INVESTIGATION OF PERMEABLE ASPHALT

TREATED BASES IN ALABAMA

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INVESTIGATION OF PERMEABLE ASPHALT

TREATED BASE IN ALABAMA

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INVESTIGATION OF PERMEABLE ASPHALT
TREATED BASE IN ALABAMA

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THESIS ABSTRACT

INVESTIGATION OF PERMEABLE ASPHALT

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The Alabama Department of Transportation currently uses a permeable asphalt treated base (PATB) course layer consisting of an AASHTO number 57 aggregate gradation with 2.0% to 2.5% asphalt content. The current mixture design employed by ALDOT has been subject of many concerns, namely stability during construction. As a result, the fourth Division has decided to use only polymer modified PG 76 – 22 binder in all of its PATB mixes.

An in depth laboratory procedure was devised in order to handle, compact, and test PATB samples, since there was no current procedure available. Field cores and plant mixed PATB were obtained from two different sites within ALDOT's fourth Division.

The cores were tested for air voids, permeability and stability. The plant mix was then compacted to match the field cores in terms of air voids. It was also found that the laboratory produced pills had similar permeability and stability characteristics as the cores obtained from the field. This consequently verified that the procedure outlined to handle, compact, and test PATB samples was suitable.

The second phase of the research project investigated alternative mixes for the use of PATB. Number 57 and number 78 gradations consisting of crushed limestone were mixed with 2.0% asphalt. The binder tested consisted of PG 67 – 22 binder and polymer modified 76 – 22 binder. Samples were compacted to a target air void content of 30%. The PATB samples consisting of the number 78 aggregates required less compaction effort than the PATB with number 57 gradation. Furthermore, the PATB with number 78 aggregate had better stability than the PATB with number 57 aggregate and had permeability values above 1,000 ft/day. Based on these results, it is recommended that a field study is performed to determine how PATB consisting of number 78 aggregate performs under traffic and in field conditions.

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CHAPTER 1

INTRODUCTION

Background

The role of permeable asphalt treated base courses is to provide sufficient permeability to drain pavement structures of intruding water and to provide a construction platform that is more stable than unstabilized open graded granular base courses. In general, permeable asphalt treated base courses (PATB) are lean asphalt mixes with asphalt contents ranging from 2 percent to 3 percent by weight. The open graded aggregate contains little to no fine material. The open graded aggregate gradations provide sufficient permeability that drains a pavement more quickly than a dense graded granular base. Stabilizing the open graded aggregate with 2 to 3 percent asphalt provides better stability for construction traffic than unstabilized open graded aggregate base courses while maintaining required peremability.

The Alabama Department of Transportation (ALDOT) currently uses PATB in flexible pavement to obtain both of these favorable results. The current mix specification requires an open graded number 57 crushed aggregate with 2 to 2.5 percent asphalt cement. The specification does not have any requirements for voids in the mineral aggregate (VMA), density, air voids, or stability. Dried aggregate is mixed with liquid asphalt binder at a temperature of 250°F. The PATB is placed in lifts with maximum thickness of 4 inches. Compaction commences once the PATB layer has cooled to

150°F. Generally, one to three passes with a static compactor is all the compaction necessary to seat the PATB adequately. Figure 1.1 shows the construction of PATB on US 280 near Camp Hill, Alabama.

Figure 1.1. PATB Paving on US 280 near Camp Hill, Alabama.

Correspondence with ALDOT Division engineers from Divisions 1, 2, and 4 provided background concerning the history and their opinions regarding the use of PATB. The amount of PATB placed as of Spring 2006, varied from 14 lane miles in Division 1, to 113 lane miles in Division 4, to 164 lanes miles in Division 2.

Even though the amount of PATB placed varied considerably, two opinions were common amongst the three division engineers. The first opinion was that the PATB seems like a viable solution to remove water from the pavements. The other opinion was that stability was inadequate. Division 1 recalled a project in which rutting in the PATB layer was a major problem during construction. This rutting created depressions and unacceptable surface roughness. Division 2 stated that slope issues and the material spreading out wider than needed are issues during construction. These issues can be attributed to the fact that current PATB specifications make it difficult to handle and compact in the field. Division 4 recounted issues of rutting both during and after construction of the PATB layers. These observations led to use of polymer modified, PG 76-22, asphalt binder in Division 4. However, modified binder significantly increases material costs.

These aforementioned issues concerning PATB as currently specified have created some interest to investigate the possibility of changing PATB specifications. The alterations are intended to minimize or eliminate these issues observed both during and after construction, and may be accomplished by changing the composition of PATB.

Objectives

There were two main objectives of this research. The first objective was to devise a laboratory methodology to handle, compact and test PATB material for engineering characteristics, mainly permeability and stability. The second objective of this research was to modify, as required, PATB specifications. This essentially entails testing trial mixes of open graded hot mixed asphalt materials and determining if they would provide adequate stability for construction of subsequent pavement layers, while simultaneously, maintaining desirable permeability characteristics.

Scope

A literature review was conducted to investigate two main topics. The first portion of the literature review was performed to fully understand the purpose and function of permeable asphalt treated base courses. This included gathering results and discussions of previously conducted research concerning the issues, both favorable and unfavorable, related to using PATB in a pavement. It also yielded direction, based on others' research methodology, on how to handle, compact and adequately test PATB material in the laboratory.

The second portion of the literature review consisted of obtaining specifications on the construction of PATB from other Southeastern state departments of transportation. This enabled comparison and contrast of ALDOT's current PATB specification against those around the region.

PATB material was collected from two construction sites within ALDOT's fourth division. Cores from the compacted PATB course, prior to lay down of subsequent layers, as well as plant mixed PATB material from the hopper of the paver were obtained. This enabled quantification of the characteristics of PATB as currently used and development of a laboratory methodology to produce PATB samples of similar characteristics in terms of percent air voids, stability, and permeability.

Once an adequate laboratory procedure was devised, aggregate and binder were mixed, compacted and tested. At this point, open graded hot mixed asphalt with variable properties was produced and tested. These materials were then evaluated to determine of they has sufficient stability to carry construction traffic while also providing sufficient permeability to drain water from flexible pavements.

CHAPTER 2

LITERATURE REVIEW

Purpose of Permeable Treated Asphalt Base Courses

It is widely known that poor drainage of a pavement can cause structural damage that can eventually lead to reduced driver safety. Rutting can cause water to pond on the pavement surface. The following excerpts from various literature explain some of the issues that arise when drainage is inadequate.

> "Water entrapped in the pavement structure not only weakens pavements and subgrades, but also generates high hydrodynamic pressures which pump out the fine materials under the pavement and result in loss of support." (Huang, 2004)

> "Entrapped water yields continuous contact with the asphalt mixture. This causes stripping of the asphalt binder from the aggregate." (Huang 2004)

> "Without adequate drainage design, variations in the permeability of the base may cause build up of hydrostatic pressures sufficient to lift the pavement from the base and lead to cracking or complete destruction of the pavement." (Lovering and Cedergren, 1962)

> "In flexible pavements, the continued presence of moisture in conjunction with heavy vehicle loads may result in stripping of asphalt from aggregate, potholes, and alligator cracking, as well as a significant reduction in unbound material strength." (Zaghoul et al., 2004)

Water may gain entry into the pavement by a number of methods. Typically water enters by infiltrating through cracks or joints on the surface. It can also permeate through the pavement surface. Water may enter the structure from below. High water tables may permit groundwater to enter from the subgrade in low lying or deep cut pavements. All in all, the unfavorable effects that water within the pavement structure include the following (Huang, 2004):

- The constant presence of water will result in the reduction of strength of unbound materials as well as the underlying subgrade soils.
- Entrapped water will exert elevated hydrodynamic pressures that arise from the repeated loading of traffic. These pressures will pump fine materials in the base and subgrade layers resulting in loss of support.
- Continued contact between the asphalt mixture and water will result in the gradual loss of adhesive bond between the aggregate particles and the asphalt binder. This occurrence is defined as stripping. This separation of the aggregate and the binder yields a reduction in strength of the layer and may result in failure of the entire pavement structure.

Lindy and Elsayed (1995) also noted that excessive pore pressures accumulated due to the presence of entrapped water and wheel impacts caused excessive deflection, cracking and raveling of asphalt mixtures. Raveling, as defined by Shahin (2005), is the wearing away of the pavement surface caused by the dislodging of aggregate particles and loss of asphalt binder. When this distress occurs, the pavement essentially disintegrates resulting in undesired roughness and lack of adequate strength. Furthermore, Zanghloul et al. (2004) cite that excessive moisture along with vehicle loads cause alligator cracking and potholes. All in all, the presence of entrapped water in a flexible pavement can cause a number of distresses that essential reduce the durability and serviceability of a pavement structure.

Economic Analysis

Premature distresses that reduce the durability of flexible pavement sections have an economic impact because of necessary maintenance and reconstruction. Cedergren (1988) states that the results of the AASHO Road Test and the WASHO Road Test basically gave rise to the practice of building highways based on strength, not drainage, for performance. Most of the original Interstate pavement structures were built with this methodology in mind. The dominant concept was that if adequate pavement and base material were used for construction, then adequate drainage methods were not necessary. Based on these design practices, many miles of pavement sections deteriorated in as little as 6 to 10 years after initial construction (Cedergren, 1988). These life spans of sections are much less than they were designed resulting in premature rehabilitation and essentially premature construction funding needs. All in all, Cedergren

(1988) concluded that 2/3 of the $329 billion required maintaining 1975 levels of condition and performance could have been saved by good, sufficient drainage of all high traffic pavements.

More recently, Zaghloul et al. (2004) performed a case study demonstrating and quantifying the benefits of providing subsurface drainage through the reduction of moisture in daylighted base courses. Daylighted base courses drain water into a side ditch adjacent to the roadway without any type of drains or collector pipes. The effect of subgrade moisture content was assessed through deflection testing by falling weight deflectometer. Based on moisture content and deflection data, Zaghloul et al. (2004) assigned a structural adequacy index that was used to define structural service life. These structural service life values were run in a life cycle analysis to determine the effect of excessive moisture on serviceability. Zaghloul et al. (2004) concluded that an increase in base moisture content from 16% to 45% results in a decrease of pavement service life from 13 to 7 years. For a 40 year period, this would result in a life cycle cost three times greater (Zaghloul et al. 2004). These results using a life cycle cost analysis clearly indicate the importance of removing entrapped water from within a pavement.

Drainage Layer Considerations

There are essentially two options that designers can use in order to prevent the loss of pavement integrity due to the presence of entrapped water. (Lindy and Elsayed, 1995). First, moisture can be prevented from entering the pavement section by sealing joints and cracks. However due to the nature of flexible pavements with time, cracks will multiply and propagate on the surface, which makes this option expensive and impractical (Lindy and Elsayed, 1995). The second option is to incorporate a drainage layer within the pavement system that easily permits the drainage of excessive moisture from the pavement. This option of including a drainage layer within the pavement system is more practical and more economically friendly in order to minimize the effects of entrapped water within a pavement.

One feasible option to include a drainage layer within the pavement section is to place a layer of open-graded, highly permeable granular base within the pavement section. Dense graded bases do not provide sufficient permeability to drain excessive moisture. Figure 2.1, from Cedergren et al. (1972), shows typical gradations and permeabilites of open graded bases and filler materials. An open graded base course consisting of 1.5 to 1 inch aggregates would possess an estimated permeability of 120,000 ft/day. As the percentage of small aggregate particles increases, or the material becomes more well graded, the permeability estimates decrease. Most unbound drainage layers have limited amounts of fine material in order to provide sufficient drainage.

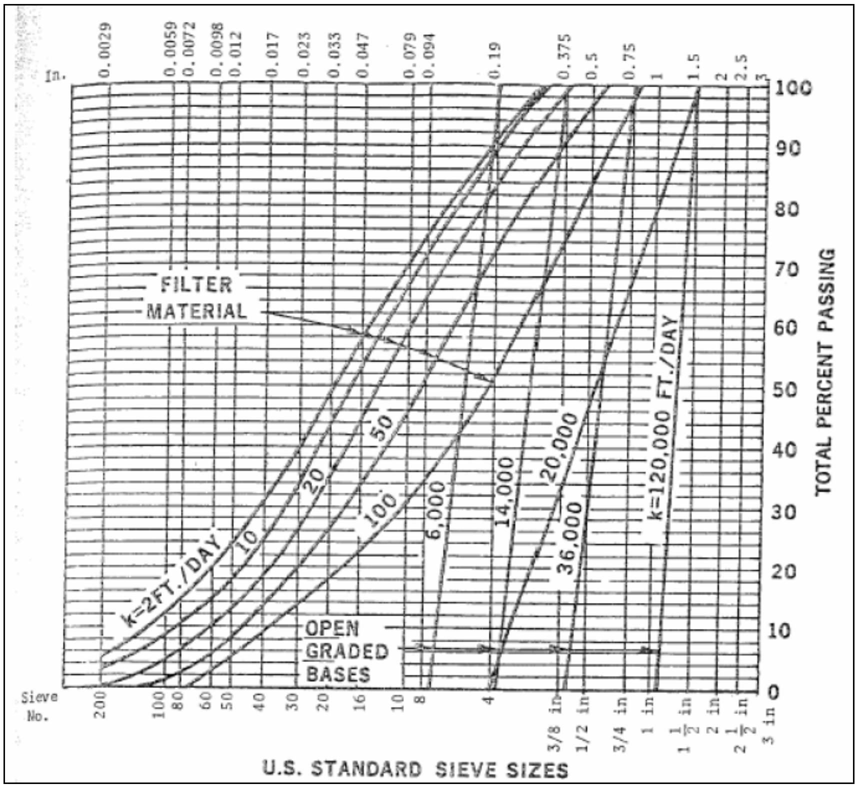

Figure 2.1. Cedergren Chart Used to Estimate Permeabilities of Typical Gradations. (from Cedergren et al., 1972)

Stability and Permeability

The use of unbound granular drainage layers also brings stability issues. The typically uniformly graded aggregates used for the layer oftentimes cannot be sufficiently compacted to provide ample support for construction traffic. The construction traffic would orient the aggregates which would result in a rough, non-uniform surface for the construction of the ensuing layers.

Lovering and Cedergren (1962) proposed using coarse material mixed with a relatively low amount of asphalt binder (2 to 3 percent) that would provide a desirable permeable base to allow drainage and withstand construction traffic much better than unbound granular drainage layers and result in a reduced thickness of the drainage layer.

By adding asphalt binder to the large aggregate particles, the permeability values decrease. Table 2.1 shows Lovering and Cedergren's results of permeabilities for untreated and asphalt treated open grade aggregates.

Table 2.1. Permeabilities of Untreated and Asphalt Treated Open Grade Aggregates. (Lovering and Cedergren, 1962)

Aggregate Size	Permeability, ft/day	
	Untreated	Bound w/ 2% Asphalt
1 - 1/2" - 1"	140,000	120,000
3/4" - 3/8"	38,000	35,000
#4 - #8	8,000	6,000

Two observations can be made from the data presented in Table 2.1. The first is that permeability decreases with decreases in aggregate size. This trend is also reflected in the Cedergren chart in Figure 2.1. The second conclusion is that although the permeability for the asphalt treated open graded aggregates is lower, they still possess roughly 75 to 80 percent of the permeability of the untreated aggregate. Therefore, adding 2 percent to 3 percent asphalt to the open graded granular base produces a lean HMA mix that possesses ample permeability and provides a more stable construction platform.

Lovering and Cedergren (1962) stress that pavements built with these permeable asphalt treated base courses are designed to prevent the clogging and silting of the pervious layer. Furthermore, drainage outlets must be provided so that water is not retained in the PATB layer and cause subsequent damage. To prevent clogging geotextiles are commonly placed beneath the PATB layer to prevent the pumping of silts and fines into the PATB layer. Also, PATB layers are commonly constructed with accompanying edge drains to provide an escape route for the water. It is vital that edge drains are maintained and cleared of any debris so that they do not become clogged. If these edge drains do become clogged, water will be retained within the drainage layer and the resulting entrapped water related distresses will become prevalent.

Because PATB can provide an acceptable working construction platform and possess sufficient permeability characteristics to drain infiltrating water, many agencies have incorporated them in their pavement designs. Harvey et al. (1998) reported that the California Department of Transportation had been using asphalt treated permeable base (ATPB) courses in the structural design of flexible pavements since the early 1980's. Maupin (2004) states that the Virginia Department of Transportation (VDOT) has been using these drainage layers since the early 1990's. Kane and Schwandt (1999) reported that the city of Chicago has used more than 2,000,000 square meters of either asphalt or cement treated permeable base material in its airport system since 1991.

The Federal Highway Administration (FHWA) provided a best practice document for drainage systems based on a number of state specifications in its Drainable Pavement Systems Participant Notebook (FHWA 1992). The FHWA (1992), states that aggregate used for permeable base contain essentially no fines and that it must be crushed in order to have good interlock. It recommends that L.A abrasion wear should not exceed 45% and should have adequate soundness (FHWA, 1992). It also recommends that the minimum coefficient of permeability of the PATB layer be 1,000 ft/day, however, permeability coefficients of 2,000 ft/day to 3,000 ft/day are preferred. The FHWA also recommends a minimum cross slope of 0.02 ft/ft and a minimum thickness of 4 inches.

Maximum outlet spacing should not exceed 250 feet and length of drainage paths should be kept to a minimum.

Baumgartner (1992) stated that a permeable base must provide both permeability and stability. Aggregate gradations must be carefully selected to achieve both desired qualities. He also states that most state highway agencies use an AASHTO number 57 or AASHTO number 67 gradation for their stabilized permeable bases. Compaction required for PATB specified by many agencies is one to three passes of a 15 or 10 ton steel-wheeled roller. Vibratory rollers should not be used because they cause degradation, densification and subsequent loss of permeability.

Fundamentals of Permeability and Stability in Permeable Asphalt Treated Base

Permeability in asphalt mixtures is dictated by a number a factors. Many studies have shown results that permeability of asphalt mixes is a function of percent air voids, size of the air voids, aggregate gradation, aggregate shape, specimen thickness and compaction procedures (Masad et al., 2004). These factors are incorporated into empirical equations that quantify the coefficient of permeability for granular materials. The Kozeny-Carman equation has been used to predict the permeability of saturated, granular materials over the years (Masad et al., 2004). The Kozeny-Carman equation is (Kozeny, 1927; Carman 1957):

$$k_s = \frac{Cn^3 {D_s}^2 \gamma}{(1-n^2)\alpha} \tag{2.1}$$

Where: k_s = the coefficient of permeability of a saturated medium

C = shape factor and for spherical particles is equal to 1/180

D_S = average diameter of particles

n = percent air voids,

γ = unit weight of water

α = fluid viscosity.

This equation was modified based on the works of Lytton (2004) and Fredlund and Rahardo (1993) to account for the behavior of asphalt coated aggregate and also degree of saturation. The resulting equation takes the form:

$$k = \frac{\overline{C}n^3}{(1-n^2)}\left[D_s\left(1+\frac{G_{sb}\left(P_B - P_{ba}\left(1-P_B\right)\right)}{G_b(1-P_b)}\right)^{\frac{1}{3}}\right]^2 \frac{\gamma}{\alpha} \tag{2.2}$$

Where: C = empirical coefficient to account for degree of saturation

G_b = binder specific gravity,

P_{ba} = percent of absorbed binder by weight of aggregate,

P_b = percent of asphalt by total weight of mix and

G_{sb} = bulk specific gravity of the aggregate

All other variables were previously identified in Equation 2.1. The equation shows that percent air voids holds the greatest influence on permeability. The average particle size also is influential in dictating the permeability of the asphalt mix. As the average diameter of the particles increase, the permeability increases. This supports the relationships shown in Figure 2.1. The more open –graded mixtures have on average, larger particle diameters which results in higher permeability. The Kozeny-Carman equation and the subsequent variants, however, were developed for granular material and dense graded asphalt mixes and may not be applicable to PATB. Nonetheless, it illustrates how the parameters involved affect the permeability coefficient.

