 3 years				

Table 5: Field Project 2 Details

Layer	Design	Field Sample		
		1	2	3
HMA, in.	8.0	8.5	8.75	9.25
ADC, in.	4.0	3.25	4.25	4.5
LFA Treated Granular, in.	6.0	6.25	7.5	6.75
Soil-Lime Water Mixing, in	10.0	7.0	5.5	6.0
Location: Highway 25, Winston County SDP-0056-01(076) Age when Tested: 3 years				

Table 6: Field Project 3 Details

Layer	Design	Field Sample		
		1	2	3
HMA, in.	8.0	8.0	8.75	9.25
ADC, in.	4.0	4.0	4.25	4.5
Cement Treated Granular, in.	6.0	5.5	7.5	6.75
Lime Treated Subgrade, in	6.0	7.0	5.5	6.0
Location: Highway 84, Covington County, NH-0015-02(114)				
Age when Tested: 1 year				

Table 7: Field Project 4 Details

Layer	Design	Field Sample		
		1	2	3
HMA, in.	9.25	9.0	10.0	9.5
ADC, in.	4.0	3.5	3.5	3.5
LFA Treated Granular, in.	6.0	6.0	5.5	5.75
Cement Treated Subgrade, in	6.0	7.0	6.0	7.5
Location: Highway 25, Winston County SDP-0056-01(081)				
Age when Tested: 4 years				

Table 8: Field Project 5 Details

Layer	Design	Field Sample		
		1	2	3
HMA, in.	9.1	8.25	8.0	8.5
ADC, in.	4.0	4.75	4.5	4.75
Crushed Stone, in.	6.0	6.0	6.5	6.5
Geotextile, in	---	---	---	---
Location: Highway 67, Harrison County, STP-0064-01(010)				
Age when Tested: 1 year				

Table 9: Field Project 6 Details

Layer	Design	Field Sample		
		1	2	3
HMA, in.	8.6	8.5	9.0	10.0
ADC, in.	4.0	4.5	5.5	4.25
Crushed Stone, in.	6.0	6.5	6.0	4.0
Geotextile, in	---	---	---	---
Location: Highway 67, Harrison County, STP-0064-00(022)				
Age when Tested: 1 year				

Asphalt Drainage Course samples varied widely in apparent quality when they were extracted from the pavement, most likely as a result of the age of the pavements. The newer pavements appeared more intact and more of the ADC lift was preserved. The newer pavements seemed to have more asphalt binder remaining on the aggregate, though this could have been a function of the asphalt stripping from the aggregate over time in some of the older pavements. The older pavements tended to exhibit more moisture damage and less adhesion between the aggregate particles. Appendix A provides pictures of the recovered cores.

Core samples were taken with a six inch diameter core bit. The drilling process through the ADC layer proceeded similarly to cutting through typical dense graded asphalt. However, extraction of the entire ADC proved more problematic in some instances. Due to the designed open graded nature of the ADC, it was typical for at least several particles of ADC to dislodge from the bottom of the sample during extraction. This issue was confounded because the ADC layer is generally placed on a granular subbase material. Because there is no bonding between the two layers, once the core was cut it would break free in the corehole and actually spin in the hole for a time before the coring process was completed. In these instances, the result was a conical shaped core bottom. The design ADC thickness was 4 inches for all projects and the actual layer thickness measured was between 3 and 5.5 inches. However, the extracted thickness was often less than the layer measured in the core hole.

There were no incidences of samples that were stripped of asphalt binder to the point that the aggregate was completely unbound. The three samples taken from each of projects 2, 3, 5 and 6 showed either no stripping or very little stripping. Projects 1 and 4 were sampled in conjunction with another research study so many more samples were collected. Approximately 25 percent of the samples in Project 4 showed low to moderate severity stripping. In Project 1, almost all of the samples showed some degree of stripping and approximately 50 percent of the samples were classified as moderate severity stripping and 50 percent of the samples were classified as low severity stripping.

From a practical standpoint, the qualitative evaluation of the ADC cores is interesting. For the three pavements that were approximately 1 year in age, no stripping or other issues were observed in the ADC layer. Two of the pavements were approximately 3 years in age. On one of these projects (Project 2), either no stripping or very minor stripping was observed. Project 1, which was also 3 years in age, had low to moderate stripping. Likewise, the only project that was approximately 4 years in age (Project 4) had low to moderate stripping. Though this was a very limited data set, it appears that stripping may begin approximately 3 to 4 years into the life of an ADC layer. This would suggest a lack of pavement drainage (within the pavement system).

CHAPTER 6 – LABORATORY CHARACTERIZATION OF ADCs

6.1 INTRODUCTION

Asphalt Drainage Course materials were prepared in the laboratory and tested to determine laboratory modulus values. As stated within Chapter 4, the testing conditions were based upon acknowledged experts in the field of mechanistic testing of pavement materials as well as an analysis of stresses and strains that may be encountered by ADC layers under typical Mississippi pavements. Table 2 presented the stress states used during the modulus testing. Test temperatures of 40, 60 and 80°F were used during testing.

6.2 TEST SPECIMENS

Specimens used during modulus testing were prepared as described in Chapter 4. Three replicates were fabricated for each of the six materials used in this study to provide three modulus test results. Table 10 presents the properties of the samples used for modulus testing. As shown within Table 10, the air void contents of the samples were all relatively similar ranging from a low of 37.6 to a high of 41.9 percent. This range of air voids closely matches the in-place air voids of the constructed ADC layer included within the mini-experiment described in Chapter 4.

Table 10: Properties of Samples Prepared for Modulus Testing

Material	Measure	Replicate		
		1	2	3
LMS1	Diameter, in.	5.87	5.86	5.88
	Height, in.	8.64	8.81	8.77
	Gmb	1.616	1.589	1.583
	Gmm	2.667	2.667	2.667
	VTM, %	39.4	40.4	40.7
Granite	Diameter, in.	5.92	5.86	5.86
	Height, in.	8.90	8.84	8.80
	Gmb	1.536	1.580	1.588
	Gmm	2.645	2.645	2.645
	VTM, %	41.9	40.2	40.0
Sandstone	Diameter, in.	5.92	5.92	5.92
	Height, in.	8.99	9.10	9.10
	Gmb	1.527	1.506	1.523
	Gmm	2.534	2.534	2.534
	VTM, %	39.7	40.6	39.9
Gravel	Diameter, in.	5.81	5.84	5.84
	Height, in.	9.24	9.23	9.30
	Gmb	1.547	1.519	1.514
	Gmm	2.478	2.478	2.478
	VTM, %	37.6	38.7	38.9
LMS2	Diameter, in.	5.92	5.92	5.92
	Height, in.	8.80	8.76	8.78
	Gmb	1.549	1.564	1.548
	Gmm	2.566	2.566	2.566
	VTM, %	39.6	39.0	39.7
LMS3	Diameter, in.	5.90	5.86	5.91
	Height, in.	8.83	8.83	8.82
	Gmb	1.562	1.582	1.560
	Gmm	2.606	2.606	2.606
	VTM, %	40.1	39.3	40.1

Gmb – Bulk Specific Gravity, Gmm – Theoretical Maximum Specific Gravity
VTM – Voids in Total Mix

Tables 11 through 16 present results of modulus testing for each of the six materials utilized in this study. Test results are provided for modulus values at 40, 60 and 80 °F. The data suggests that the modulus values for the limestone materials are slightly higher than the other stone materials. ADCs fabricated with the gravel aggregate had the lowest modulus values.

Table 11: Laboratory Resilient Modulus Test Results for LMS1

σ1	σ2	σ3	MR @ 40°F	MR @ 60°F	MR @ 80°F
psi	psi	psi	psi	psi	psi
6.9	4.8	4.8	164,483	125,096	83,555
11.8	9.7	9.7	185,282	128,682	90,902
16.7	14.6	14.6	202,211	133,601	96,568
10.0	4.8	4.8	152,792	121,347	79,271
14.9	9.7	9.7	166,844	128,321	82,976
19.8	14.6	14.6	175,758	129,457	86,820
24.8	14.6	14.6	157,542	128,504	84,993
19.8	9.7	9.7	154,062	125,096	80,902
14.9	4.8	4.8	146,092	120,588	77,280
29.8	14.6	14.6	155,626	130,075	83,731
24.9	9.7	9.7	153,802	124,696	80,124
19.9	4.8	4.8	148,284	122,748	76,242

Table 12: Laboratory Resilient Modulus Test Results for Granite

σ1	σ2	σ3	MR @ 40°F	MR @ 60°F	MR @ 80°F
psi	psi	psi	psi	psi	psi
6.9	4.8	4.8	143,894	107,803	80,657
11.8	9.6	9.6	147,301	106,627	79,801
16.6	14.5	14.5	148,778	111,096	81,866
9.9	4.7	4.7	131,281	102,428	74,376
14.7	9.6	9.6	139,119	104,269	78,459
19.6	14.4	14.4	142,152	105,741	79,440
24.6	14.5	14.5	141,986	104,711	73,971
19.6	9.5	9.5	139,432	102,909	71,542
14.9	4.7	4.7	134,366	100,829	69,128
29.6	14.5	14.5	144,919	108,322	70,953
24.7	9.6	9.6	142,564	106,525	67,869
19.8	4.7	4.7	139,552	104,827	65,147