Lindly and Elsayed, (1995) developed a method to estimate a laboratory coefficient of permeability for asphalt treated base course with multiple size aggregates present. Three types of aggregates (crushed limestone, crushed granite and uncrushed river gravel) and two different gradations (maximum aggregate size = 1.5 inches) were tested. Five replicates to account for test variability were conducted on each combination. The resulting regressions equations was:

$$k = 852.298 - 248.665 P_b + 97.507 V_a - 95.52 P_{\#8} \qquad (2.3)$$

Where: k = laboratory coefficient of permeability, ft/day

P_b = percent asphalt cement by total weight of sample

V_a = percent air voids by total volume of sample

$P_{\#8}$ = percent by weight passing the #8 sieve (2.36 mm)

Similar to the result in Table 2.1, adding asphalt to the open grade layer will reduce the permeability. Likewise, as the percent air voids increases the permeability increases, for there are more open channels for the water to pass through the sample. As the number passing the No. 8 sieve increases, the estimated value of permeability decreases, which agrees with the data in Figure 2.1 and Table 2.1.

The stability of an asphalt mixture is often explained in terms of Marshall Stability. Marshall Stability is defined as the maximum load carried by a compacted specimen tested at 60°C (140°F) at a loading rate of 2 inches per minute (Roberts et al. 1996). Roberts et al. (1996) state that stability is affected by the aggregate interlock and the viscosity of the asphalt binder at 60°C. Therefore, one method to increase stability of an asphalt mixture is to change to a stiffer asphalt binder. Bejarano and Harvey (1994) state that deformation problems during the construction of CalTrans ATPB led to the use of aggregate with 90% crushed particles and a change in asphalt from AR-4000 to the stiffer AR-8000. If stability issues occur, however, changing the grade of asphalt and increasing crushed particles may not be an option. Maupin (2004) in attempting to find a more stable gradation for a permeable asphalt treated layer, suggested blending existing Number 68 and Number 8 aggregates to create more aggregate contact points, which in turn will provide more strength.

Permeability and stability are inversely related. Permeability is mostly influenced by the amount, size, and interconnectivity of voids within the asphalt. More and larger voids will result in higher permeability but lower stability for the HMA. Higher durability is achieved by reducing air voids. In order to quantify if a PATB mixture is adequate for use, there needs to be a center of balance that permits acceptable permeability while maintaining sufficient stability.

Issues and Concerns Related to the Use of PATB

There are some issues and concerns pertaining to the use of PATB during construction, long term performance and in the laboratory. In this section, these issues are discussed based on findings from other research projects.

Issues and Concerns Related to the Construction of PATB

Even though PATB provides a more stable construction platform than unbound open-graded granular material, there is concern about the lack of sufficient stability during construction. Maupin (2004) investigated the possibility of improving the current asphalt treated permeability base courses used by the VDOT. This research was initiated based on the premise that the current base course used by the VDOT was difficult to place and durability was of concern. Furthermore, there was a problem with increased roughness transferring to the subsequent layers during paving. This was attributed to the fact that the open-graded asphalt treated base course contained mostly large aggregates that did not provide a smooth layer for the subsequent layers (Maupin 2004).

In order to alleviate these aforementioned problems, Maupin (2004) opted to test mixtures with smaller nominal aggregate size in order to fix the construction issues and provide a smoother surface. The tests used by Maupin to determine if the new mixes would prove sufficient were gyratory volumetrics, asphalt draindown, permeability, and Marshall Stability. Maupin (2004) concluded that the results of the stability testing lacked any significant relevance and were not presented. After 65 revolutions in a gyratory compactor, the samples had finer aggregate gradation had air void contents near 20 percent. Furthermore, for samples with asphalt contents less than 4.5%, the permeability values were all above 1,000 ft/day. After concluding the finer gradation would yield acceptable permeability values, a field test was performed. The field tests indicated that finer graded PATB supported construction traffic without any difficulty. The permeability values for cores with air voids ranging from 20 percent to 24 percent, were 870 ft/day to 1,600 ft/day. All in all, Maupin (2004) concluded that the implementation of a finer gradation for PATB was a success.

Issues and Concerns Related to the Long Term Performance of PATB

Harvey et al. (1998) investigated long term performance of CalTrans' ATPB by performing stripping tests and by investigating the loss of support field studies. The

ATPB had 90 to 100 percent of particles between the 9.5 and 19.0 mm sieve with 2.0 to 2.5 percent AR-8000 asphalt.

Field performance observations determined that stripping of asphalt in the ATPB layer was common in flexible pavements within ten years of construction. For some instances, where there was a large amount of water entering the pavement section, no asphalt was found on the ATPB aggregate particles. In three of the nine samples there was evidence of fines intrusion, but clogging was not an issue. In order to fix the stripping issue, Harvey et al. (1998) recommended that the asphalt contents of CalTrans ATPB be increased to 2.5 to 3.0 percent.

Harvey et al. (1998) also performed simulations to predict the behavior of ATPB layers. They tested and compared three cases: a Class 2 aggregate base with no ATPB layer, an ATPB layer before it had experienced soaking, and an ATPB layer after it had been subjected to periods of soaking. Predictions were based on the results of resilient modulus testing of as-compacted unsaturated and saturated samples. The results indicated that ATPB layers improve the predicted pavement fatigue lives when compared to an unbound granular base. This improvement is no longer relevant, however, when water damage occurs in the ATPB (Harvey et al., 1998). The loss of cohesion between the aggregate particles due to the stripping of asphalt causes the layer to behave more like an unbound open-graded base material.

This behavior was actually predicted by Lovering and Cedergren (1962) to occur. They stated that the stripping of the asphalt films would occur in the lean asphalt mixes, but this would occur long after construction and have no effect on the drainage capacity of the layer. It can be said from these observations from Harvey et al. (1998) and

Lovering and Cedergren (1962) that the PATB layer can serve as an adequate drainage layer, but should probably not be designed as a structural layer due to the loss of support.

Issues and Concerns about the Laboratory Testing of PATB

PATB is an open-graded HMA mixture with very little asphalt binder. Because PATB is open-graded, there are some concerns about sample preparation and testing. The two main laboratory issues are compaction and testing for air voids.

Lindy and Elsayed (1995) tested a number samples for permeability to develop their equation for estimating the hydraulic conductivity of asphalt treated base courses (Equation 2.3). They stated that because of the lack of fines, the standard Marshall hammer could not be used for compaction because it would crush the large aggregate particles and pump the asphalt to the surface of the samples (Lindy and Elsayed, 1995). Instead they employed a static load to compact specimens to a pre-set height to obtain samples with unit weights in a desired range.

The gyratory compactor has now become the method of choice for compaction. Maupin (2004) used a gyratory compactor to prepare samples in his research. Maupin (2004) did not consider crushing of aggregate particles and produced samples with desired characteristics in terms of permeability and stability. The number of gyrations set by Maupin (2004) was 65. Also note that Maupin was compacting a finer type PATB sample to an air void content of approximately 20 percent. The gyratory compactor or static loading appear to be viable options to compact asphalt treated permeable base samples.

Another laboratory issue of concern is the testing for the bulk specific gravity of PATB samples. Because the PATB samples are so porous, traditional bulk specific gravity testing procedure (AASHTO T166) cannot be performed because a saturated surface dried sample does not hold the absorbed water and yields misleading results (Buchanan and White, 2005). Research has shown that vacuum sealing technology using the Corelok vacuum sealing device produces more accurate bulk specific gravity of coarse graded mixtures. The Corelok procedure (ASTM D6752) was employed by Cooley et al. (2002) and has major advantages over parafilm or paraffin procedures (AASHTO T275) because of time requirements.

Buchanan and White (2005) performed a study to illustrate the bulk specific gravity differences between water displacement and Corelok procedures. Their results indicate that there are clear differences between the bulk specific gravity calculated by traditional water displacement methods (AASHTO T166) and the Corelok procedure (ASTM D6752). The Corelok procedure more accurately determined the bulk specific gravity of coarse graded Superpave mixes. Considering that porosity of the PATB samples is even greater than coarse graded Superpave samples, the Corelok procedure is the most appropriate method to determine the bulk specific gravity of PATB samples.

The Corelok vacuum sealing method is outlined in ASTM D6752. The dry samples is weighed and then sealed in the Corelok vacuum device. The sealed sample is weighed again and submerged in water. The submerged weight is recorded and then sample is removed from the seal. The weight is then recorded and compared to the initial recording. If a change of mass greater than 5 grams is observed, then the test is repeated. An increase in mass would be caused by water entering a punctured bag. Equation 2.4 is used to calculate the bulk specific gravity.

$$G_{mb} = \frac{A}{\left[C + (B - A) - E - \frac{B - A}{F_T}\right]} \tag{2.4}$$

Where: A = initial mass of dry specimen in air, g

B = mass of dry sealed sample, g

C = final mass of specimen after removal from sealed bag, g

E = mass of sealed specimen underwater, g

Ft = Apparent specific gravity of plastic sealing bag

The Corelok device is accompanied with software entitled Gravity Suite, which performs the calculation of the bulk specific gravity once the data are entered. Gravity Suite also calculates the percent air voids when the theoretical maximum specific gravity is known. The theoretical maximum specific gravity (Rice Density) is calculated after performing AASHTO T209. The calculation of percent air voids of a compacted asphalt sample in shown in Equation 1.5 (Roberts et al. 1996).

$$\% AirVoids = \left(1 - \frac{G_{mb}}{G_{mm}}\right) * 100 \tag{2.5}$$

Where: G_{mb} = Bulk Specific Gravity

G_{mm} = Maximum Specific Gravity

Summary

In summary, the use of PATB as a drainage layer has been widely documented as a method to provide sufficient permeability and stability during the construction process. It is important to permit the quick passage of intruding water from a pavement section in order to minimize potential distresses, resulting premature failure, and construction and rehabilitation costs. PATB in general is an open-graded mixture with crushed aggregate lean asphalt mixture, with asphalt contents ranging from 2.0 percent to 3.5 percent. PATB layers should have a minimum coefficient of permeability of 1,000 ft/day.

CHAPTER 3

PATB CONSTRUCTION SPECIFICATIONS

Introduction

ALDOT's specifications for the construction of PATB were thoroughly investigated in order to meet the requirements of the objectives of this research. Other states' specifications for the construction of PATB were also investigated and summarized. The states investigated were selected based on their proximity to Alabama. A more thorough investigation of the other state specifications for the construction of PATB is presented in the Appendix.

ALDOT's Specifications for PATB

Construction of permeable asphalt treated base courses (PATB) funded by the Alabama Department of Transportation (ALDOT) must follow the specifications set forward in ALDOT's 2002 Edition of Standard Specifications for Highway Construction, mainly those stated in Section 327. There are four main categories included in the specifications for constructing PATB. The general requirements are aggregate gradation, binder content, sampling and testing frequency, and construction requirements.

ALDOT's General Requirements for Constructing PATB

ALDOT mandates that PATB is to be constructed as an open graded, hot laid, central plant mixed asphalt base course. The specifications have no requirements for density, air voids, voids in mineral aggregate (VMA), or stability.

Asphalt Binder

The grade of the liquid asphalt binder is to be the same grade and type as the overlying surface course layers, most commonly PG 67-22 performance grade liquid asphalt binder unless specified otherwise by the construction plans. For example, Division 4 has recently been using polymer modified PG 76-22 asphalt binder in all of its PATB mixtures. Performance graded asphalt must be produced by the refining of petroleum. Air blown or oxidized asphalt is prohibited. The asphalt used is to be free of water, homogeneous, and must not foam when heated to 347 °F. Other requirements for performance graded asphalt include that the binder must meet the requirements stated in AASHTO section M320 Table 1 and those stated in Table 3 of Article 804.07 of the ALDOT specification manual. The specifications addressed in these tables cover flash point temperature, rotational viscosity, and dynamic shear of the liquid asphalt binder.

ALDOT also regulates that the liquid asphalt binder content for PATB ranges from 2 to 2.5 percent by weight. Additives may be added to the mixture in order to eliminate any potential stripping problems. Stripping is determined to be a problem after verification from vapor and moisture susceptibility testing (ALDOT – 255). It should also be noted that Reclaimed Asphalt Pavement (RAP) is prohibited in the PATB.

Aggregate

The aggregate used in the PATB is ALDOT designated No. 57 crushed stone. According to the specifications all crushed stone shall be clean, tough, durable fragments, free of shale, conforming to the requirements of physical testing, while fitting the specified gradation. This specific stone type is mostly coarse aggregates and possesses very little fines. The gradation for PATB is displayed in Table 3.1. Figure 3.1 shows the grading envelope for PATB.

Table 3.1. Gradation Specification for PATB Aggregates.*

Sieve (Square Mesh Type)		% Passing by Weight
1.5 inch	37.5 mm	100
1 inch	25 mm	95 - 100
1/2 inch	12.5 mm	25 - 60
No. 4	4.75 mm	0 - 10
No. 8	2.36 mm	0 - 5
No. 200	74 um	0 - 2

* from the ALDOT Standard Specifications from Highway Construction, 2002

All aggregates used must comply with general aggregate requirements stated by ALDOT. First, the aggregate must originate from a predetermined qualified source listed by ALDOT. The physical tests required are durability and soundness. For bituminous applications, the percent wear of coarse aggregates from the Los Angeles Test (AASHTO T 96) must not exceed 48%. For soundness requirements, ALDOT states that a 90% minimum value must be exhibited by coarse aggregates after 5 cycles of sodium sulphate soundness testing (AASHTO T 104). The crushed stone must not be heated above 280°F during the drying process. The mix of the dry aggregate and the liquid asphalt binder is not to exceed 250°F.

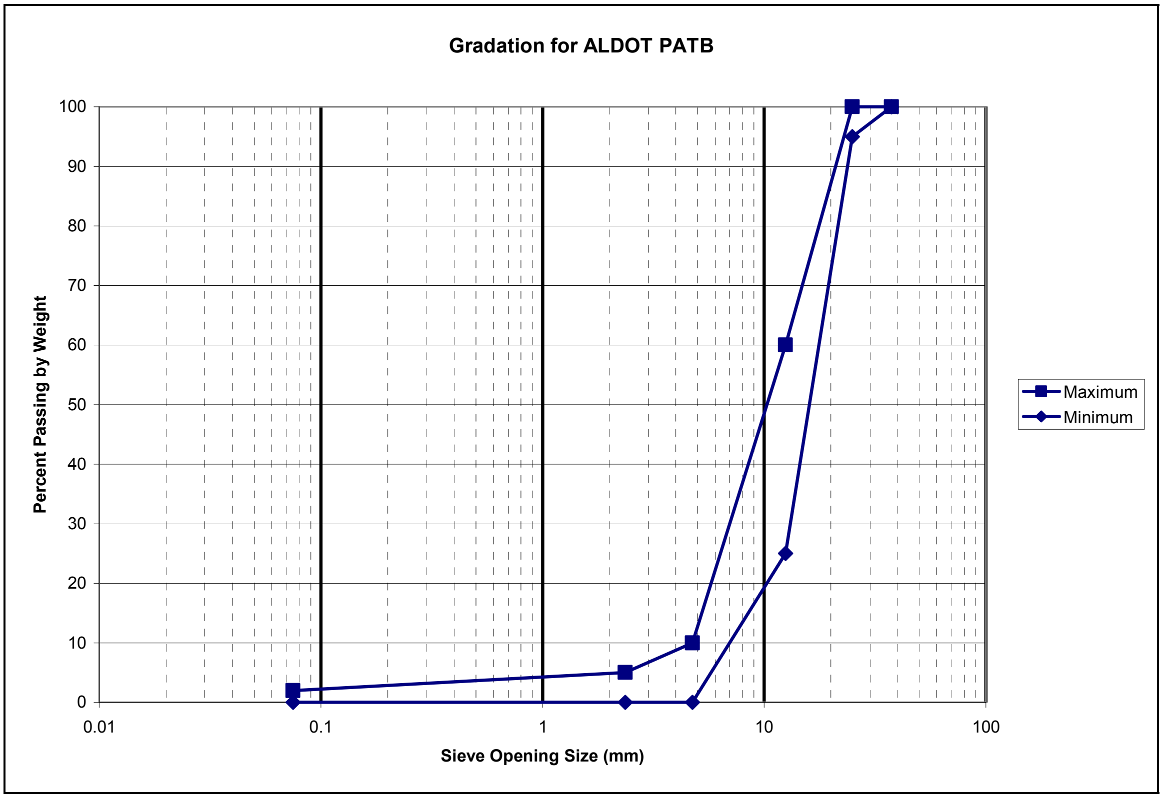

Figure 3.1. Gradation Composition for Aggregate in ALDOT PATB.

Geotextiles

A geotextile fabric may also be used in the construction of the PATB. Its purpose is to prevent the passage of fine material into the PATB layer from the underlying layer while also permitting the passage of water and retaining coarse material from the PATB layer. The fabric must consist of non-woven synthetic fibers and must be resistant to temperatures encountered during the placement of the PATB. It must lie across the entire section of the roadway free of debris and loose aggregate. Upon lay down of the geotextile fabric no more than three days may pass before the paving of PATB. Any damage to the fabric is to be repaired immediately and prior to the lay down of the PATB. The fabric must also meet the requirements set forth in AASHTO M 288 for Separation Geotextile Class 3.

Sampling and Testing Frequency

ALDOT only requires testing of the asphalt content, mixture gradation, and stockpile aggregate gradation. Common tests that are not required are Marshall stability and flow, air void content, VMA, retained tensile strength, and maximum specific gravity. Tables 3.2 and 3.3 show the mandated sampling and testing for the aforementioned requirements.

Table 3.2. Sampling Requirements for PATB.*

Sampling Requirements			
Control Parameter	**Sample Size**	**Sampling Methodology**	**Sampling Location**
AC Content	25 lbs. (12 kg)	AASHTO T 168 & ALDOT 210	Loaded Truck
Mixture Gradation	25 lbs. (12 kg)	AASHTO T 168 & ALDOT 210	Loaded Truck
Stockpile Gradation	10 lbs. (5 kg)	AASHTO T 2	Stockpile

* from the ALDOT Standard Specifications from Highway Construction, 2002

Table 3.3. Testing Requirements for PATB.*

Testing Requirements				
Control Parameter	**Sample Size**	**Testing Methodolgy**	**ALDOT Testing Frequency**	**Contractor Testing Frequency**
AC Content	25 lbs. (12 kg)	ALDOT - 319	1 per day	1 / 600 tons
Mixture Gradation	25 lbs. (12 kg)	ALDOT - 371	1 per day	1 / 600 tons
Stockpile Gradation	10 lbs. (5 kg)	AASHTO T 11 & T 27		1 / 1000 tons

* from the ALDOT Standard Specifications from Highway Construction, 2002

Construction Requirements

The construction requirements for PATB are very similar to those for hot mix asphalt pavement surfaces except for the following. Vibratory type compaction is not permitted to perform the compaction of PATB because it most likely will result in overcompaction. Only static type compaction is prescribed to compact the PATB. The applied loading ranges from 0.5 to 1.0 tons per foot of roller width. Under direction of the field engineer, the roller is to make one to three passes on the PATB once the mix has cooled to 150°F.

The PATB course is not to be exposed to the elements for more than five calendar days. Another construction issue may arise if rutting occurs. In order to reduce the amount rutting on the newly constructed PATB, no traffic is permitted to park or drive along the travel lane or shoulder areas of the PATB. ALDOT does allow limited use of the inside edge of the PATB for purposes including delivery.

Summary of PATB Construction from Other State Agencies

Many states across the Southeast region of the United States have specifications for the construction of permeable asphalt treated base courses. The nomenclature of the PATB may vary from state to state. For example PATB is referred to as Open Graded Asphalt Base Course in Arkansas. Other variations include Asphalt Drainage Course (Mississippi). The specifications in the following summary are from the specifications published by: Arkansas, Florida, Louisiana, Mississippi, and Tennessee. These states were selected based on their geographic proximity to Alabama.

Three major categories were used to compare and contrast state practices for the construction of asphalt treated permeable base course. These categories are: asphalt binder, aggregate, and construction technique. Subjects discussed for asphalt binder will include the type of performance grade binder recommended, the asphalt content recommended, and the use of anti – stripping additives. Aggregate gradation and physical requirements of the aggregates will be the main topics for the comparison and contrast of the aggregates across the Southeast. Construction specifications may be the most variable between the various departments of transportation. Allowable construction traffic, rolling temperatures and permissible air temperature for construction are some of the requirements that vary the most.

Asphalt Binder in PATB across the Southeast

Alabama, Arkansas, Florida, Louisiana, Mississippi, and Tennessee all have specifications for the asphalt binder used in their permeable base course. Table 3.4 presents the specifications for a number of variables pertaining to the binder used. These variables are those that are most common to all of the specifications investigated. It should be noted that these values are those most commonly used and set forth in the various specification literature. These values (performance grade, rate of application, and use of anti – stripping additive) may vary project to project depending on the construction plans and material testing. The percent coverage of aggregate is a visual inspection that ensures that there is sufficient coating of the aggregate after handling and lay down.

Figure 3.2 shows the asphalt content ranges set forth in each state's specifications. Alabama (2.0% to 2.5%) and Arkansas (2.5% to 3.0%) have the tightest ranges of asphalt content, only by 0.5%, whereas Tennessee (2.0 to 4.0%) and Louisiana (2.0 to 4.0%)

have the widest ranges of 2.0%. All in all, the asphalt content for PATB ranges between 2.0 – 4.0% by weight of mixture for all of the states. Lower asphalt contents may yield mixtures that possess coating and stability issues; however those mixtures with higher asphalt content may have lower permeability and draindown problems. As shown, ALDOT's specification calls for the lowest maximum asphalt content value, which may cause the PATB layer to not be very durable. Mississippi (2.1% to 2.9%), Arkansas (2.5% to 3.0%), Alabama (2.0% to 2.5%) and Florida (2.0% to 3.0%) all do not exceed 3% asphalt content by weight of mixture.

Table 3.4. Specifications for Asphalt Binder in PATB for Various State Agencies.

State Agency	Asphalt Content	Performance Grade*	Anti-Stripping Additive	Anti-Stripping Additive Content	% Coverage of Aggregate
Alabama	2.0 - 2.5	PG 67 - 22	If Necessary **	Varies	No Requirement
Arkansas	2.5 - 3.0		If Necessary **	0.5 - 0.75%	No Requirement
Florida	2.0 - 3.0	PG 67 - 22	Yes	0.50%	95%
Louisiana	2.0 - 4.0	PG 76 -22m	If Necessary **	0.5 - 1.2%	90%
Mississippi	2.1 -2.9	PG 67 - 22	If Necessary **	1.00%	
Tennessee	2.0 - 4.0	PG 64 - 22	If Necessary **	Varies	100%
* Unless otherwise designated in plans					
** If stripping becomes an issue after moisture susceptibility testing or aggregate coating is not sufficient					

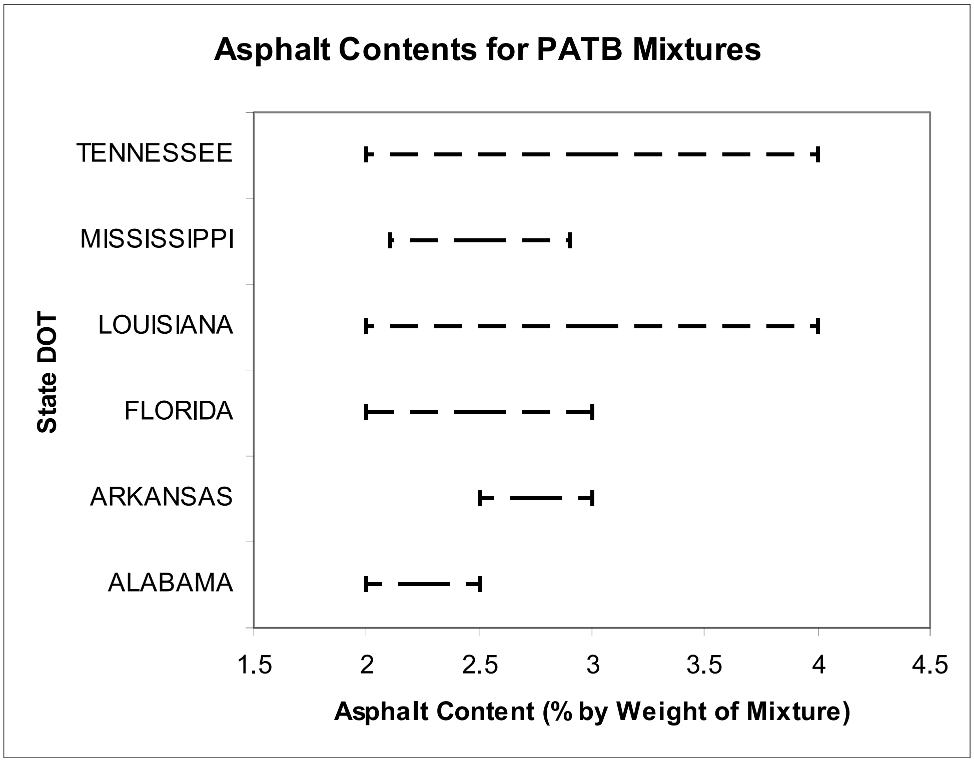

Figure 3.2. Asphalt Content Ranges by Weight of Mixture for PATB.

All of the investigated states, except Louisiana, specify that binder used in construction must comply with those parameters presented in AASHTO M320 Table 1. These parameters target flash point temperatures, viscosity, dynamic shear, creep stiffness, and direct tension for the various types of performance grade binders. Louisiana specifies that binders used in its permeable base courses must comply with Table 1002-1 in the 2000 LDOTD Specification Manual. The parameters are very similar to those featured in AASHTO M320 Table 1.

Permeable asphalt treated base courses commonly exhibit stripping problems, especially in areas where large amounts of water can enter the PATB layer. Due to this issue, all of the states use anti–stripping additives in the PATB mixture, if needed. Only Florida mandates that an anti–stripping agent be added to the PATB mixture. Alabama, Arkansas, Louisiana, Mississippi, and Tennessee all call for anti-stripping additive if issues arise. ALDOT calls for the addition of the agent if stripping issues arise after moisture susceptibility testing. Louisiana and Tennessee call for the anti-stripping additive if desired aggregate coverage is not achieved

The rate at which the anti-strip additive is added to the mixture also deviates across the Southeast. Alabama and Tennessee do not specify any value or range at which the additive is to be applied. Tennessee states that additive is to be added so that desired coating is achieved. Louisiana is similar to Tennessee in that it calls for the addition of additive, starting at a rate of 0.5% of asphalt binder but not to exceed 1.2%, to achieve desired coverage. Arkansas, Florida, and Mississippi all set forth ranges at which the additive is to be applied. A range of 0.5% by weight of asphalt binder appears to be the most common value.

Aggregate in PATB across the Southeast

The two main specifications that separate the aggregate used in PATB are gradation and physical requirements. Table 3.5 shows the various gradations used by the Southeastern DOTs in the construction of PATB. For data pertaining to Arkansas open graded base course, only Type IV is compared, since it is more comparable to the specifications of the other states in terms of both asphalt content and aggregate gradation.

Table 3.5. Aggregate Gradations for PATB in Various Southeastern States.

Sieve (Square Mesh Type)		% Passing by Weight					
		ALDOT	AHDT (T4)	FDOT (no. 57)	LDOTD	MDOT	TDOT
2 inch	50 mm						100
1.5 inch	37.5 mm	100		100		100	70 - 100
1 inch	25 mm	95 - 100	100	95 - 100	100	80-100	
3/4 inch	19 mm		90 - 100		90 - 100		55 - 80
1/2 inch	12.5 mm	25 - 60		25 - 60		25 - 60	
3/8 inch	9.5 mm		20 - 55		20 - 55		
No. 4	4.75 mm	0 - 10	0 - 10	0 - 10	0 - 10	0 - 10	0 - 11
No. 8	2.36 mm	0 - 5	0 - 5	0 - 5	0 - 5	0 - 5	
No. 100	0.150 mm	0 - 2					0 - 4
No. 200`	0.075 mm						0 - 3

As shown in Table 3.5, the gradations for the investigated specifications do not vary significantly, especially between the 1 inch and No. 8 sieve. Table 3.5 shows that Arkansas (Type IV) and Louisiana use the same aggregate gradation. Alabama, Florida, and Mississippi use the nearly same gradation except that ALDOT requires that 0 – 2% pass the No. 100 sieve. Tennessee has the most unique gradation, since TDOT allows a larger percentage of stones, up to 30%, greater than the 1.5 inch sieve in the blend.

By definition, permeable base courses do not contain large amount of fines, therefore, all states have similar limits of fines in their specifications. Tennessee is the only state that includes particles passing the No. 200 sieve. All of the other states allow for a maximum of 5% passing the No. 8.

Physical requirements of the aggregates vary by state. These requirements include LA Abrasion, soundness, and percentage flat and elongated. Table 3.6 lists the properties by state.

Table 3.6. Physical Requirements of Aggregate Used in PATB.

State Agency	Toughness (max. % wear) AASHTO T96	Soundness (max. % loss) AASHTO T104	Fractured Faces* (One: Two)	Mixing Temperature	Minimum Flat/ Elongated Particles
Alabama	48%	10%		Not greater than 250° F	10%
Arkansas	40%	12%	(98%:80%)	275 - 325 F	10%
Florida	45%	12%		230 - 285 F	10%
Louisiana	40%	15%		200 - 260 F	
Mississippi	45%	20%		220 - 250 F	
Tennessee		9%	(75%)†		
*for Aggregate greater than No. 8 sieve					
† for Aggregate greater than No. 4, for two fractured faces					

Toughness is intended to forecast the aggregate's ability to resist abrasion during construction and in service. The toughness requirements for aggregates subjected to the Los Angeles abrasion testing are not significantly different. Alabama allows for the highest values of loss at 48 percent. All values range between 40 to 48 percent maximum loss. Soundness estimates an aggregate's ability to resist weathering while in service. Most of the values for maximum loss after 5 cycles of sodium sulfate soundness testing fall between 9 and 15 percent by weight. Mississippi allows for a permissible soundness loss (20 percent), which is significantly higher than that of the other agencies.

The ability for the aggregates to sufficiently interlock is extremely important for the open graded course bases. Because the permeable base is not very dense, the aggregate interlock provides structure to the pavement layer. The percentage of fractured faces describes the amount of angularity that an aggregate has. Higher angularity yields better interlock. Only Arkansas and Tennessee specify the percentage of aggregate with fractured faces. Both states require at least 75 percent of the aggregate (either passing the No. 4 or No. 8 sieves) must have at least two fractured faces. Flat and elongated particles may cause the PATB mixture to be more difficult to handle and seat with rollers. Alabama, Arkansas and Florida specify that no more than 10 percent of the stones by weight of the complete aggregate mixture may be flat and elongated particles.

The temperature at which the aggregates and asphalt binder are mixed is another specification common to almost all of the Southeastern DOTs. The range of the mixture temperatures are displayed in Figure 3.3. Alabama's specifies that the finished mixture of aggregate and binder does not exceed 250°F. Arkansas specifies the highest mixture temperatures ranging from 275 – 325°F. Mississippi has the tightest range of allowable mixing temperatures which is 220 to 250°F. Louisiana allows the coolest minimum mixing temperature that ranges from 200°F to 250°F. Florida's maximum permissible mixing temperature is higher than Louisiana, Alabama, and Mississippi ranging from 230°F to 285°F.

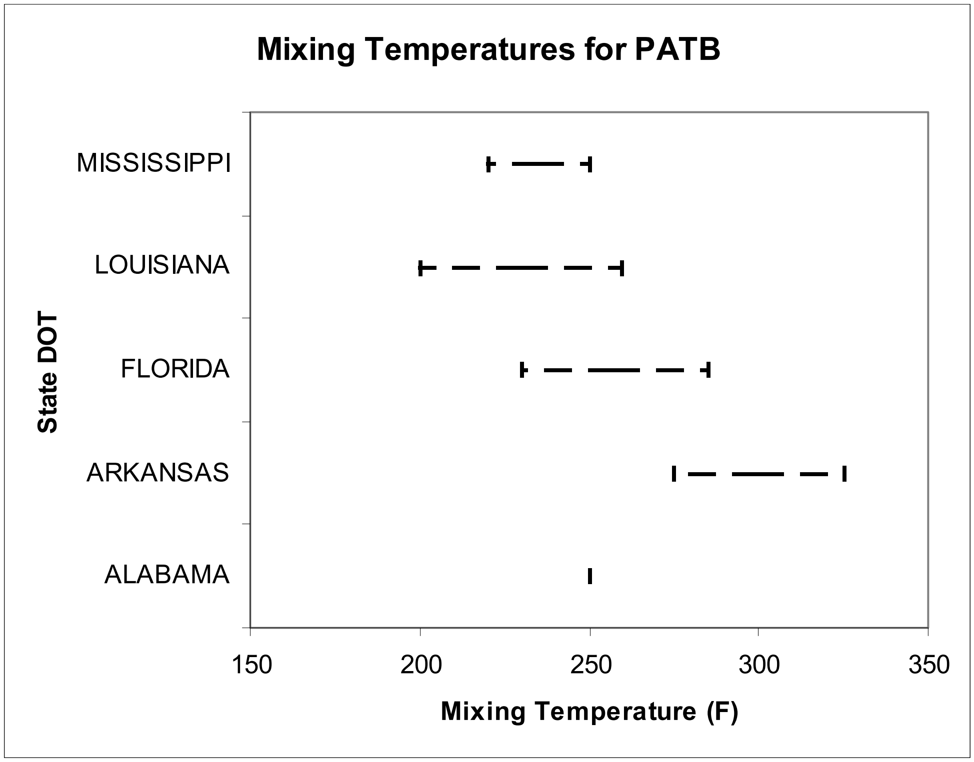

Figure 3.3. PATB Mixing Temperatures for Various Agencies.

Construction Technique for PATB across the Southeast

The specifications pertaining to the construction of PATB is unique for each state agency. Air temperature, compaction temperature, type of rollers, traffic allowed on the base course, and construction time frames are some of the specifications discussed. Table 3.7 shows some of these parameters including minimum air temperature required for lay down and protection requirements of permeable asphalt base course by state.

Table 3.7. Construction Requirements for PATB.

State Agency	Minimum Temperature for Construction (F)	Protection Requirements	Time Constraints
Alabama	40	No requirements	Subsequent layer placed within 5 Calendar Days of PATB paving
Arkansas	40	No requirements	No requirements
Florida	50	No requirements	Paved No Longer than 2 hours after ATPB is mixed
Louisiana	No requirements	Protected from Mud and Fines	15 Maximum Days until Subsequent Layer is constructed
Mississippi	40	Not Laid on Frozen/ Wet Layer	No requirements
Tennessee	No requirements	Protected from Rain/ Ice/ Fines	30 Days Until Construction of Next Layer No Construction between Nov. 1 and April 1

Permeable asphalt treated base course is susceptible to the intrusion of fines or any other material that will cause clogging. For these reasons, the states set forth regulations protecting the PATB. Tennessee and Louisiana both make it the responsibility of the contractor to provide protection to the PATB from the elements as well as any fines, dirt, or mud. It is also important that the PATB layer is not exposed for excessive periods of time. Alabama, Louisiana, and Tennessee all regulate time periods until the subsequent overlying layer is constructed. These durations are quite variable. For example, Alabama allows only 5 days whereas Tennessee allows 30 days. Best practice would be to construct the next layer as soon as possible. The air temperature at which construction occurs is also an issue. Arkansas and Mississippi all mandate that construction may not take place if temperatures are below 40°F. The ambient air temperature plays a major role in the cooling of the layer, which in turn affects compaction timing.

Each state sets forth specifications about the temperature for compaction and equipment to be used. Table 3.8 shows these specifications for the states researched. The specifications for compaction do not vary significantly from state to state. Compaction temperatures do vary on the maximum side of the ranges, however compaction must be completed before the temperature cools below 100°F. Vibratory compaction is not permitted. The weight of the rollers also is not significantly different nor is the number of passes required. Compaction practice for PATB across the Southeastern US is relatively the same.

Table 3.8. Compaction Effort Specifications.

State Agency	Compaction Temperature (F)	Type of Roller	Vibratory Allowed	Weight of Roller	Number of Passes
Alabama	Max. 150	Static Steel Wheel	No	0.1 -1.0 ton/foot of width	1 to 3
Arkansas	100 - 180	Static Steel Wheel	No	2 axle / 3 - 5 tons	1 to 3
Florida	100 - 190	Static Steel Wheel	No	8 to 12 tons	
Louisiana	100 -160	Static Steel Wheel	No	5 to 10 tons	1 to 3
Mississippi	100 - 150	Static Steel Wheel	No	8 to 12 tons	1 to 3
Tennessee		Static Steel Wheel	No	Min. 7.25 tons	

The specifications set forth by ALDOT in its construction of PATB are not significantly different than those specifications of Southeastern state highway agencies. Most of the states reviewed use similar asphalt contents and aggregate gradations for its PATB compared to ALDOT. Louisiana and Tennessee allow up to 4.0% asphalt content, which is much higher than the maximum of 2.5% permitted by ALDOT. Construction techniques do vary from state to state but are not drastically different. ALDOT could consider implementing some of these practices during construction, like protecting the paved PATB layer from the elements before paving of the subsequent layers, but they probably will not solve the stability issue. The stability issues that arise due to the current PATB in Alabama most probably originate in the aggregate structure of the mixture used by ALDOT.

Chapter 4

PHASE I

INVESTIGATING CURRENT PATB MATERIAL

In order to fulfill the requirements of determining a revised PATB blend with more stability during construction, yet maintaining adequate drainage, two stages were necessary. The preliminary stage was to thoroughly investigate the current PATB used in Alabama in accordance to the requirements set forth in the 2002 construction specifications. This stage entailed obtaining PATB material from a number of sites, as well as cores from compacted PATB layers. The second stage involved investigating a number laboratory prepared blends with various aggregate skeletons and asphalt contents. Upon completion of these two stages, conclusions were made concerning the results.