Table 13: Laboratory Modulus Test Results for Sandstonte

σ1 psi	σ2 psi	σ3 psi	MR @ 40°F psi	MR @ 60°F psi	MR @ 80°F psi
6.9	4.8	4.8	108,409	107,887	70,274
11.8	9.7	9.7	117,496	109,820	73,011
16.7	14.6	14.6	124,594	112,587	74,661
9.9	4.8	4.8	119,207	102,924	67,272
14.8	9.7	9.7	124,456	107,413	69,166
19.7	14.6	14.6	128,869	110,179	70,981
24.7	14.6	14.6	131,518	105,223	71,743
19.8	9.7	9.7	129,927	102,938	69,433
14.9	4.8	4.8	125,860	99,710	67,036
29.7	14.6	14.6	135,460	107,411	71,535
24.8	9.7	9.7	134,394	105,607	68,980
19.9	4.8	4.8	131,195	102,651	66,899

Table 14: Laboratory Modulus Test Results for Gravel

σ1 psi	σ2 psi	σ3 psi	MR @ 40°F psi	MR @ 60°F psi	MR @ 80°F psi
6.9	4.8	4.8	97,925	94,608	57,843
11.8	9.7	9.7	101,819	95,195	57,842
16.7	14.6	14.6	102,985	98,464	59,640
9.9	4.8	4.8	105,947	97,305	55,878
14.8	9.7	9.7	109,743	100,234	56,835
19.7	14.6	14.6	111,599	101,413	57,148
24.7	14.6	14.6	116,764	104,816	56,239
19.9	9.7	9.7	115,914	103,183	54,821
14.9	4.8	4.8	113,543	99,800	53,251
29.7	14.6	14.6	122,613	107,234	54,753
24.7	9.7	9.7	121,854	106,343	51,423
19.9	4.8	4.8	120,587	104,463	54,098

Table 15: Laboratory Modulus Test Results for LMS2

σ1 psi	σ2 psi	σ3 psi	MR @ 40°F psi	MR @ 60°F psi	MR @ 80°F psi
6.9	4.7	4.7	214,281	177,238	100,712
11.8	9.6	9.6	209,317	181,382	112,559
16.7	14.5	14.5	236,543	191,977	121,492
10.0	4.8	4.8	225,831	160,696	93,727
15.0	9.6	9.6	243,183	171,533	100,944
19.9	14.6	14.6	255,097	178,067	106,658
24.8	14.5	14.5	234,780	176,949	108,419
19.8	9.6	9.6	224,948	166,553	100,990
14.9	4.7	4.7	203,951	156,951	94,368
29.8	14.5	14.5	226,065	170,878	106,169
24.9	9.6	9.6	212,017	164,273	101,150
20.0	4.7	4.7	196,615	152,368	93,967

Table 16: Laboratory Modulus Test Results for LMS3

σ1 psi	σ2 psi	σ3 psi	MR @ 40°F psi	MR @ 60°F psi	MR @ 80°F psi
7.0	4.8	4.8	196,235	161,585	69,194
11.9	9.8	9.8	228,786	163,136	69,573
16.9	14.7	14.7	228,844	174,465	71,532
10.0	4.8	4.8	197,766	145,157	77,098
14.9	9.8	9.8	202,144	157,041	85,266
19.8	14.7	14.7	209,593	164,741	93,128
24.9	14.7	14.7	207,563	161,381	90,780
19.9	9.7	9.7	202,108	155,791	85,505
15.2	4.8	4.8	185,090	146,959	77,385
29.9	14.7	14.7	200,354	161,465	88,883
25.1	9.7	9.7	196,941	158,679	82,941
20.4	4.8	4.8	189,701	151,964	74,165

Figures 3 through 8 graphically present the results of modulus testing for the ADC mixes comprised of the six materials. Within the these figures, modulus values are plotted versus bulk stress. These figures indicate some effect of temperature on the measured modulus value. This indicates that in a laboratory environment the visco-elastic properties of the asphalt binder slightly influences the modulus values of the ADC mixes. Also of interest within the figures is that the modulus values do not vary significantly with a change in bulk stress. Because the modulus did not vary significantly with changes in bulk stress and there was only a minor effect of temperature on modulus values, it appears that Von Quintus'(1) suggestion that the ADC layer be considered a high quality crushed stone layer was valid. As such, a typical modulus value was developed for ADC layers that can be utilized during pavement design. Modulus values from the ADC fabricated from the gravel aggregate were not included in developing this typical value.