Phase I – Investigating Current PATB Material

Permeable asphalt treated base material was collected at two sites in Alabama. The first construction site visited was the widening of highway US 280 near Camp Hill, Alabama just north of Ross Road near milepost 91. The second construction site was the widening of Interstate 20 just east of the Eastboga exit in Talladega County. The steps taken to determine the properties of the current PATB were.

1. Obtain PATB material and cores from construction sites in Alabama.
2. Perform laboratory tests on the PATB cores
    a. Determine volumetric properties
    b. Stability testing
    c. Permeability testing
3. Preparation and Compaction of PATB material in gyratory compactor
4. Perform height and volumetric analysis to ensure sufficient compaction
5. Perform tests on the PATB laboratory compacted pills
    a. Determine percent air voids
    b. Permeability Testing
    c. Stability Testing
6. Analyze aggregate gradation

Obtaining PATB material and cores from construction sites in Alabama

US 280 near Camp Hill, Alabama had recently undergone widening to transform the two-lane highway into a four lane divided highway. PATB was placed underneath the new west bound lanes. PATB material was collected on December 6th, 2005. The air temperature of 50°F was above the minimum required temperature as stated in the construction specifications. The PATB layer constructed on US 280 was approximately 4" thick. It was placed upon a rubberized asphalt film that covered a dense stone layer followed by a 6" aggregate base and lime stabilized subgrade. The temperature of this base layer was approximately 56°F during paving of the PATB layer. As shown in

Figure 4.1, the temperature of the freshly laid PATB material was between 200 and 250°F upon exiting the paver.

Figure 4.1. Temperature of Freshly Paved PATB on US 280.

The PATB material appeared to be very rich in asphalt binder despite the specified 2.0 to 2.5 percent asphalt by weight content. It contained mainly large aggregate with little to no fine aggregate particles. It was clear to see with this particle aggregate skeleton that the PATB drainage layer would be very permeable. Compaction at this site consisted of one static roll after the PATB layer cooled to at least 150°F. Ten 5 gallon steel buckets were filled with PATB material sampled from the hopper of the paver. This material was used for laboratory investigation. On December 7th, 2005, cores were taken from roughly the same location on US 280 where the bucketed material was acquired. This ensured that the unpaved material came from the same source as the cores. It should also be noted that these cores were obtained from a location that had already undergone compaction. These cores contained mostly large aggregates with a high number of visible air voids. Figure 4.2 is a picture of one of the cores taken from US 280.

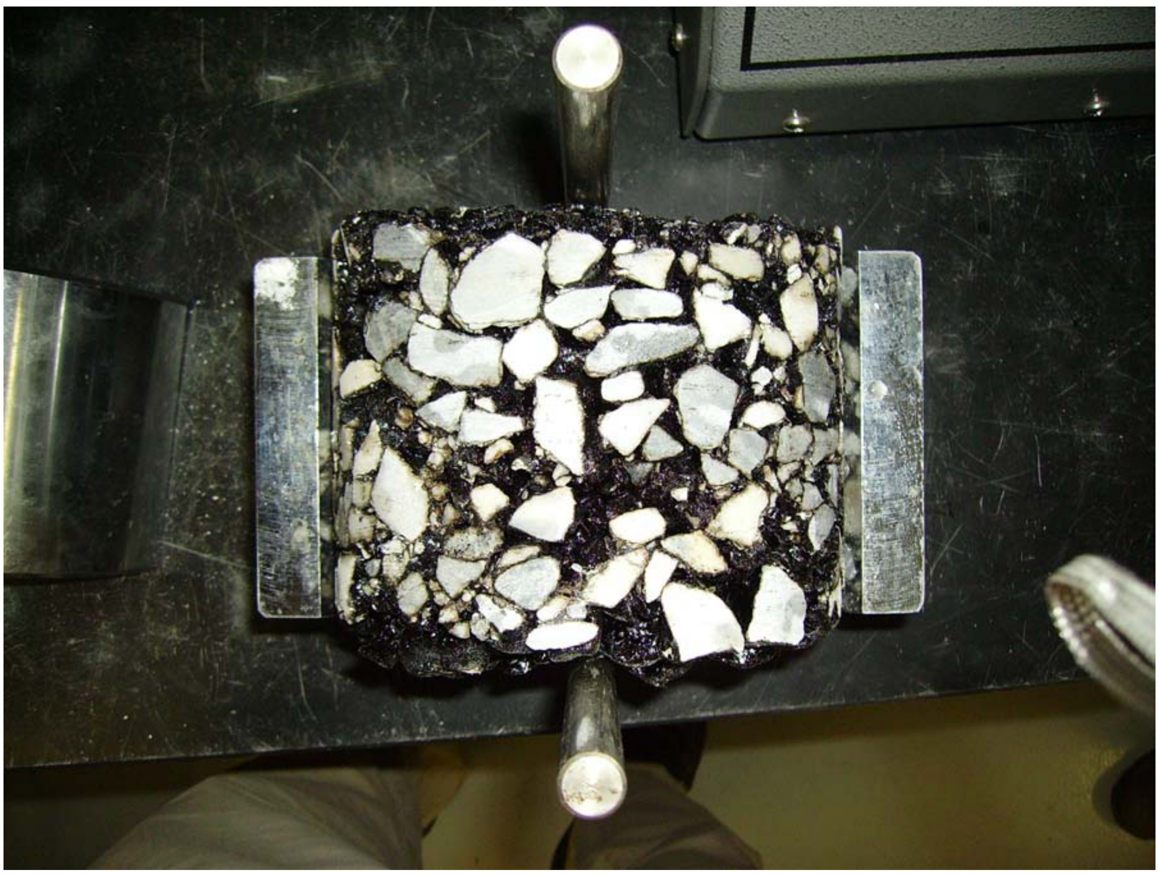

Figure 4.2. Core Obtained from US 280 Construction Site.

Interstate 20 was also being widened between Eastaboga and Coldwater, Alabama in Talladega County. PATB was used across the entire base of the newly widened pavement. In late May of 2006, material was collected on the newly constructed inside lane of westbound I-20 just east of the Eastaboga exit. Cores were taken from a previously compacted location. Seven buckets of fresh PATB were taken directly from the hopper. Field compaction of the PATB on Interstate 20 consisted of the same one static roll after the PATB had cooled. Time until compaction was much longer since the PATB was not cooling down quickly since the air temperature exceeded 90°F in the afternoon sun. It was observed that the traffic driving on the compacted PATB was causing some shoving and rutting in the base course layer. Traffic on these layers might have caused some breakdown of the aggregates. Figure 4.3 shows one of the cores taken from Interstate 20. This core looks very different from US 280 in Figure 4.2. These I-20 cores had more visible aggregate breakdown and a lower amount of visible air voids. The breakdown of aggregates might have been caused by traffic on the PATB layer or over compaction. The fresh PATB material taken from the site on I-20 also appeared to be much finer than the fresh PATB material obtained on US 280.

There were many lessons learned concerning the handling of the PATB cores. Due to budgetary constraints and a lack of PATB construction sites, a limited number of cores were obtained for investigations. These cores were placed in the laboratory at room temperatures similar to how most HMA samples can be placed prior to testing. These PATB cores, however, oftentimes crumbled while waiting for testing. This resulted in a limited number of tests and results for analysis. PATB field cores and laboratory pills should be frozen if stored prior to testing to limit the possibility of sample disintegration.

Figure 4.3. Core Obtained from Interstate 20 Construction Site.

Perform Testing on the PATB Field Cores

Laboratory testing of the cores taken from the field was the next step in studying the current PATB specification. Testing included determining the percent air voids, permeability, and stability of the PATB cores. Considering that the cores had already undergone field compaction and that there is no specification for air voids, it was desired to determine the percent air voids of the field compacted PATB. This task was performed using the Corelok device (ASTM D 6752), shown in Figure 4.4. As stated in the previous chapter, the Corelok device uses a vacuum sealing method that enables the calculation of bulk density for highly permeable samples.

The percent air voids could be calculated after the theoretical maximum specific gravity of the PATB material was found by performing a Rice Density (AASHTO 209-99) and determining the bulk specific gravity of each individual core. The Rice Density test yielded a theoretical maximum density (TMD) of 2.73 for the PATB from the US 280 cores and a TMD of 2.72 for the PATB from the I-20 cores. The percent air voids was calculated using Gravity Suite software, which accompanies the Corelok device. Table 4.1 shows the percent air voids of the US 280 cores.

Table 4.1. Percent Air Voids of Cored US 280 PATB Samples.

Sample ID	Bulk Specific Gravity (g/cm3)	% Air Voids
US 280 # 1	1.885	31.0
US 280 # 2	1.856	32.0
US 280 # 3	1.877	31.2
US 280 # 4	1.873	31.4
Average	1.873	31.4
Standard Deviation	0.012	0.438

Figure 4.4. CORELOK Machine.

Table 4.2 shows the percent air voids of the I-20 cores. Based on visual inspection, the I-20 cores were expected to have much lower air voids than the US 280 cores. The results of Tables 4.1 and 4.2 show that the percent air voids were only slightly different. US-280 cores had average percent air voids of 31.4%, whereas the I-20 cores had an average of 28.4%.

Table 4.2. Percent Air Voids of Cored I-20 Material.

Sample ID	Bulk Specific Gravity (g/cm3)	% Air Voids
I 20 #1	1.953	28.2
I 20# 2	1.944	28.5
Average	1.948	28.4
Standard Deviation	0.006	0.214

The next step taken was to determine the stability of the cored PATB material. Recall that the current 2002 ALDOT construction specifications have no requirements for stability for PATB material. Stability testing was performed in accordance to AASHTO T 245. The testing procedure calls for soaking of pills in a water bath at a temperature of 60 °C ± 1 °C (140 ± 1.8°F) for 30 to 40 minutes. The pills are then removed and positioned in the load frame within 30 seconds. The loading rate is 2 inches per minute (50.8 mm/min) until failure of the specimen.

Upon testing, two major observations were made. The first observation was that the PATB material had very low stability or resistance to flow during loading. The stability graphs for the PATB cores resembled a linear line (Figure 4.5). For some of the PATB cores tested, no maximum stability was achieved, but rather the maximum permissible flow was achieved. Stability curves for regular HMA will appear semi-parabolic. The stability increases more rapidly than flow and will eventually reach a maximum. The second observation was that upon removal from the hot bath, two of the cores literally crumbled upon removal. Figure 4.5 shows the stability versus flow for both US-280 and I-20 cores. The two other samples from US 280 had crumbled before stability testing could be performed. It can be said that the results of the cores are similar. No core tested had a significant stability value. All cores failed at stability values less than 500 pounds. The results of these tests support ALDOT engineers' observations that the PATB material has little stability during the construction process.

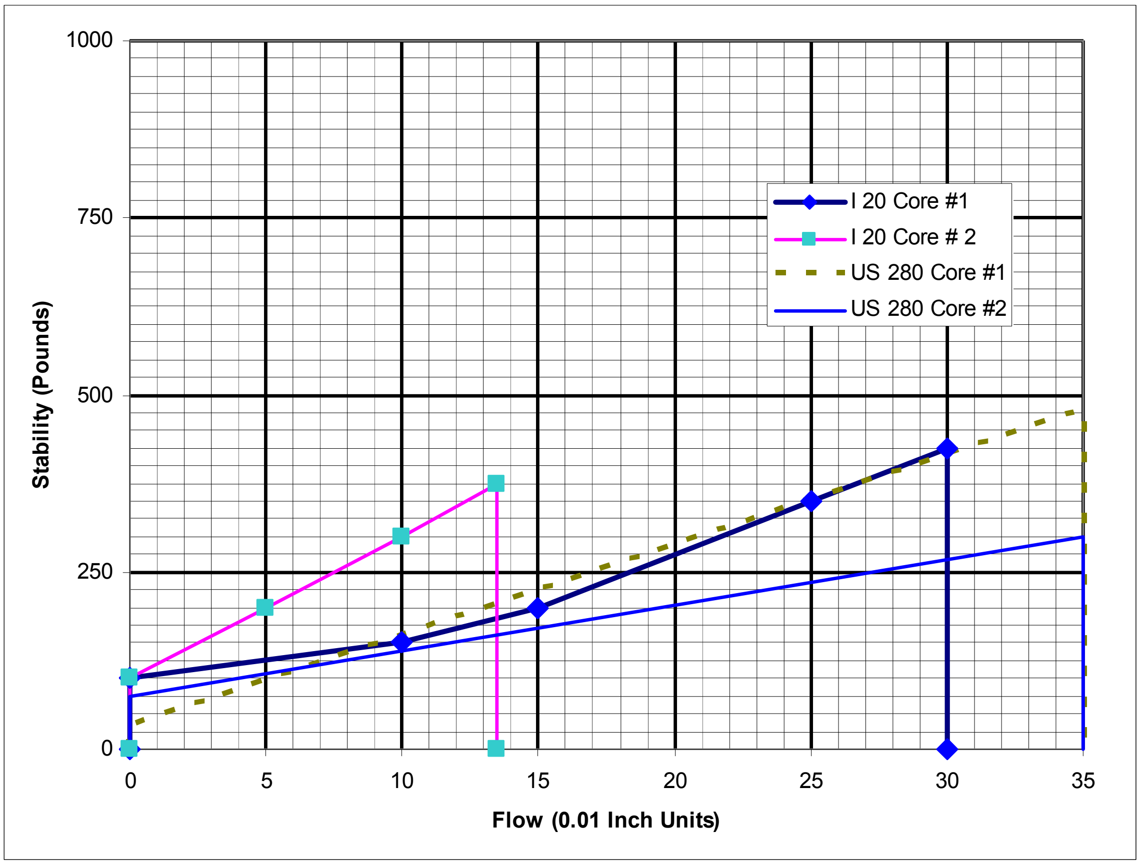

Figure 4.5. Stability of PATB Field Cores.

Permeability testing was also performed on the I-20 cored material. There were no US 280 cores left intact to perform permeability testing. Permeability testing done in the laboratory procedure was performed in accordance to the Florida DOT test method for measurement of water permeability of compacted asphalt mixtures, designation FM 5-565. The FDOT device, as it is referred to in other literature was used and is shown in Figure 4.6. The test employs the falling head approach of determining water permeability. This apparatus allows water in a graduated cylinder to flow through an asphalt sample. The change in the amount of head during a measured amount of time enables the calculation of the permeability coefficient for the asphalt sample based on Darcy's Law. Equation 4.1 is used to calculate the coefficient of permeability, *k.*

$$k = \frac{aL}{At} \ln(h_1 / h_2) * \tau_c \qquad (4.1)$$

Where: k = coefficient of permeability, cm/s

a = inside cross sectional area of the graduated cylinder, cm^2

L = average thickness of test specimen, cm

A = average cross sectional area of the test specimen, cm^2

t = elapsed time between h_1 and h_2, s

h_1 = initial head across the test specimen, cm

h_2 = final head across the test specimen, cm

τ = temperature correction for the viscosity of water, 20°C used as standard

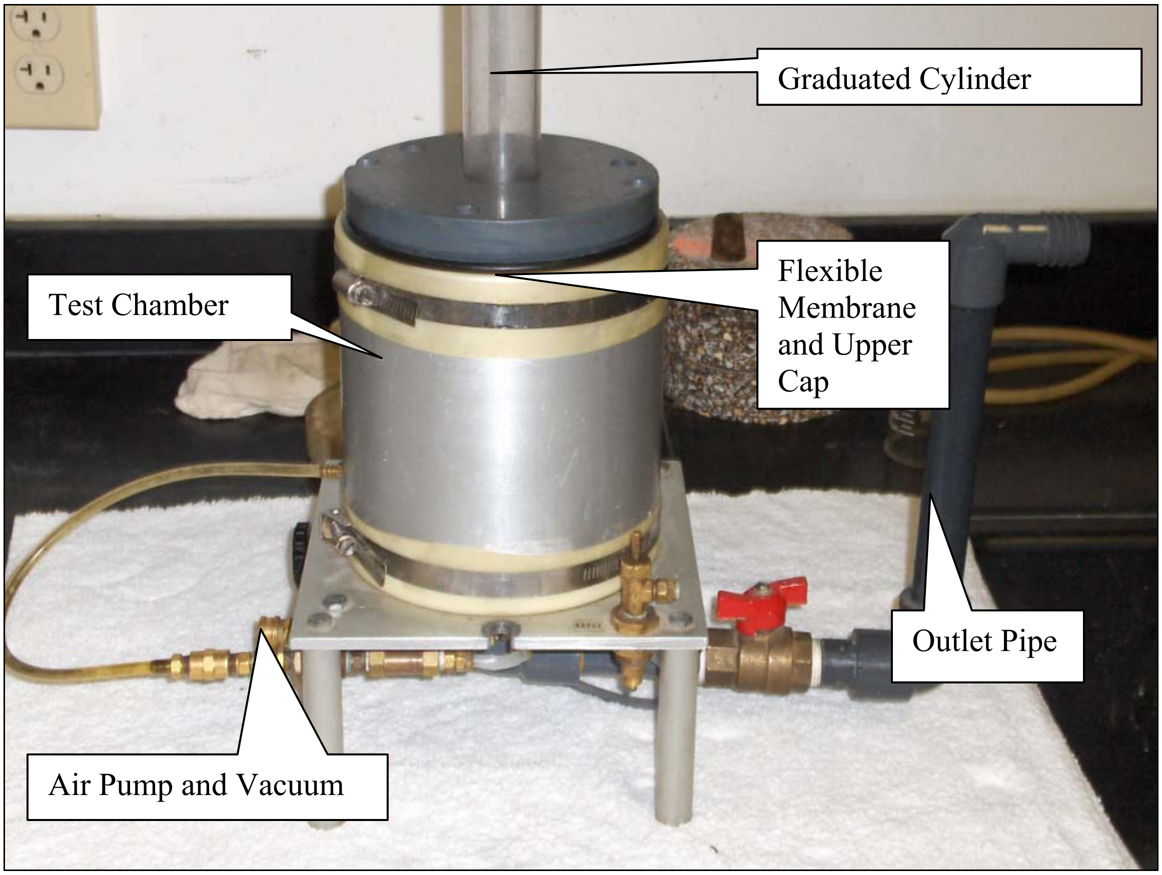

Figure 4.6. FDOT Permeameter Device.

The specifications of the test advise that the samples tested for permeability have the same height as those layers in the field. However, the cores could not fit in the sealing tube and upper cap assembly so they had to be cut down to approximately 4 inches in height. There was a concern, however, that by cutting the samples down to four inches would result in a change in the percent air voids. Air void contents were determined before and after cutting and showed very little change in air voids content. All values resulted in a change of percent air voids of less than 1.0%. Therefore, it was concluded that cutting the sample had no significant effect on the air void content of the sample.

The preparation of the samples requires that all samples are washed to remove any loose, fine material. Then the samples are submerged in water in order to saturate the sample. Any remaining air in the test chamber is removed with the vacuum. The sample is then seated on the pedestal and the test chamber is placed around the sample. The upper cap is then placed on top of the sample and held tight with clamps. The membrane is then inflated to a pressure of 9.5 to 10.5 psi. The pressure is to remain within this range during all testing. The graduated cylinder is positioned in the upper cap and filled with water. The apparatus is then rocked to dislodge any trapped air in the upper cavity. The cylinder is then filled to the initial head level. Outlet pipe is then opened and the time measured for the initial head to reach a designated final head and recorded. For this study, an initial head value of 60 cm and a final head of 10 cm were used. Temperature of the water is also recorded during the testing procedure to account for the viscosity of water. The coefficient of permeability is then calculated using equation 4.1. This specification calls for at least three runs, however in this study, four runs were employed to calculate the permeability of the PATB material.

The results of the permeability testing for the cored I-20 material are shown in Table 4.3. The two cores tested had an average permeability of nearly 1,700 ft/day. This number is much greater than the recommended 1,000 ft/day criteria noted by Baumgartner (2001). Therefore, it can be concluded that even the I-20 material that had obvious aggregate breakdown could meet the requirement for water permeability for PATB layers.

Table 4.3. Permeability Results of Cored I-20 PATB.

Sample ID	Average % Air Voids	Number of Samples	Average Permeability (ft/day)	Standard Deviation
I-20 Cores	28.4	2	1,671	12.1

Preparation and Compaction of PATB Material in Gyratory Compactor.

The next task was to take the material taken from US 280 and I-20 in the buckets and make pills with percent air voids similar to that of the cores previously tested. The first challenge encountered was to establish an effective method to prepare this material for compaction. After exercising a number of trial and error scenarios, it was determined that the best method to prepare the PATB material was to place the material in an oven at 200°F overnight. Heating the material at this temperature for this duration yielded desirable outcomes. These outcomes included that the PATB was thoroughly heated, possessed excellent workability and no asphalt drain down was observed. The material was attempted to be compacted at 150°F, which is the maximum compaction temperature for ALDOT specifications, but it was found that the PATB material could not be handled at the designated temperature. Therefore it was necessary to heat the PATB to 200°F in order to handle, prepare, and compact the material.

Once the material was ready for compaction, 4,200 grams of PATB material was put in the heated molds for compaction. Eight samples were prepared for the initial compaction and preparation procedure. The higher number of samples was selected to investigate the variability of the PATB material. The results show that there was little variation between these samples and it was later found that fewer samples were needed because of the lower variability. Compaction was completed using the Pine Gyratory

Compactor, which is shown in Figure 4.7. Based on literature pertaining to the laboratory preparation and testing of similar open graded asphalt treated base material, (Maupin et al., 1997) 40 revolutions in the gyratory compactor was selected as the initial compaction effort. The pressure was set approximately at 600 kPa with an internal angle of 1.16°.

Figure 4.7. SUPERPAVE Gyratory Compactor Used in Laboratory Study.

Upon completion of the compaction effort and extraction of the sample from the mold, it was clear to see that 40 revolutions in the gyratory compactor was far too high to match the percent air voids of the pills and the cores taken from US 280. As shown in

Figure 4.8, the material from US-280 experienced a large amount of internal aggregate crushing after 40 revolutions in the gyratory compactor. It was then decided that a much lower compaction effort was needed to compact the PATB material so that it was more comparable to the cores obtained in the field.

Figure 4.8. Aggregate Crushing in PATB Pill after 40 Revolutions.

Perform height and volumetric analysis

The US-280 pills were then tested in the Corelok device to determine bulk specific gravity (ASTM D6752) of each pill and subsequently the percent air voids. Table 4.4 shows the results of this procedure. It should be noted that all of these samples were tested using the single bag method. The results were rather consistent. Six pills compacted at 40 revolutions had a percent air void content around 21%. It should also be noted that only the US-280 material was initially compacted to 40 revolutions in the gyratory compactor. These results led to the conclusion that a lower number of revolutions was required to match the percent air voids of the cores and laboratory compacted pills for the US-280 material. It should be noted that specimens #5 and #7 crumbled before they could be tested for bulk specific gravity.

Table 4.4. Air Voids for 40 Gyrations using US 280 PATB

Specimen	Bulk Specific Gravity (g/cm3)	% Air Voids
#1	2.141	21.6
#2	2.150	21.3
#3	2.141	21.6
#4	2.149	21.3
#6	2.157	21.0
#8	2.144	21.5
Average	2.147	21.4
Standard Deviation	0.006	0.224

Instead of employing a trial and error method to quantify the number of revolutions required to produce PATB pills with similar air void content to the cores from US 280, a method described by Roberts et al. (1996) was used to estimate the required number of revolutions. During each revolution, the compactor measures the height of the sample. Knowing this value and the diameter of the pill, a constant value of 150 mm, an estimated volume of the pill at each revolution could be calculated. With the weight and estimated volume known for each pill produced, a calculated bulk specific gravity was quantified as shown in Equation 4.2.

$$G_{mb\,CALC_i} = \frac{M}{\pi r^2 h_i} \tag{4.2}$$

Where: $G_{mb\,CALCi}$ = Calculated Bulk Specific Gravity at *i*th revolution

M = Mass of samples (g)

r = Radius of sample (mm) = 150 mm

h_i = Height of sample at *i*th revolution (mm)

i = 1 to n (number of revolutions)

Since the compacted pills produced were not perfect cylinders, a correction factor was calculated. The Gravity Suite software along with the maximum specific gravity value enabled the calculation of a corrected bulk specific gravity of the compacted pills. The correction factor was taken to be the result of the actual bulk specific gravity from Gravity Suite divided by the calculated bulk specific gravity from Equation 4.2 at the height of the last revolution.

$$C.F. = \frac{G_{mbACTUAL}}{G_{mbCALC_n}} \tag{4.3}$$

Where: C.F. = Correction Factor

$G_{mb\,ACTUAL}$ = Bulk Specific Gravity of Pill Determined by Gravity Suite

$G_{mb\,CALC\,n}$ = Bulk Specific Gravity from Eq. 4.2 at final revolution, n

Since a correction factor could be computed for each pill, a more representative value for the actual bulk specific gravity was calculated based on a given height at each revolution. Equation 4.4 was employed to estimate the percent air voids of each pill based on the height at a given revolution. A spreadsheet was created to calculate the results of these equations. This spreadsheet also easily displayed the change in percent air voids based on the number of revolutions in the gyratory compactor.

$$\%AirVoids_i = \left[\frac{1 - \left(\dfrac{C.F.*G_{mb\,CALCi}}{G_{mm}} \right)}{\gamma_{water}} \right] *100\% \qquad (4.4)$$

Where: $\%$ Air $Voids_i$ = Percent Air Voids at *i*th revolution

C.F. = Correction Factor

$G_{mb\ CALC\ i}$ = Calculated Bulk Specific Gravity at *i*th revolution

G_{mm} = Theoretical maximum bulk specific gravity

γ_{water} = Unit weight of water, 1.0 g/cm^3

This process of determining the percent air voids based on the number of gyrations was used on six prepared specimens that had undergone 40 revolutions in the gyratory compactor. Figure 4.9 is a plot of the percent air voids versus the number of gyrations for these six specimens. Since the same amount of material was used and the material was prepared in the same manner as mentioned, it was assumed that there would be very little variation in the plots. The plots in Figure 4.9 support this assumption. Furthermore, the results show that an estimated percent air voids of 30% is met near 5 revolutions in the gyratory compactor. Once again, the purpose of this process was to target the number of revolutions required to match the field compaction effort that yielded a 31.4 percent air void content for the cores taken from US 280.

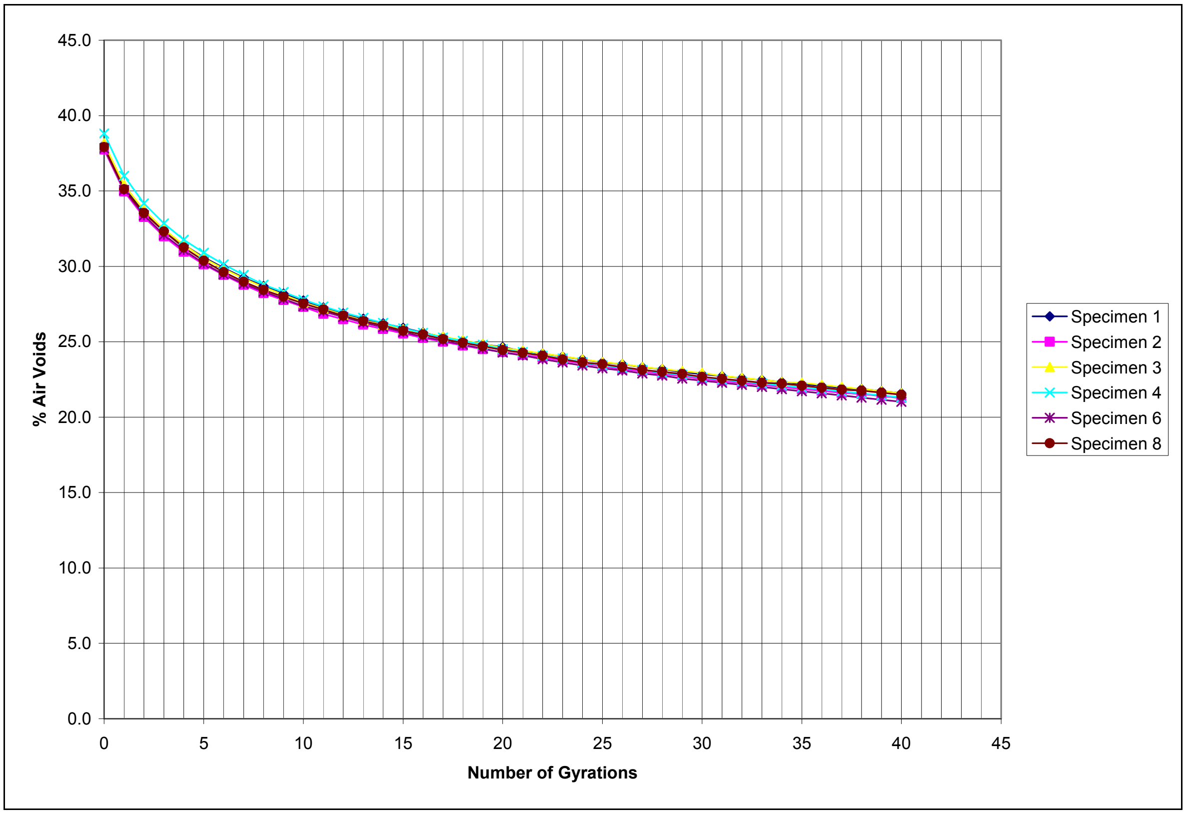

Figure 4.9. Percent Air Voids versus Number of Gyrations for 40 Revolutions (US 280 Material).

PATB material was then prepared for compaction and subjected to five revolutions in the gyratory compactor. The change in compaction effort yielded results quite different than the preliminary results. The first observation was that samples prepared and subjected to a mere 5 revolutions in the compactor required a longer cooling period before extraction. The pills cooled in the mold at room temperature for at least one hour. If the pill was not adequately cooled before extraction, the warm PATB material crumbled immediately. The second observation was that the aggregate experienced little if any crushing after five revolutions. Figure 4.10 shows one pill on the left after 5 revolutions and one pill, on the right, after 40 revolutions. The pill on the right has much more crushing of the aggregate and resembles the cores taken from US 280 much less than the pill on the left.

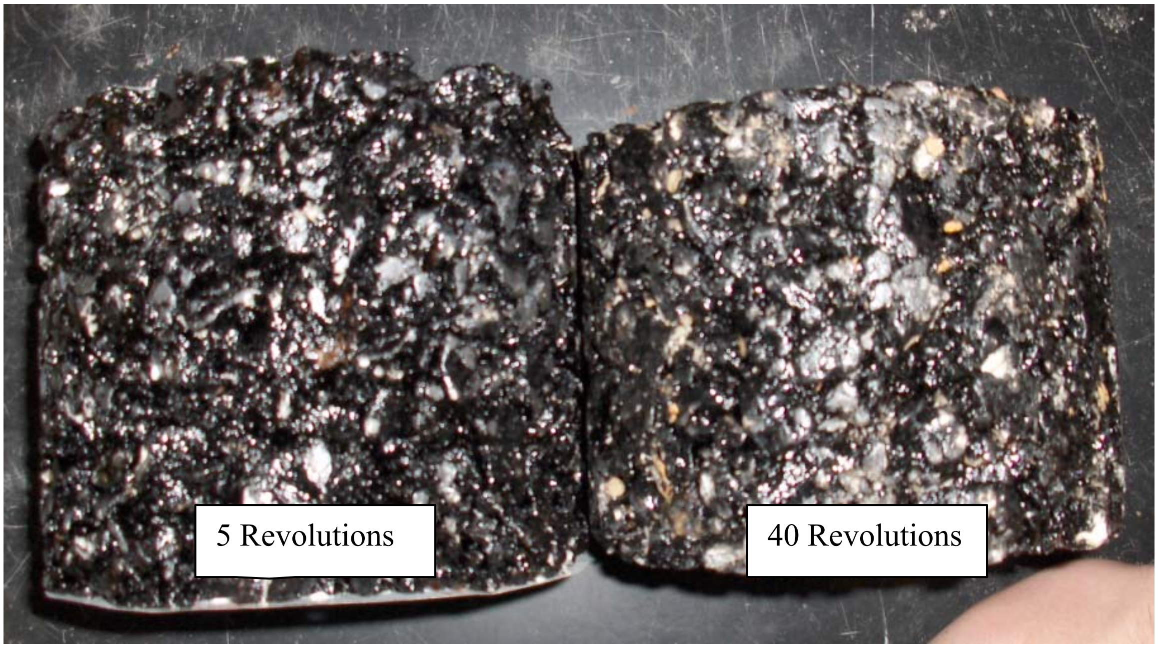

Figure 4.10. US 280 PATB Pills at 5 and 40 Revolutions.

The pills produced and compacted at five revolutions were then tested in the Corelok device to quantify the actual bulk specific gravity and then the percent air voids. Table 4.5 shows the results. As expected from observation, these pills were much closer to the cores taken from US 280. The pills subjected to five revolutions had an average percent air voids of 28%.

Table 4.5. Air Voids for 5 Gyrations using US 280 PATB.

Specimen	Bulk Specific Gravity (g/cm3)	% Air Voids
# 1	1.968	27.9
# 2	1.963	28.1
# 3	1.948	28.7
# 4	1.972	27.8
# 5	1.976	27.6
Average	1.965	28.0
Standard Deviation	0.011	0.397

The process of plotting the percent air voids versus the number of gyrations was performed on the pills subjected to five revolutions. Figure 4.11 shows that an estimated 2 revolutions would produce samples with a percent air voids closer to 30%. Two revolutions was selected as the target compaction effort for the following reasons. From Figure 4.9, it was assumed that 5 revolutions would provide enough compaction to produce pills with 30 percent air voids. However, the pills subjected to five revolutions had air void contents slightly lower than 30 percent (average of 28.0 percent). Therefore, these pills were then plotted for percent air voids versus compaction effort using the method presented by Roberts et al. (1996). Figure 4.11 shows that the target number of revolutions was three to produce pills with 30 percent air voids. Because five revolutions produced pills with slightly lower air void contents than 30 percent, it was decided that 2 revolutions would be a better compaction effort to account for the variability in the estimation of percent air voids. Once again the process was repeated and pills were produced after two revolutions in the gyratory compactor. Similar to those pills subject to five revolutions, the pills produced and subjected to only two revolutions in the gyratory compactor required cooling before extraction from the mold. It was also seen that very little to no aggregate crushing had occurred to the PATB material when subjected to the low compaction effort.

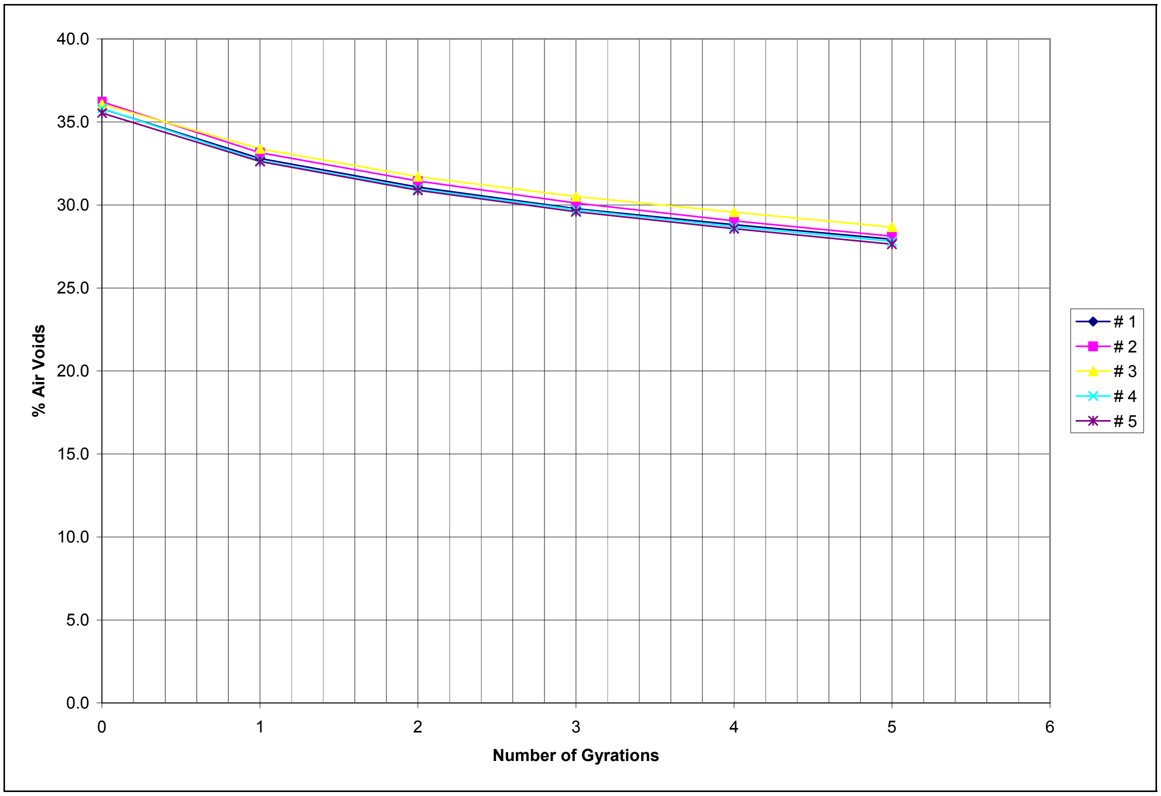

Figure 4.11. Percent Air Voids versus Number of Gyrations for 5 Revolutions (US 280 Material)

The pills subjected to two revolutions presented an issue for testing in the Corelok machine. These pills were also comprised of 4,200 grams of PATB material and since they were compacted with much less effort, they had a larger height than the previously prepared pills. The larger heights created a geometry that easily punctured the bags used by the Corelok to produce a vacuumed sample. By cutting the samples to a height of roughly 4 inches solved the Corelok issue. The bulk specific gravities and percent air voids of these samples subjected to two revolutions in the gyratory compactor are shown in Table 4.6. The values for percent air voids are very close to those values of percent air voids from the cores taken from US 280. Therefore, it was determined that the most representative samples are those prepared and subjected to two revolutions.

Table 4.6. Air Voids for 2 Gyrations using US 280 PATB.