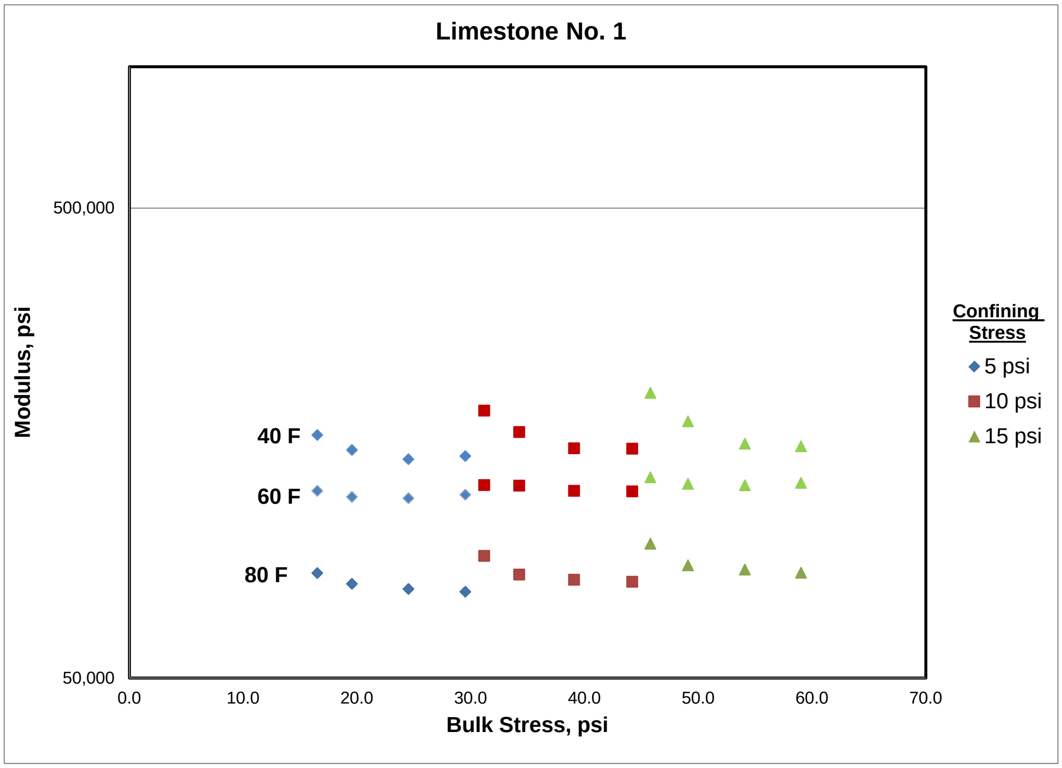


Figure 3: Laboratory Modulus Test Results for LMS1

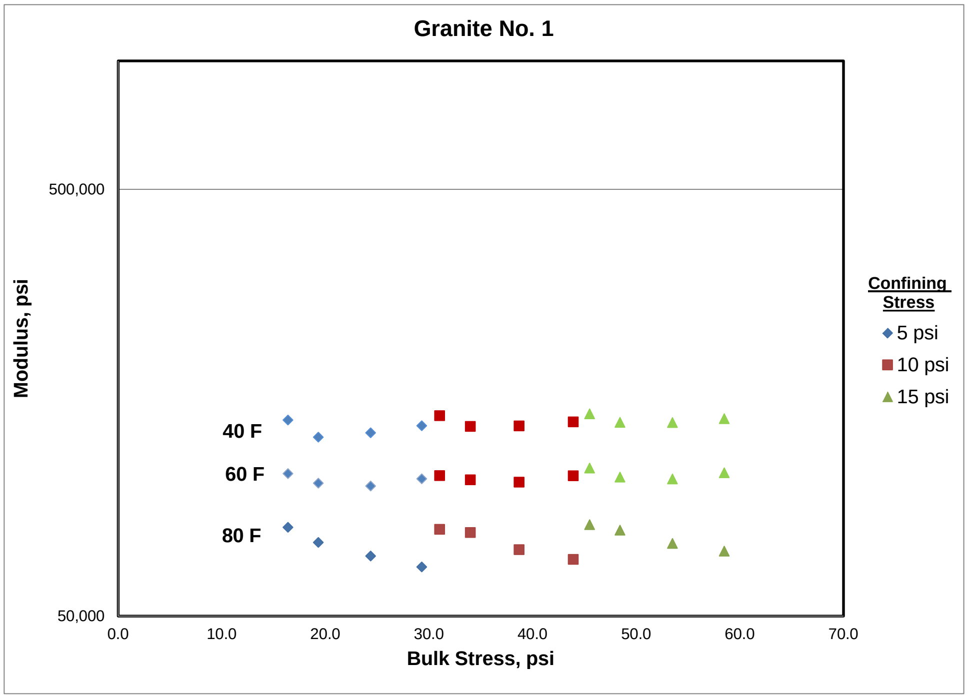


Figure 4: Laboratory Modulus Test Results for Granite

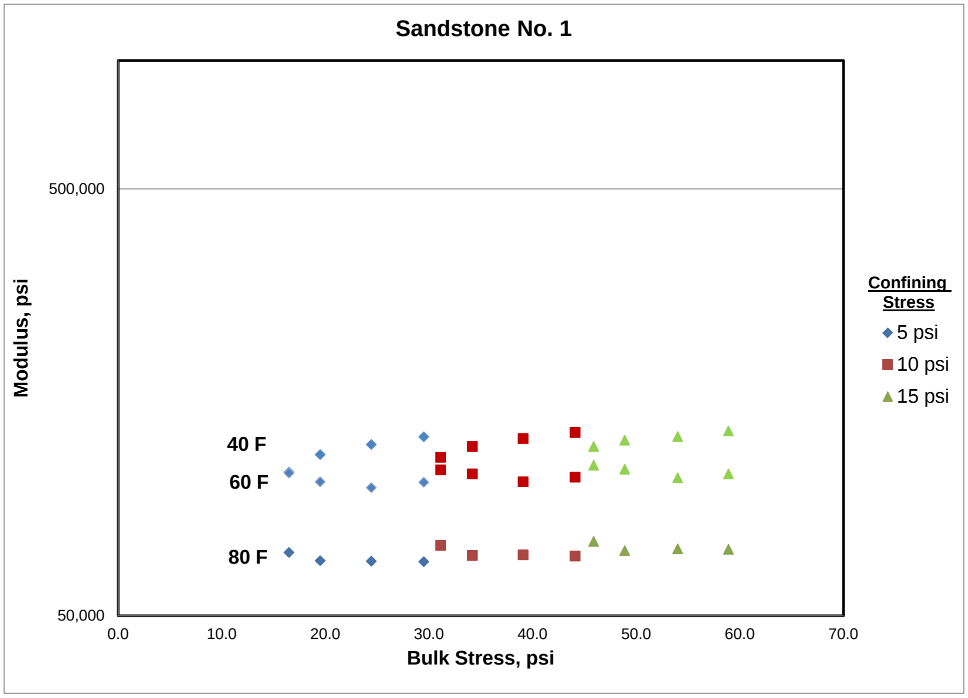


Figure 5: Laboratory Modulus Test Results for Granite

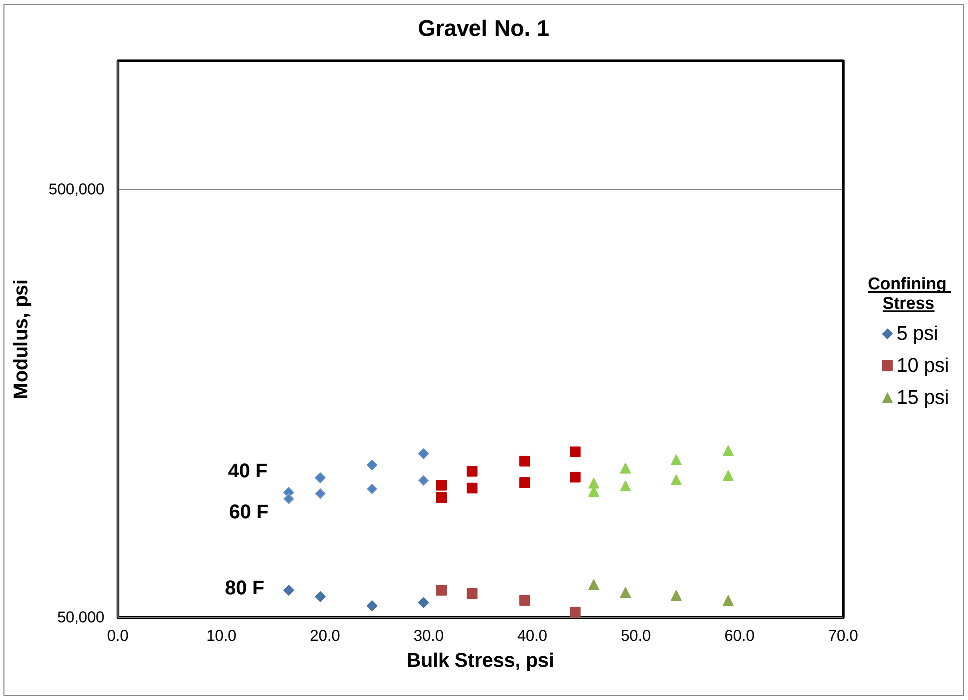


Figure 6: Laboratory Modulus Test Results for Gravel

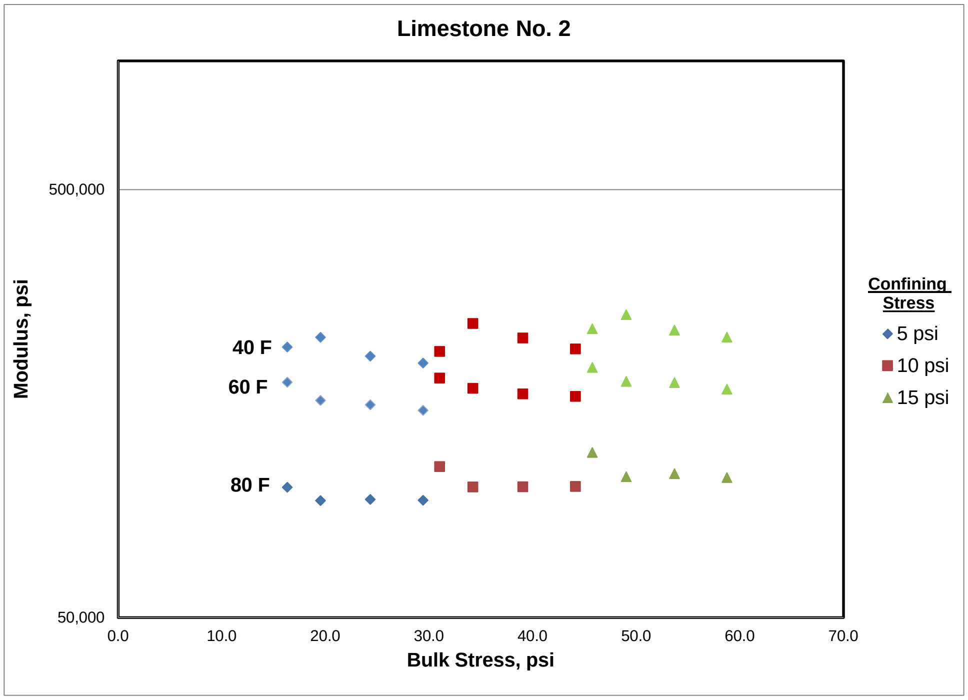


Figure 7: Laboratory Modulus Test Results for LMS2

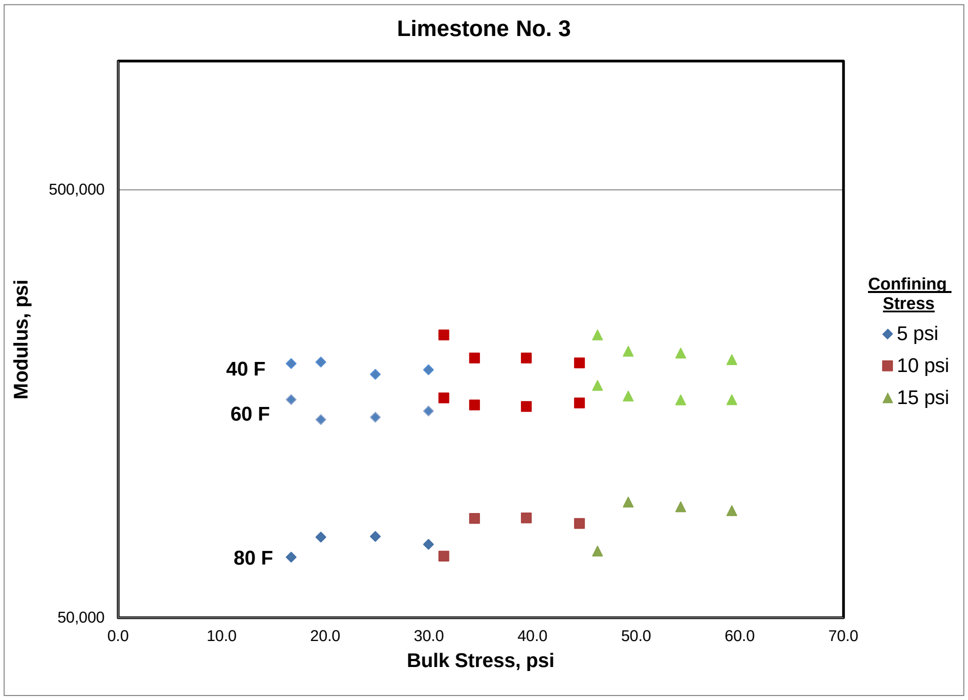


Figure 8: Laboratory Modulus Test Results for LMS3

If the assumption that ADC is a high quality stone material is true, several considerations are required when developing a typical modulus value. First, the evaluation of the cores obtained during the field study indicated that the ADC layers begin to degrade after 3 to 4 years in-service. This project, nor the literature, suggests a rate of deterioration in ADC layers. Therefore, this consideration must be taken into account when developing a typical modulus value for ADC.

Secondly, the modulus values presented in this chapter were based upon ideal, laboratory testing of ADC samples. Unfortunately, the condition of the cores obtained from the field projects did not allow for the testing of in-service ADC materials. However, the relatively thin layer thickness of the ADC layers may have precluded the ability of accurate modulus values. Consideration must be given to the differences between ideal, laboratory samples and ADC material after construction.

Thirdly, the MDOT Standard Specifications allow limestone, sandstone, and granite for inclusion within ADC materials. Though the differences weren't large, the limestone materials did produce ADC with slightly higher modulus values. Consideration should not include the modulus results for the gravel ADC mixtures.

Finally, the test results did indicate a slight influence of temperature on the modulus values. Obviously, modulus values of any material that includes visco-elastic asphalt binder will decrease as temperature increases. Therefore, from a stiffness standpoint, warmer temperatures will be more critical.