Specimen	Bulk Specific Gravity (g/cm3)	% Air Voids
#1	1.897	30.5
#2	1.889	30.8
#3	1.879	31.2
Average	1.888	30.9
Standard Deviation	0.009	0.346

The ensuing step was to compact the plant mix I-20 PATB material to match field compaction. Figure 4.12 shows the percent air voids versus compaction effort for the I-20 material. Specimens 1, 2 and 3 were compacted at two revolutions. These samples all were tested to have air void contents near 35.0%. Specimens 4, 5 and 6 were compacted to five revolutions apiece. They yielded results that were around 33.0% air voids. Forty revolutions were required for the I-20 material to produce pills with air void contents near 28.0% as shown in Figure 4.13. The variability between the target number of revolutions to match field compaction effort for the US 280 and I-20 plant mixed PATB may be explained the difference in gradation, toughness of the aggregate and angularity of the aggregate particles. Differences in binder content and grade could also explain the variability. Since raw aggregate used for these construction sites was not obtained, the aforementioned explanations could not be thoroughly investigated. All in all, the most important goal, matching field compaction effort, was achieved for both materials.

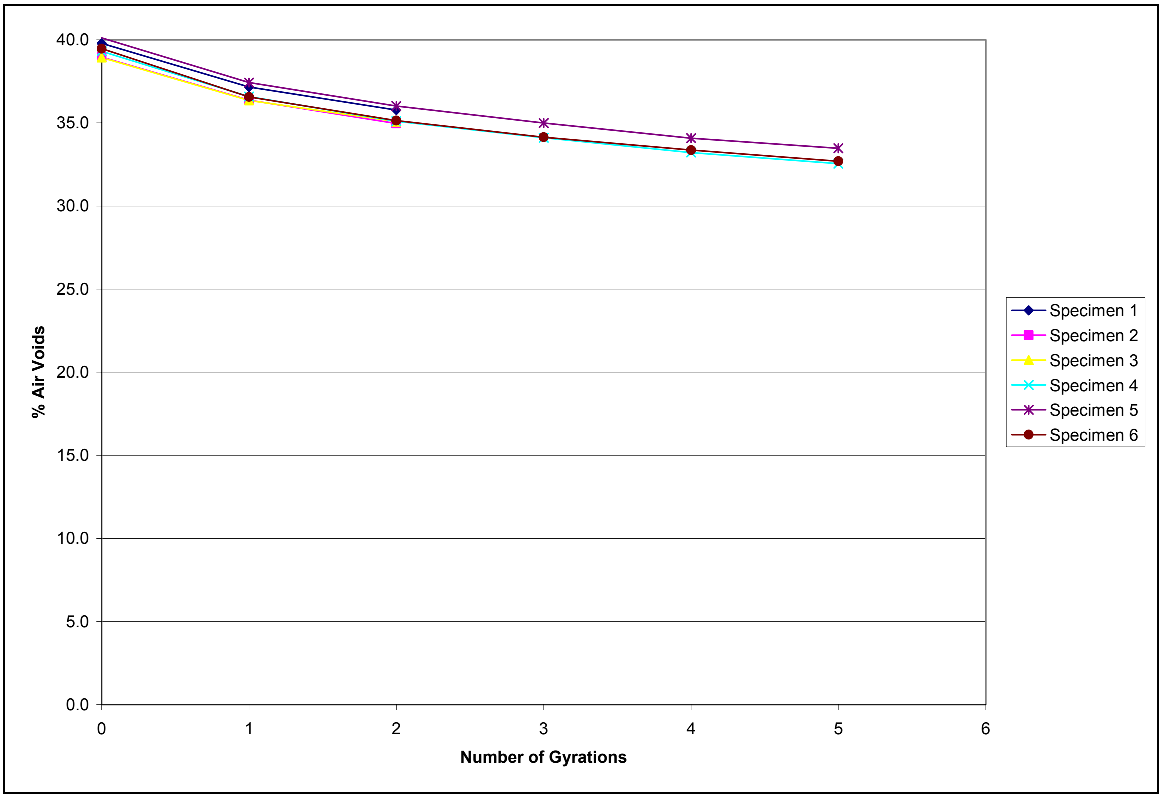

Figure 4.12. Percent Air Voids versus Low Compaction Effort (I-20 Material).

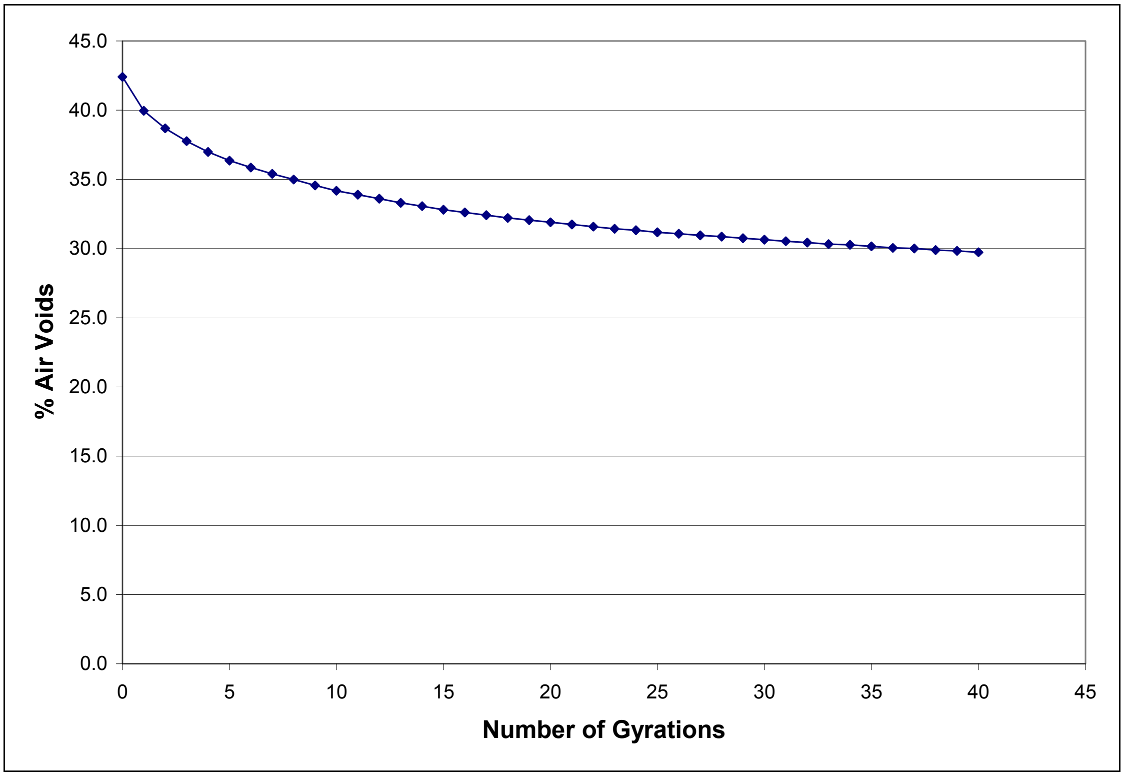

Figure 4.13. Percent Air Voids versus High Compaction Effort (I-20 Material).

Perform Tests on Laboratory Compacted Pills

Samples that were subjected to the various compaction efforts were then tested for permeability. Based on other literature, it was expected that those samples that were compacted at a higher number of revolutions with smaller percent air voids would have less permeability. Also, based on the US DOT Drainable Pavement Systems Demonstration Project, (1997) and Baumgartner (2002) a minimum permeability of 1,000 ft/day is required for permeable base layers. The results of the permeability testing are presented in Table 4.7.

Table 4.7. Permeability Testing Results for PATB Pills.

Material Type	Number of Revolutions	% Air Voids	Number of Samples	Average Permeability (ft/day)	Permeability Standard Deviation
US 280	40	21.4	3	1,205	29.5
US 280	5	28.0	5	1,735	37.1
US 280	2	30.9	3	1,850	24.3
I20	5	33.1	2	1,846	47.3

As expected, the samples that were subjected to two revolutions in the gyratory compactor had the highest average permeability. The lower compaction effort produces pills with higher air voids, which in turn increases permeability. Figure 4.14 shows a scatterplot of the permeability results versus percent air voids for the US 280 samples. The linear regression equation shows a very good fit for permeability versus air voids. The R-squared value of 93.84% means that most of the variability of the permeability is accounted for by the variability in air void content.

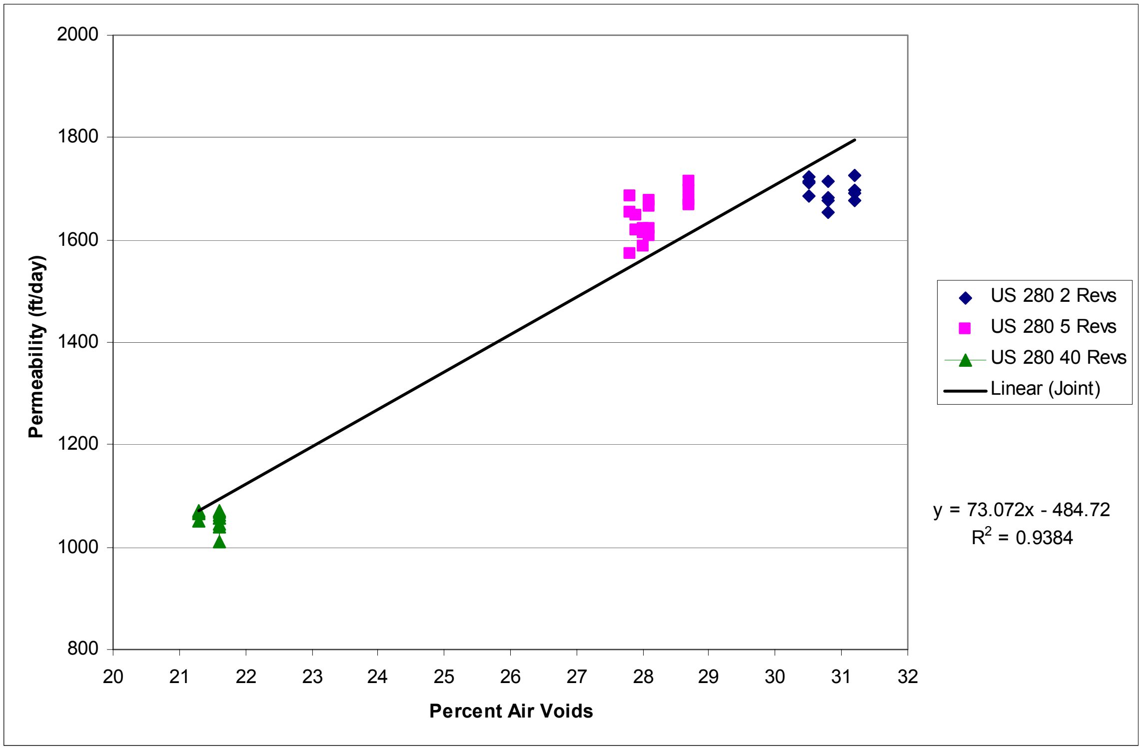

Figure 4.14. Permeability versus Percent Air Voids for US 280 Samples.

Based on the stability testing of the cores obtained from US 280 and I20 with polymer modified 76 – 22 binder, it was assumed that the prepared PATB pills would have little or no stability. Figure 4.15 shows the results of stability testing for PATB pills prepared in the laboratory for both US 280 and I20 material. The I20 pills failed at much lower stability and flow. The US 280 material failed in a similar manner to that of the cores, at a very high flow value with stability below 500 pounds. The two other US 280 pills that had air void contents near 30 disintegrated before stability testing could be performed.

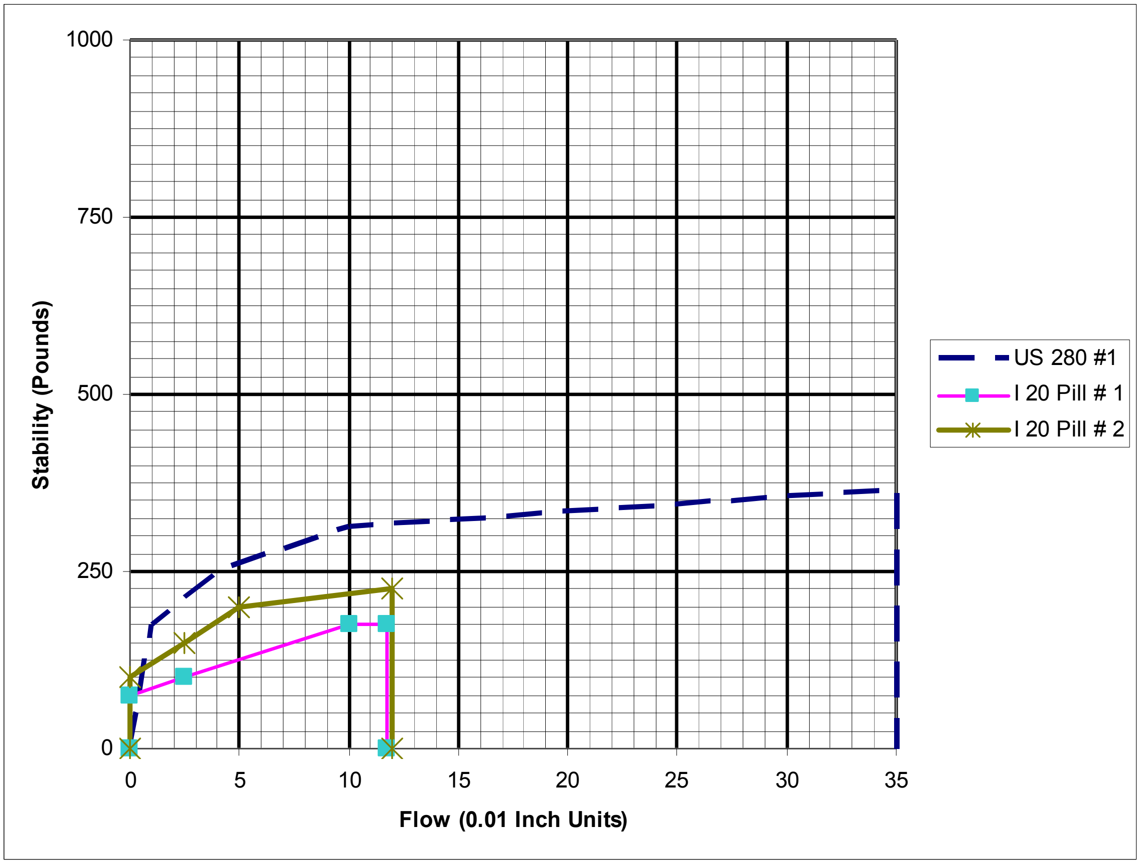

Figure 4.15. Stability Results for PATB Pills.

Analyze Aggregate Gradation

The US 280 and I-20 materials behaved very differently when subjected to compaction in the laboratory. The US 280 material experienced much more crushing of the aggregate than the I-20 material during compaction. It was decided to perform gradation analyses to investigate the following. First, it was desired to compare the I-20 material against the US 280 material before compaction. The compaction efforts (2 revolutions for US 280 and 40 revolutions for I-20) used to obtain the desirable air void content was drastically different even though the percent air voids of each material was rather close (31.4% for US 280 and 28.4% for I-20). The second observation of importance was to quantify how much aggregate breakdown occurred during compaction. The US-280 material had significant and visible aggregate breakdown when subjected to 40 revolutions in the gyratory compactor, whereas the I-20 material had very little.

Burndowns and sieving were performed on the virgin, or uncompacted, PATB material from both sites. The main purpose of the burndown procedures was to quantify the gradations of the mixes. Asphalt contents were not accurately reported since the correction factor for the burndown process was not known. However, if a correction factor of zero was assumed for all samples, then the asphalt content results ranged from 1.9% to 2.7%. Approximately 4000 grams of material was put in an oven and heated to 548°C until the asphalt binder was burned away from the aggregate. Figure 4.16 shows the aggregate gradation for both samples. As expected based on visual inspection, the I-20 material was slightly coarser than the US 280 PATB at 3/8 inch and number 4 sieve sizes, however it was finer at the 3/4 inch sieve. Both the I-20 and US280 PATB had similar amounts of fine particles.

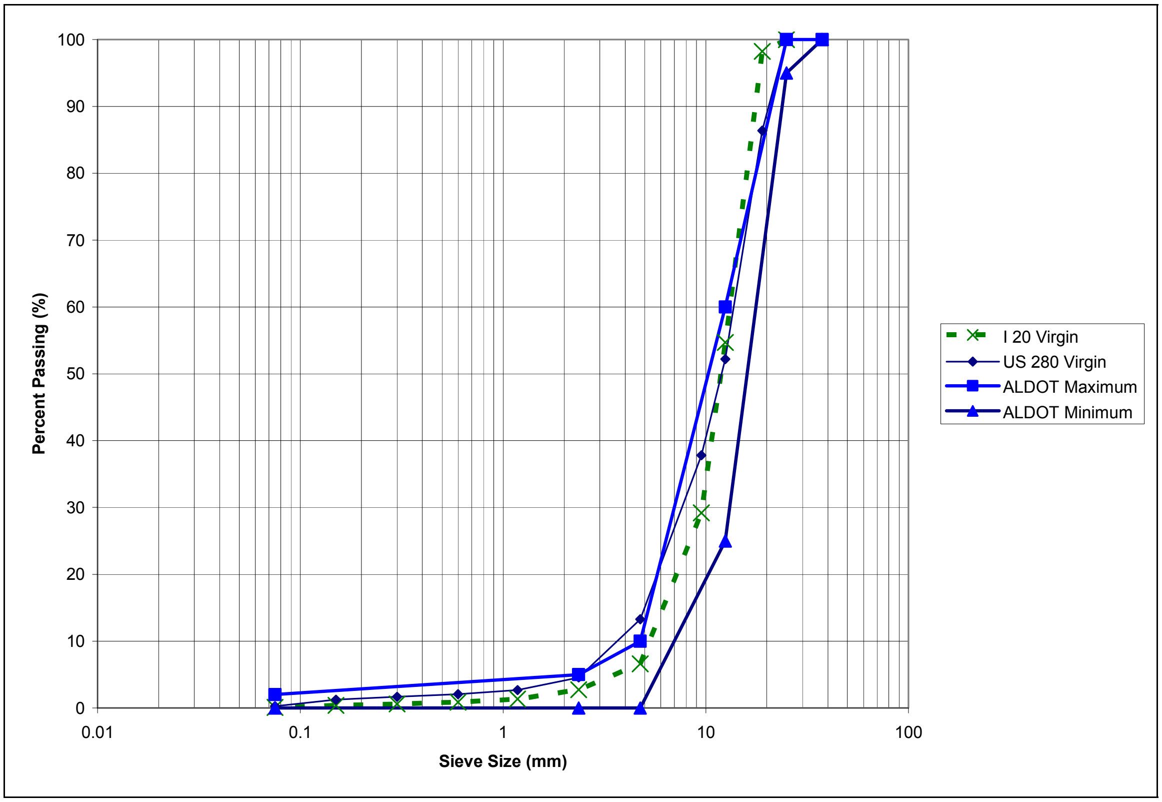

Figure 4.16. Aggregate Gradation for Virgin PATB Material.

Figure 4.17 shows aggregate gradation of virgin and compacted US 280 PATB material, as well as the gradation band for ALDOT PATB aggregate. It can be seen that the previously compacted aggregate particles were much finer than the virgin PATB material. Despite the crushing of the aggregate in the gyratory compactor, the resulting aggregate gradation falls mostly within the specified gradation band.