The above mentioned considerations were used to develop a typical modulus value for ADC materials. Engineering judgment was the primary factor in selecting a typical modulus value of 60,000 psi. This value can be compared to the estimated resilient modulus of a good quality Limestone 825-B developed for MDOT (2) of 34,000 psi.

CHAPTER 7 – INCORPORATION OF ADC INTO ELMOD

7.1 SUMMARY OF FWD TESTING AND ANALYSIS AND STRCUTRUAL DESIGN RECOMMENDATIONS

Generally, most state DOTs that use drainable bases install edge drains. Edge drains may be installed either during original construction or as a “retrofit”. It has been the experience of many of these states that if edge drains are installed and working properly (ie., draining water from under the pavement, as designed), drainable bases can, and do, function quite well and may even contribute to the overall structural strength of the pavement.

It has been the experience of a number of DOTs that edge drains are not generally maintained (monitored and unclogged, as needed). Therefore, it has been found that drainable bases may in fact do more harm than good due to the “bathtub” effect where sub-pavement water cannot drain away into the ditches and surrounding landscape as designed. As pointed out in Chapter 5, the ADC layers within Mississippi pavements that are older than 3 to 4 years do appear to be experiencing stripping.

It is difficult to accurately calculate the modulus of elasticity of any relatively thin layer that is “sandwiched” between a thicker surface course and a bound subbase or treated subgrade. In the case of Mississippi’s use of ADC’s, the thickness of this layer is generally in the 3.5 to 5.5 in. range, while the asphalt concrete surface course is generally twice as thick or more.

Beneath the ADC layer, treated materials are generally used, whether these are cement-, lime-, and/or fly ash-treated, or in some instances a limestone base course. The latter material, in fact, has some binding qualities, even though it isn’t treated with external cementitious materials in the strictest sense of the word.

With such a “sandwich” construction, looking for the modulus of an intermediate layer with FWD data is a lot more of an art than a science. Nevertheless, the ELMOD program was utilized using two different methodologies (input assumptions), with the overall conclusions shown in Table 17. Method 1 involved the use of a “seeding” ratio between the cementitious subgrade or subbase layers and the subgrade (=5 for limestone base; =7.5 for treated subgrades). Method 2 involved the use of a seeding ratio between the ADC and the limestone base or treated subgrades. For this seeding ratio, the Dorman and Metcalf equation was used to initiate the backcalculation process. In this case, the ratio was allowed to vary in order to achieve the best-fit solution using ELMOD^{TM}. The second methodology was further simplified by averaging the layer thicknesses at each test site.

In Table 17, the results in terms of the ADC modulus (ksi) are shown in the lefthand columns for each of the two methods. The method that resulted in the lowest (-

best convergence) root mean square value is then shown, with either a "+", "-" or "=" sign to indicate whether the most likely result will be somewhat larger, somewhat smaller, or is in fact very close to the value shown using either of the two backcalculation approaches used in the ELMOD analyses.

At the time of testing, the effectiveness of the edge drains was not known. The values shown, once again, are very approximate and should only be considered relative to one-another, not as "absolute" values of the moduli of the ADC materials used in the State of Mississippi.

Table 17: Summary of FWD Test and Analysis Results for ADC Layers

Method 1	Method 2	Approx. ADC (ksi) @ App. °F	
Highway 84		Jeff Davis County NH-0015-02(115)	
89	284	89 (+)	82
93	223	93 (+)	82
361	294	361 (-)	83
Highway 25		Winston County SDP-0056-01(076)	
155	447	447 (-)	80
878	1503	878 (+)	80
80	384	384 (-)	80
Highway 84		Covington County NH-0015-02(114)	
200	351	200 (+)	97
459	836	459 (+)	99
31	108	108 (-)	100
Highway 25		Winston County SDP-0056-01(081)	
276	709	709 (-)	90
467	922	467 (+)	89
451	1045	451 (+)	90
Highway 67		Harrison County STP-0064-01(010) Southbound, 4 miles south of 605	
194	199	197 (=)	81
185	165	175 (=)	82
87	132	110 (=)	82
Highway 67		Harrison County STP-0064-00(022) Northbound, 4.2 miles north of 605	
138	87	112 (=)	85
57	72	65 (=)	84
276	97	97 (+)	85

As can be seen, the modulus results shown in the above table are highly variable. Also as indicated, it is not known whether the edge drainage systems were working properly at the time of the test.

Still, there are few results that are so low (or in the one case too high) that there must be either an error in the calculations or sufficient degradation of the ADC to suspect that the edge drains were in fact not working properly. These "outlier" test points are shown in Table 18.

Table 18: Suspect ADC Outliers from Table 17

Winston County – Highway 25	Point 2	>878 ksi
Covington County – Highway 84	Point 3	<108 ksi
Harrison County – Highway 67	Point 2 (NB)	≈65 ksi

For the remaining areas, while the test results do not necessarily indicate whether or not proper drainage was taking place at the time of the test, a range of suggested ADC design moduli and the corresponding structural design coefficients (a_2) from the 1993 AASHTO Design Guide can be suggested, as shown in Table 19. When using Table 19 at any level, it is assumed that proper drainage is taking place and that edge drains are, at a minimum, routinely maintained.

Table 19: Suggested Design Moduli and a_2 – values for ADC (edge drain maintained)

Design Assumptions	Design Modulus	Structural Design Coefficient (a2)
Ultra Conservative (assuming no maintenance to edge drains)	60 ksi	0.10
Conservative Design (assuming edge drains are routinely maintained)	100 ksi	0.12
Normal Design (assuming edge drains are well maintained)	150 ksi	0.18
Optimistic Design (assuming perfect drainage is maintained throughout entire life of the project)	200 ksi	0.22

CHAPTER 8 – CONCLUSIONS AND RECOMMENDATIONS

8.1 INTRODUCTION

This research had two primary objectives. First, the research was to characterize ADC layers for default input values into Mississippi's M-E pavement design system. Secondly, the research was to characterize ADC layers in the field to provide inputs for ADC layers in ELMOD5. The following sections provide conclusions and recommendations from the research conducted to accomplish these two objectives.

8.2 CONCLUSIONS

The following are conclusions based upon the research conducted to accomplish the project objectives.

- In-place air void contents of ADCs are approximately 40 percent.
- The method developed for preparing ADC modulus test samples successfully produced samples near 40 percent air voids.
- Stripping was not observed in ADC layers that had been in-service for less than 3 years.
- Low to moderate stripping was generally observed in ADC layers that had been in service for 3 or more years.
- Laboratory modulus values for ADC mixes comprised of limestone aggregates were slightly higher than for ADC mixes comprised of sandstone, granite and gravel aggregates.
- Laboratory modulus values of ADC mixes comprised of gravel mixes were the lowest.
- Temperature did have an effect on the laboratory measured modulus values of ADC. However, the effect of temperature was not pronounced.
- Bulk stress did not have a significant effect on laboratory measured values of ADC.
- ADC layers are considered similar to a high quality stone layer.
- Modulus values for ADC determined from backcalculation of FWD data were highly variable.

8.3 RECOMMENDATIONS

The following are recommendations based upon the research conducted to accomplish the project objectives.

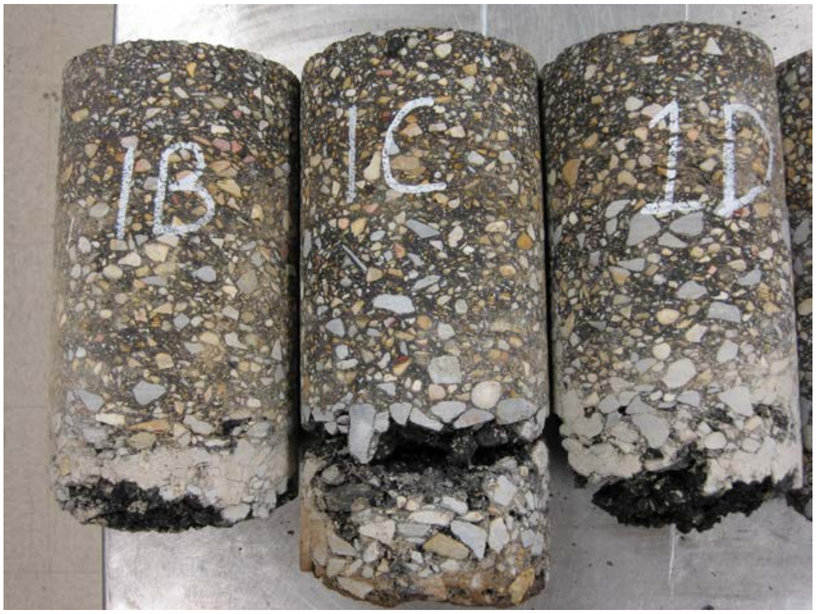
- A default modulus value of 60,000 psi is recommended for use in designing pavements.
- Table 19 provides the recommended modulus values for input into ELMOD5.

REFERENCES

1. Von Quintus, H. Personal Communication. March 3, 2007.

2. James, R.S., L.A. Cooley, Jr., R.C. Ahlrich. “Summary of Lessons Learned from the MDOT MEPDG Materials Library Study.” Prepared for the Mississippi Department of Transportation. FHWA/MS-DOT-RD-09-224. June 2010.

APPENDIX A



Photograph No. 1 - Project 1

Photograph No. 2 – Project 2

Structural Characterization of Asphalt Drainage Course Layers
MDOT State Study 181

Photograph No. 3 - Project 3

Photograph No. 4 – Project 4

Photograph No. 5 - Project 5

No picture available

Photograph No. 6 – Project 6