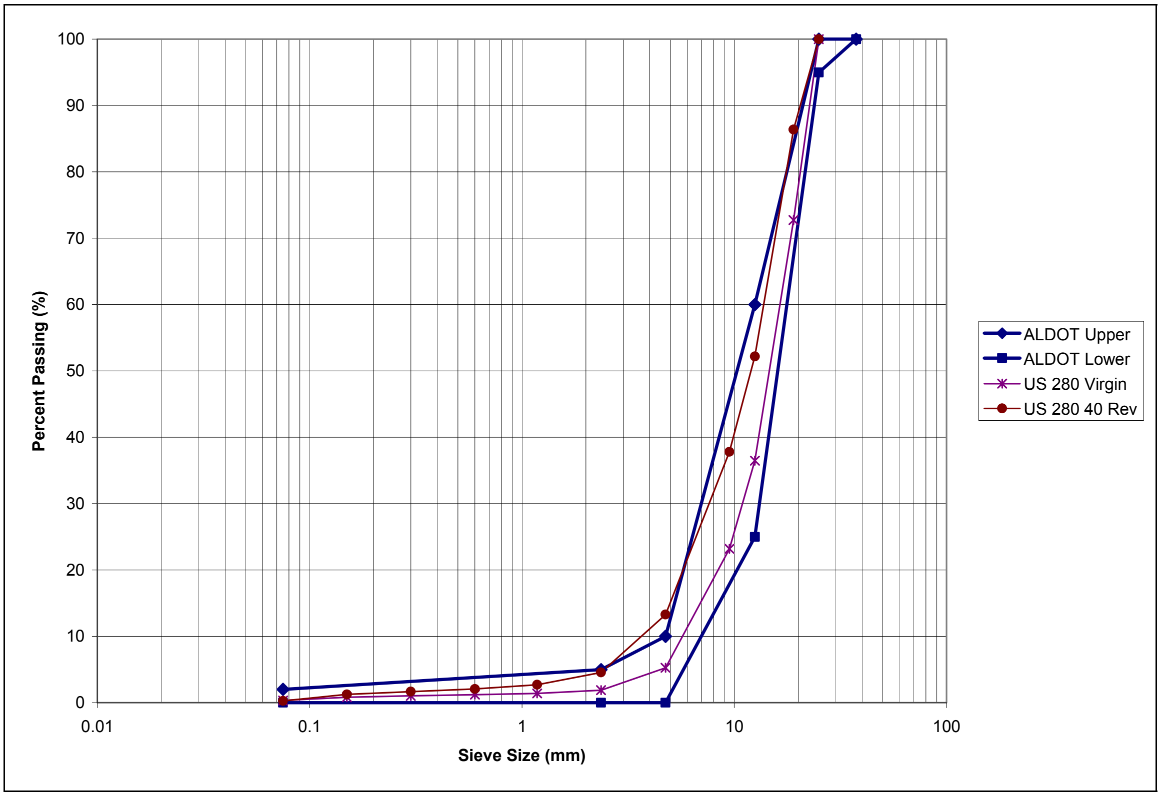

Figure 4.17. Gradation Comparison of Compacted and Virgin US 280 Aggregate.

The I-20 PATB material had much less visible aggregate breakdown than the US-280 PATB material upon gyratory compaction to 40 revolutions. Figure 4.18 shows the gradation for the virgin and compacted I-20 PATB material. The results show that some breakdown did occur, but it was not as much as the change in gradation of the US-280 material.

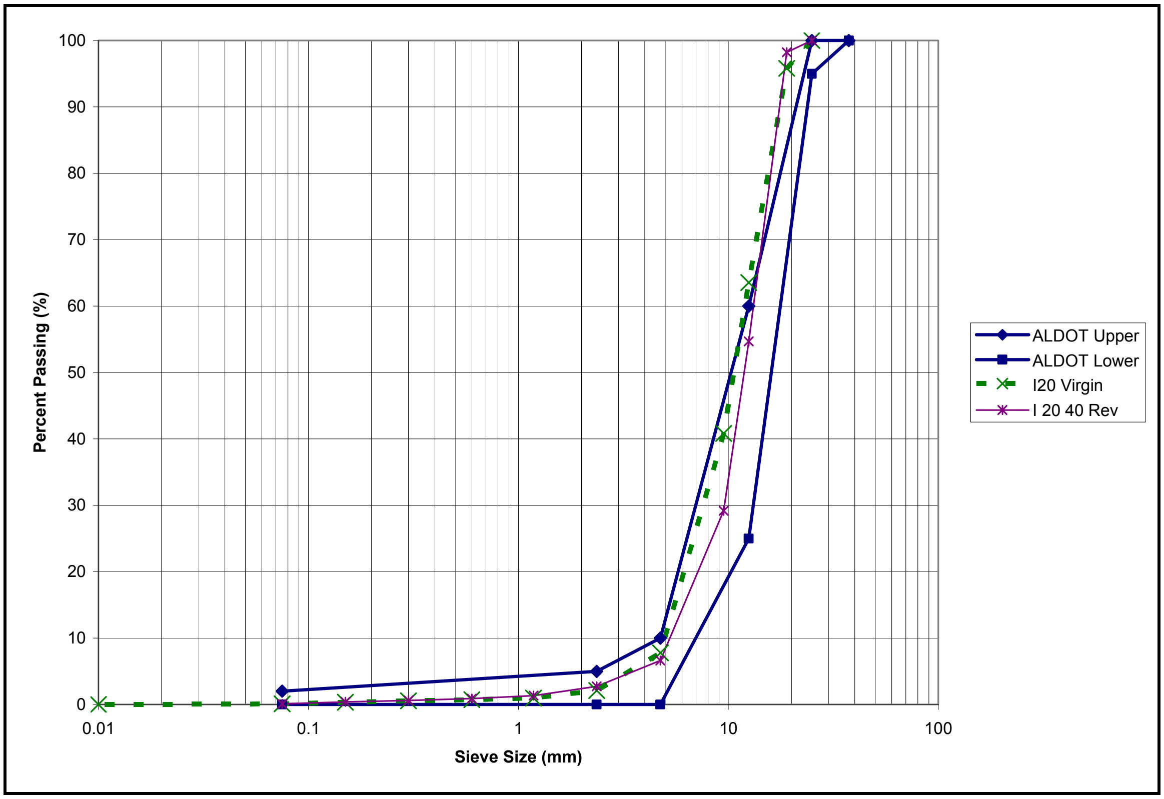

Figure 4.18. Gradation Comparison of Compacted and Virgin I-20 PATB Material.

Table 4.8 shows quantitative comparison of the change in gradation for both types of PATB material for all sieve sizes. The US 280 material underwent the most amount of crushing at the larger sieve sizes (3/4", 1/2", and 3/8"). This could be explained by the fact that the larger aggregates within the mixture are those that experience crushing during compaction. Considering that the I-20 virgin material had less aggregates retained on the 1/2 inch and 3/8 sieves, it could be expected that the I-20 material would have less aggregate breakdown. All in all, the gradations of the material taken from both sites are quite comparable, which is contrary to the initial assumptions based on visual inspection of the cores. The negative value for the I-20 material at the 3/4 inch sieve could be cause by segregation or the samples were non-representative.

Table 4.8. Breakdown Comparison of US 280 and I 20 PATB.

Sieve (Square Mesh Type)		Percent Passing by Weight		US 280 Breakdown	Percent Passing by Weight		I 20 Breakdown
		US 280 Virgin	US 280 40 Revs	% Pass 40 Revs - % Pass Virgin	I 20 Virgin	I 20 40 Revs	% Pass 40 Revs - % Pass Virgin
1 inch	25 mm	100.00	100.00	0.00	100.00	100.00	0.00
3/4 inch	19 mm	72.72	86.38	13.66	95.82	98.23	-2.41
1/2 inch	12.5 mm	36.49	52.20	15.71	63.55	54.69	8.86
3/8 inch	9.5 mm	23.18	37.81	14.63	40.81	29.19	11.62
No. 4	4.75 mm	5.26	13.30	8.03	7.77	6.61	1.16
No. 8	2.36 mm	1.89	4.56	2.67	2.10	2.74	-0.64
No. 16	1.18 mm	1.41	2.69	1.29	0.97	1.33	-0.36
No. 30	0.600 mm	1.22	2.06	0.84	0.74	0.88	-0.14
No. 50	0.300 mm	1.04	1.67	0.63	0.53	0.60	-0.06
No. 100	0.150 mm	0.83	1.26	0.42	0.33	0.41	-0.08
No. 200	0.075 mm	0.32	0.29	-0.04	0.09	0.09	0.00
Pan	0 mm	0.01	0.01	0.00	0.01	0.01	0.00

Phase I Summary

The investigation of the currently used PATB material based on the material obtained from the two sites in ALDOT Division 4 lead to many conclusions. First of all, the cores taken from the field had comparable percent air voids, despite the different appearances of the aggregate skeletons. The average percent air voids of the US-280 cores was 31%, whereas the average for the I-20 cores was around 28%. Both cores possessed very little to no stability upon testing, which supports the notion that PATB is inadequate in supporting construction traffic. Permeability of the cores was greater than the minimum value of 1,000 ft/day recommended by Baumgartner (1992) and the US DOT.

A viable method was devised to handle and prepare PATB specimens in the laboratory. Pills were produced consisting of I-20 and US-280 material that had similar air void contents, stability, permeability and aggregate gradation of the cores obtained from the respective sites. The number of gyrations required to meet the air voids of the cores taken from the field were quite different. The US-280 PATB required 2 revolutions to obtain an air void content of roughly 31%, whereas the I-20 PATB material needed 40 revolutions to reach an air void content of 28%. This variation can be attributed to the fact that the US-280 material was finer than the I-20 material. The finer aggregate gradation requires less compaction effort to achieve the same air void content. The pills prepared in the laboratory had very little to no stability and possessed adequate permeability characteristics. All of these results testify that a better mix design might be able to be devised in order to provide better stability, yet allow for sufficient permeability.

CHAPTER 5

PHASE II

ALTERNATIVE GRADATION OF PATB

The second phase of the methodology consists of two parts. The first portion was to mix raw aggregate and binder that are similar to the cores and pills produced in the previous section. The second portion of this phase was to investigate different mixtures that possibly could be substituted for the current PATB used by ALDOT. A number of different aggregate gradations at various asphalt contents were prepared and tested for air voids, stability, and permeability. Material was collected from East Alabama Paving's site in Opelika, Alabama. The material collected for research purpose included ALDOT number 57 and number 78 aggregate, as well as, PG 64-22 and polymer modified PG 76-22 asphalt binder. Table 5.1 shows the gradation for both Number 57 aggregate and number 78 aggregate used in this phase. The number 78 gradation is finer than the number 57 and has a smaller nominal maximum aggregate size. The two aggregate blends consisted of crushed limestone and possessed very little fines. The results of the second phase are presented in the ensuing chapter.

Table 5.1. Aggregate Gradation Table for Number 57 and Number 78 Stone.

	Percent Passing by Weight		Percent Passing by Weight	
Sieve Size (in)	ALDOT Number 57	ALDOT Number 78	Laboratory Number 57	Laboratory Number 78
1.5	100		100	
1	95 - 100		96	
3/4		100		100
1/2	25 - 60	90 - 100	38	92
3/8		40 - 75		48
#4	0 - 10	5 – 25	7	19
#8	0 - 5	0 - 15	3	12
#16		0 - 5		2

The procedure to prepare the samples was as follows. Aggregate was weighed to approximately 4,000 grams and dried in an oven overnight at a temperature of 350°F. Asphalt binder heated to a temperature of 325°F was added to the aggregate and mixed. The target effective asphalt content for most samples was 2.0%. The mixed samples were then cured at 200°F for duration of two hours. Samples were then compacted and cooled before extraction from the mold in order to reduce the chance of the sample disintegrating upon removal.

The first blend of aggregate and binder investigated was ALDOT number 57 limestone mixed with PG 64–22 asphalt binder. This mix was originally used in the fourth division, but undesirable shoving and rutting occurred quite frequently during construction. This led the division to substitute PG 76-22 modified asphalt binder for the PG 64-22 binder. Nine samples were produced and compacted according to the procedure described in the research methodology second phase. Table 5.2 shows the volumetric calculations for PATB samples comprised of number 57 stone and PG 64 -22 binder. The percent air voids for these samples was calculated based on a theoretical maximum specific gravity of 2.744 determined by AASHTO 209-99. The results show a trend that was not expected. As the compaction effort decreases, or as the number of revolutions in the compactor is reduced, the average percent air voids goes down. Further investigation of these results presented in Table 5.2 show much variability within the samples that had undergone similar compaction effort.

Table 5.2 Volumetric Calculations Number 57 Stone with PG 64-22 Binder.

Samples	Binder	% AC	Number of Revolutions	Bulk SG	% Air Voids	Average	Standard Deviation
J	PG 64-22	2.00%	40	1.957	28.7	28.5	0.781
K	PG 64-22	2.00%	40	1.986	27.6		
L	PG 64-22	2.00%	40	1.943	29.2		
M	PG 64-22	2.00%	25	2.018	26.5	27.6	1.018
N	PG 64-22	2.00%	25	1.963	28.5		
O	PG 64-22	2.00%	25	1.982	27.8		
P	PG 64-22	2.00%	20	1.981	27.8	27.6	1.057
Q	PG 64-22	2.00%	20	2.017	26.5		
R	PG 64-22	2.00%	20	1.960	28.6		

In order to explain this variability, the same procedure was applied to plot the estimated percent air voids versus compaction effort based on height data and measured bulk specific gravity. Figure 5.1 shows the estimated percent air voids versus the number of revolutions for the nine samples of number 57 stone and PG 64–22 binder. The plots are not as concise as those shown in the previous chapter. However, it can be seen that at approximately 20 revolutions the estimated percent air voids is around the target value of 30%.

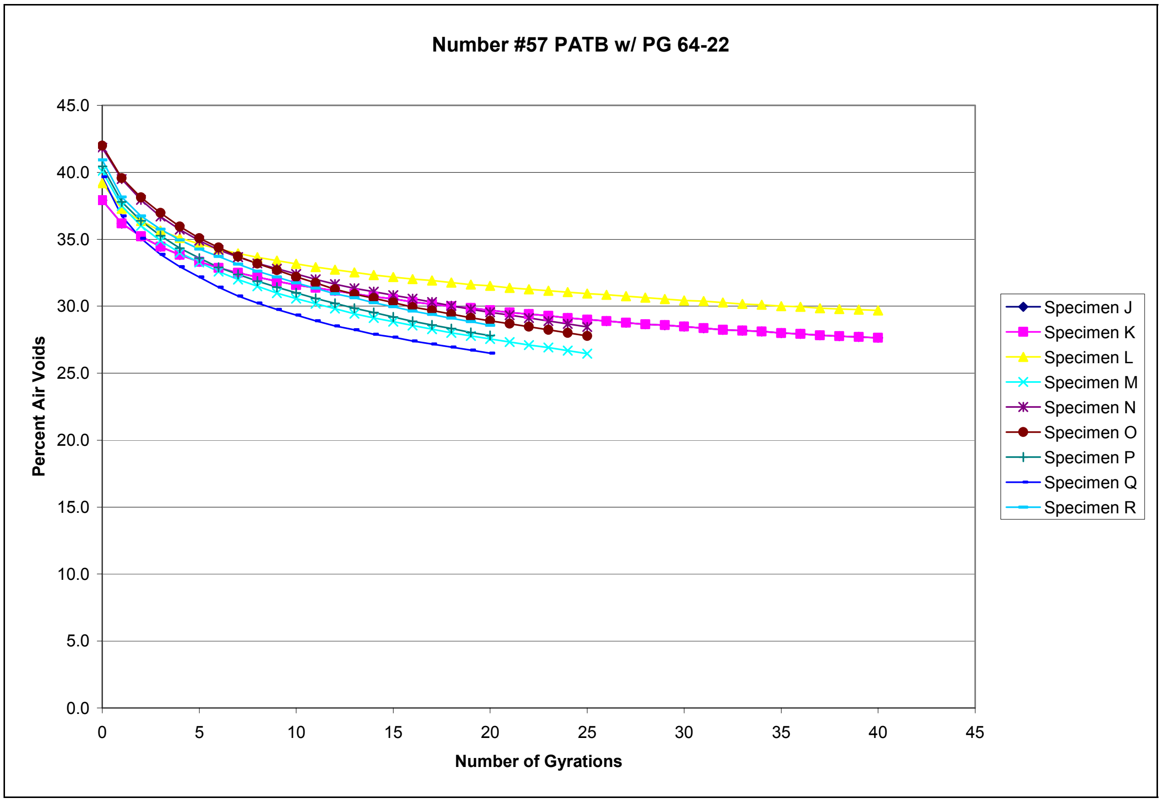

Figure 5.1. Percent Air Voids versus Compaction Effort for Number 57 Stone with PG 64 -22 Binder.

A further investigation of the data for the nine samples at the 20th revolution may better explain the variability in the results shown in Figure 5.1. It was thought that the variability of the estimated percent air voids between the samples at the 20th revolution could be explained by the changing geometry of the pill upon removal from the compaction mold. This changing geometry most likely would be a change in height after removal. This change in height would affect the validity of the correction factor used to predict the percent air voids during compaction. Table 5.3 shows the estimated bulk specific gravity and percent air voids based on the height at the 20th revolution. The other data presented is the calculated bulk specific gravity and percent air voids using a correction factor following the method presented by Roberts et al. (1996).

Table 5.3. Analysis of No. 57 Stone with PG 64-22 Binder at the 20th Revolution.

Sample	Bulk Specific Gravity (g/cm^3)	Estimated Percent Air Voids	Bulk Specific Gravity (g/cm^3)*	Estamted Percent Air Voids*
J	1.736	36.73	1.957	28.7
K	1.785	34.96	1.986	27.6
L	1.718	37.40	1.943	29.2
M	1.846	32.74	2.018	26.5
N	1.784	34.97	1.963	28.5
O	1.826	33.45	1.982	27.8
P	1.818	33.75	1.981	27.8
Q	1.837	33.07	2.017	26.5
R	1.766	35.66	1.960	28.6
Average	1.791	34.75	1.979	27.90
Standard Deviation	0.045	1.64	0.026	0.95

The data show that the bulk specific gravity and percent air voids calculated using the correction factor method has less variability than the bulk specific gravity and percent air voids calculated by height alone. This supports the use of the correction factor method as being an acceptable method to predict percent air voids during compaction. Taking the height data and calculating the bulk specific gravity based on mass compacted is expected not to be a very precise method due to the geometry of the pills produced. The PATB pills are not prefect cylinders with smooth surfaces, but rather, cylinders with rough surfaces due to the open graded aggregate used. Using a correction factor to calculate the bulk specific gravity accounts for this lack of smooth surfaces. This essentially reduces the variability of the results as shown in Table 5.3. The standard deviations reported for the correction factor method are less than the standard deviations reported for the height method to calculate bulk specific gravity and air voids.

Nine samples were produced consisting of number 57 limestone aggregate and PG 76 – 22 modified binder. Table 5.4 shows the volumetric data of these nine samples. It should be noted that the percent air voids was based on a theoretical maximum density of 2.742 for those samples with 2.5% asphalt content and 2.764 for those samples with 2.0% asphalt content by weight. Sample F crumbled upon removal from the compaction mold and could not be tested for bulk specific gravity, permeability or stability. The average percent air voids for those samples that had undergone similar compaction efforts had smaller standard deviations than those samples comprised of number 57 stone and PG 64 – 22 binder. Furthermore, as the total compaction effort decreased, the percent air voids within the samples increased. This trend was expected. The first six samples of number 57 stone with PG 76 - 22 modified binder were produced with 2.5% asphalt content by weight of mixture. These samples had much visible asphalt draindown, therefore the remaining samples were produced with 2.0% asphalt content by weight of mixture.

Table 5.4. Volumetric Calculations No. 57 Stone with PG 76 – 22 Modified Binder.

Samples	AC Type	% AC	Number of Revolutions	Bulk SG	% Air Voids	Average	Standard Deviation
A	PG 76-22	2.50%	40	1.979	27.8	27.7	0.185
B	PG 76-22	2.50%	40	1.981	27.7		
C	PG 76-22	2.50%	40	1.988	27.5		
D	PG 76-22	2.50%	20	1.888	31.1	30.3	1.203
E	PG 76-22	2.50%	20	1.935	29.4		
F	PG 76-22	2.50%	20	CRUMBLED			
G	PG 76-22	2.00%	20	1.912	30.8	30.4	0.370
H	PG 76-22	2.00%	20	1.929	30.2		
I	PG 76-22	2.00%	20	1.930	30.2		

Figure 5.2 shows the estimated percent air voids versus compaction effort for the eight samples of PATB with number 57 stone and PG 76–22 modified binder. The plots in Figure 5.2 show that there is considerably less variability between the samples when compared to the variability of those samples with number 57 stone and PG 64–22 binder. The other pertinent information from Figure 5.2 is that the number of gyration for the eight samples to reach 30% estimated air voids is around 20 revolutions. This number is the same as those samples with number 57 stone and PG 64–22 binder. The results show that for samples comprised of number 57 limestone aggregate, the number of revolutions required to attain 30% air voids is approximately 20. Likewise, it can be concluded that the type of asphalt binder used, PG 64–22 or PG 76–22 modified binder, is not a factor in determining the amount of revolutions required to achieve the desired air void content.

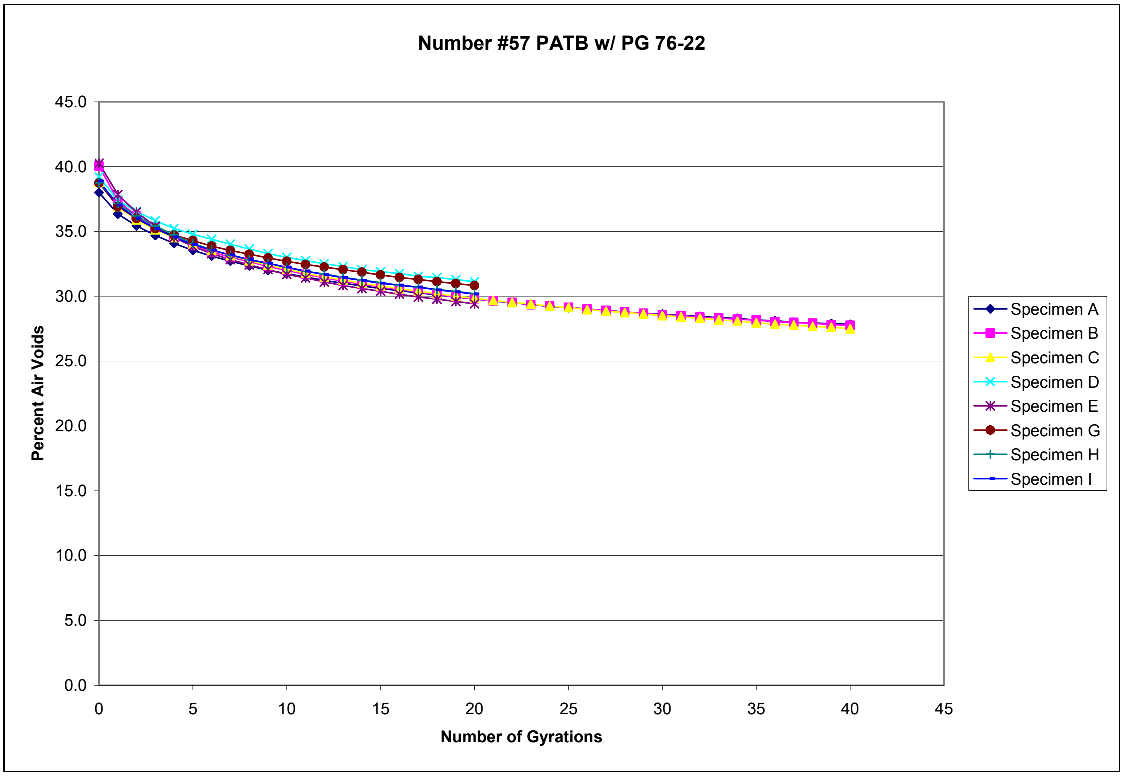

Figure 5.2. Percent Air Voids versus Compaction Effort for Number 57 Stone with PG 76 – 22 Modified Binder

The PATB pills with number 57 aggregate and the two types of asphalt binders were then tested for permeability. Again, these samples were tested in accordance with Florida's test method for permeability of compacted asphalt mixtures, designation FM 5-565. Four runs were performed on each sample. The average of the four runs for each sample and the respective standard deviation is shown in Table 5.5. Recall that sample F had crumbled before testing of bulk specific gravity. Samples N, O, and R, which were number 57 aggregate with PG 64 – 22 binder, crumbled before permeability testing could be conducted. In general, the results are similar to those obtained in the previous chapter. The permeability coefficients on average ranged between 1,500 ft/day and 1,650 ft/day.

Table 5.5 Permeability Testing Results for PATB with Number 57 Stone.

Samples*	AC Type	% AC	% Air Voids	Permeability (ft/day)	Standard Deviation
A	PG 76-22	2.50%	27.8	1544	29.84
B	PG 76-22	2.50%	27.7	1563	15.77
C	PG 76-22	2.50%	27.5	1513	60.02
D	PG 76-22	2.50%	31.1	1648	31.70
E	PG 76-22	2.50%	29.4	1606	33.54
G	PG 76-22	2.00%	30.8	1647	30.36
H	PG 76-22	2.00%	30.2	1613	20.18
I	PG 76-22	2.00%	30.2	1510	31.57
J	PG 64-22	2.00%	28.7	1474	27.18
K	PG 64-22	2.00%	27.6	1423	10.57
L	PG 64-22	2.00%	29.2	1638	17.09
M	PG 64-22	2.00%	26.5	1625	8.26
P	PG 64-22	2.00%	27.8	1677	16.96
Q	PG 64-22	2.00%	26.5	1618	14.58

*Samples F, N, O, and R crumbled before permeability testing was performed

Figure 5.3 shows the permeability coefficient plotted against percent air voids for the samples. The error bars shown represent ±1 standard deviation from the average of four runs for each sample. In general, the PATB samples with number 57 aggregate and PG 76-22 modified binder have a better trend in showing that permeability increases as the percent air voids increase (R squared = 48.61%). The samples with PG 64-22 binder are a little more scattered and do not show a definite trend (R squared = 3.13%). All in all, it can be said that the samples have permeability comparable to those samples produce with plant mixed PATB and meet the requirements for a minimum permeability of 1,000 ft/day recommended by the FHWA (1996).

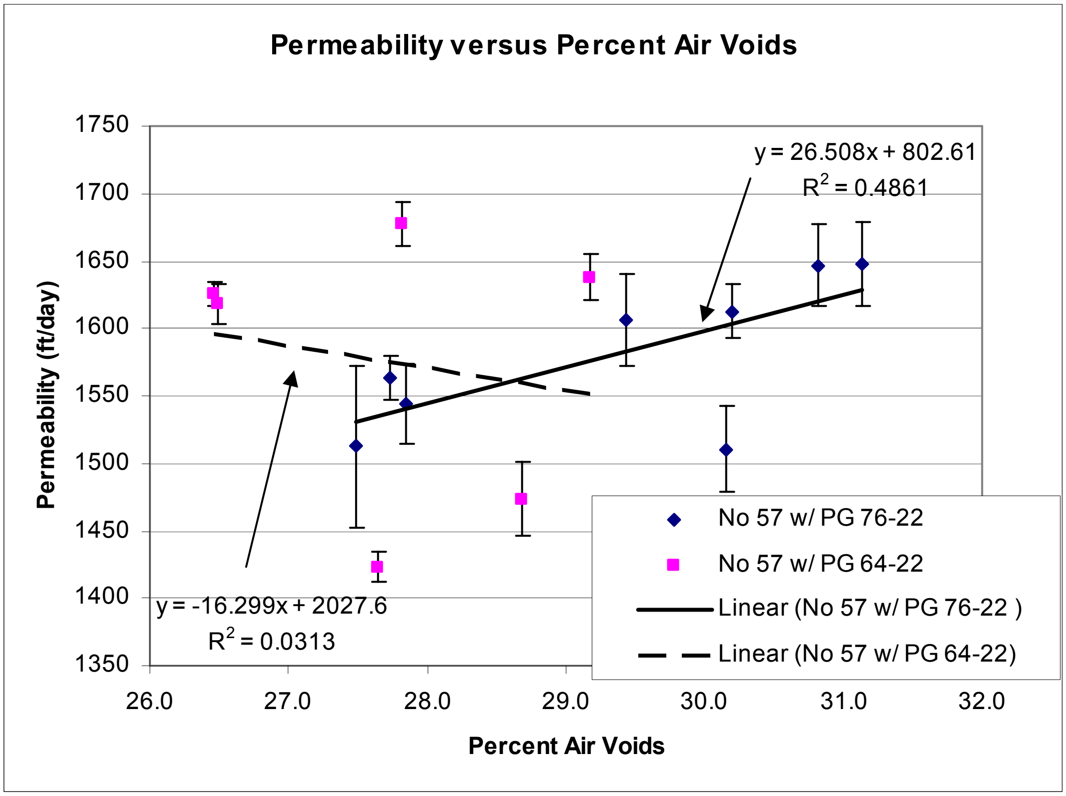

Figure 5.3. Permeability versus Percent Air Voids for PATB with Number 57 Stone.

The next step was to test the number 57 aggregate PATB samples for stability in accordance to AASHTO T245. Of the nine samples prepared with number 57 aggregate with PG 64 -22, only three actually could be tested for stability. Recall that three of the nine (Samples N, O, and R) had crumbled prior to permeability testing. Samples M, P, and Q crumbled while setting in the 60°C bath. This leads to the conclusion that the PATB consisting of number 57 limestone aggregate with PG 64 – 22 binder that was subjected to lower compaction effort (20 or 25 revolutions) in not a very suitable engineering material. Those samples that were subjected to 40 revolutions were tested for stability. Table 5.6 shows the stability testing results. The stiffness index is calculated as the maximum stability divided by the flow measured at maximum stability (Brown, 2006). The results indicate that the three samples had stability characteristics similar to the results of the previous chapter. They did not possess very much stability and had little resistance to flow. It should be noted that the maximum flow allowed during testing is 35 in/in. Those samples whose reported flow at maximum stability was 35 in/in actually reached the limit of the Marshall apparatus during testing.

Table 5.6. Stability Testing Results for Number 57 PATB with PG 64 – 22 Binder.

Samples	AC Type	% AC	Revolutions	% Air Voids	Maximum Stability (lbs)	Flow @ Max Stab (in/in)	Stiffness Index
J	PG 64-22	2.00%	40	28.7	700	35	20.00
K	PG 64-22	2.00%	40	27.6	800	35	22.86
L	PG 64-22	2.00%	40	29.2	250	35	7.14

Table 5.7 shows the stability testing results for the samples consisting of number 57 aggregate and PG 76-22 modified binder. It was expected that samples with modified binder would have higher maximum stability values than those with unmodified binder. It was also expected that the samples with higher asphalt cement content would have higher stability as well. The results for samples A, B, and C show that the samples of number 57 aggregate with modified binder and a slightly higher asphalt cement content have higher maximum stability values than those samples with unmodified binder subjected to the same amount of compaction. The samples subjected to 40 revolutions with PG 64 – 22 binder had maximum stability values that ranged between 250 and 800 pounds. The samples with PG 76 – 22 modified binder subjected to 40 revolutions with 0.5% by weight more asphalt cement had maximum stability values that ranged from 750 pounds to 1,100 pounds. All in all, these results were expected.

The results shown in Table 5.7 also show that the samples subjected to 20 revolutions had, in general, lower maximum stability values. Sample E, however, had results that fit better with the samples subjected to 40 revolutions. This could be explained by the fact that Sample E had an asphalt content of 2.5% by weight, which is slightly higher than the asphalt contents of Samples G, H, and I. Samples G, H, and I had much lower maximum stability, which in turn, results in lower stiffness index values.

Table 5.7. Stability Testing Results for Number 57 PATB with PG 76-22 Binder.

Samples*	AC Type	% AC	Revolutions	% Air Voids	Maximum Stability (lbs)	Flow @ Max Stab (in/in)	Stiffness Index
A	PG 76-22	2.50%	40	27.8	800	32.5	24.62
B	PG 76-22	2.50%	40	27.7	1100	35	31.43
C	PG 76-22	2.50%	40	27.5	750	35	21.43
E	PG 76-22	2.50%	20	29.4	950	35	27.14
G	PG 76-22	2.00%	20	30.8	450	35	12.86
H	PG 76-22	2.00%	20	30.2	575	35	16.43
I	PG 76-22	2.00%	20	30.2	625	35	17.86

* Sample D crumbled in hot bath

Number 78 aggregates were then used to produce PATB samples with both PG 64 – 22 and PG 76 – 22 modified asphalt binder. The asphalt content used was 2.0% to provide better comparison against the sample with number 57 aggregate and to minimize asphalt draindown. Table 5.8 shows the volumetric results for PATB with number 78 aggregate with 2.0% by weight PG 64 – 22 binder. Three compaction efforts were used; 40, 25 and 10 revolutions. Air void content was calculated based on the bulk specific gravity of each sample and a theoretical maximum specific of 2.691 for the mix. The results show that as the compaction effort is reduced, the average air void content increases. Furthermore, there is little variability between the samples that were subject to the same compaction effort.

Table 5.8. Volumetric Calculations Number 78 Stone with PG 64-22 Binder.

Samples	AC Type	% AC	Revolutions	Bulk SG	% Air Voids	Average	Standard Deviation
A2	PG 64-22	2.00%	40	2.025	24.7	24.3	0.409
B2	PG 64-22	2.00%	40	2.047	23.9		
C2	PG 64-22	2.00%	40	2.035	24.4		
D2	PG 64-22	2.00%	25	2.023	24.8	24.7	0.162
E2	PG 64-22	2.00%	25	2.031	24.5		
F2	PG 64-22	2.00%	25	2.027	24.7		
G2	PG 64-22	2.00%	10	1.943	27.8	27.3	0.481
H2	PG 64-22	2.00%	10	1.956	27.3		
I2	PG 64-22	2.00%	10	1.969	26.8		

Figure 5.4 shows the plots of estimated percent air voids versus the number of gyrations for the nine samples consisting of number 78 aggregate and PG 64 – 22 binder. Similar to the plots of samples with number 57 and PG 64 – 22 binder, there is easily observed variability between the samples as the compaction effort changes. It is also shown that, on average, roughly 10 revolutions are required to achieve 30% air voids. This is approximately half the compaction effort needed to produce samples with number 57 aggregate with a 30% air void content.

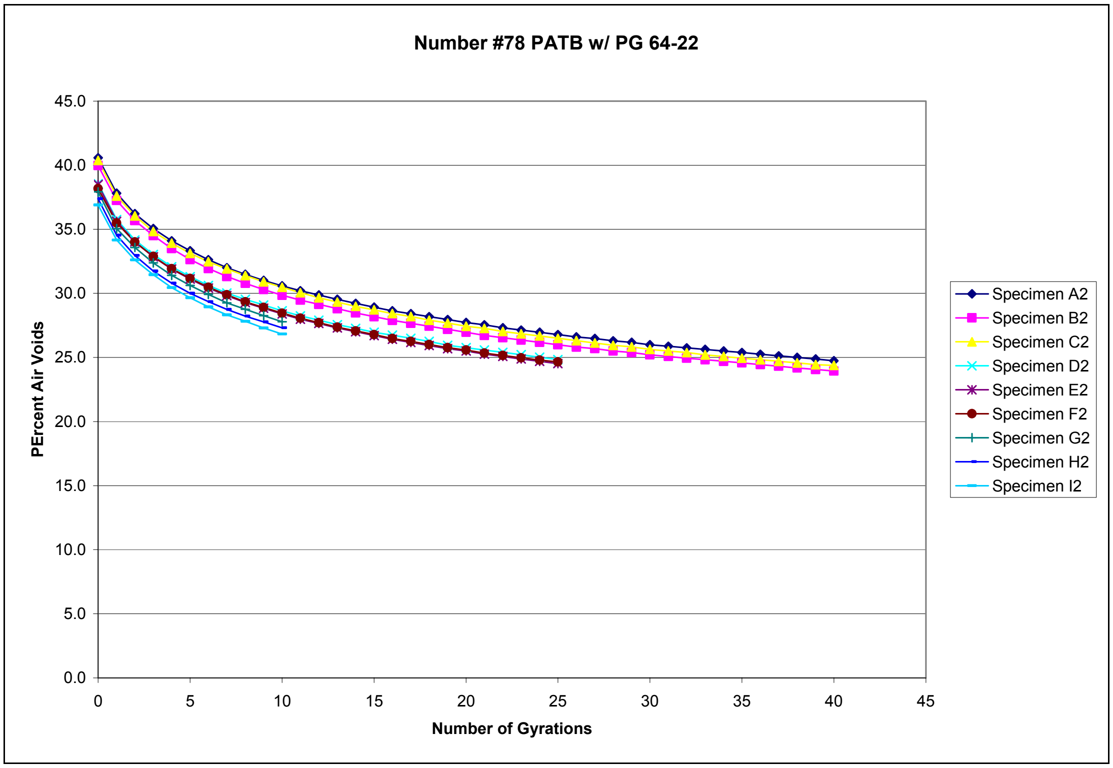

Figure 5.4. Percent Air Voids versus Compaction Effort for Number 78 Stone with PG 64 -22 Binder

Number 78 limestone aggregate was then mixed with polymer modified PG 76 -22 binder to investigate another possible mixture for PATB. Table 5.9 shows the volumetric results for the eight samples comprised of number 78 aggregate with PG 76-22 binder. Percent air voids was calculated based on a theoretical maximum specific density of 2.710. As expected, the data shows that as the compaction effort is reduced the average percent air voids increases.

Table 5.9. Volumetric Calculations Number 78 Stone with PG 76-22 Binder.

Samples	AC Type	% AC	Revolutions	Bulk SG	% Air Voids	Average	Standard Deviation
J2	PG 76-22	2.00%	40	2.072	23.6	24.6	0.957
K2	PG 76-22	2.00%	40	2.035	24.9		
L2	PG 76-22	2.00%	40	2.022	25.4		
M2	PG 76-22	2.00%	25	2.011	25.8	25.8	0.052
N2	PG 76-22	2.00%	25	2.012	25.8		
O2	PG 76-22	2.00%	25	2.013	25.7		
P2	PG 76-22	2.00%	10	1.914	29.4	29.8	0.555
Q2	PG 76-22	2.00%	10	1.893	30.2		

Figure 5.5 shows the plots of estimated percent air voids versus the compaction effort for the eight samples of 78 aggregate with PG 76-22 binder. Similar to the plots in Figure 5.4, the number 78 aggregate samples, on average, require 10 revolutions to achieve 30% air voids. The plots in Figure 5.5 do have less variability than those plots in Figure 5.4. This may be attributed to the fact that the samples with PG 76-22 modified binder set up better in the mold and held, more consistently, the shape of the pill.

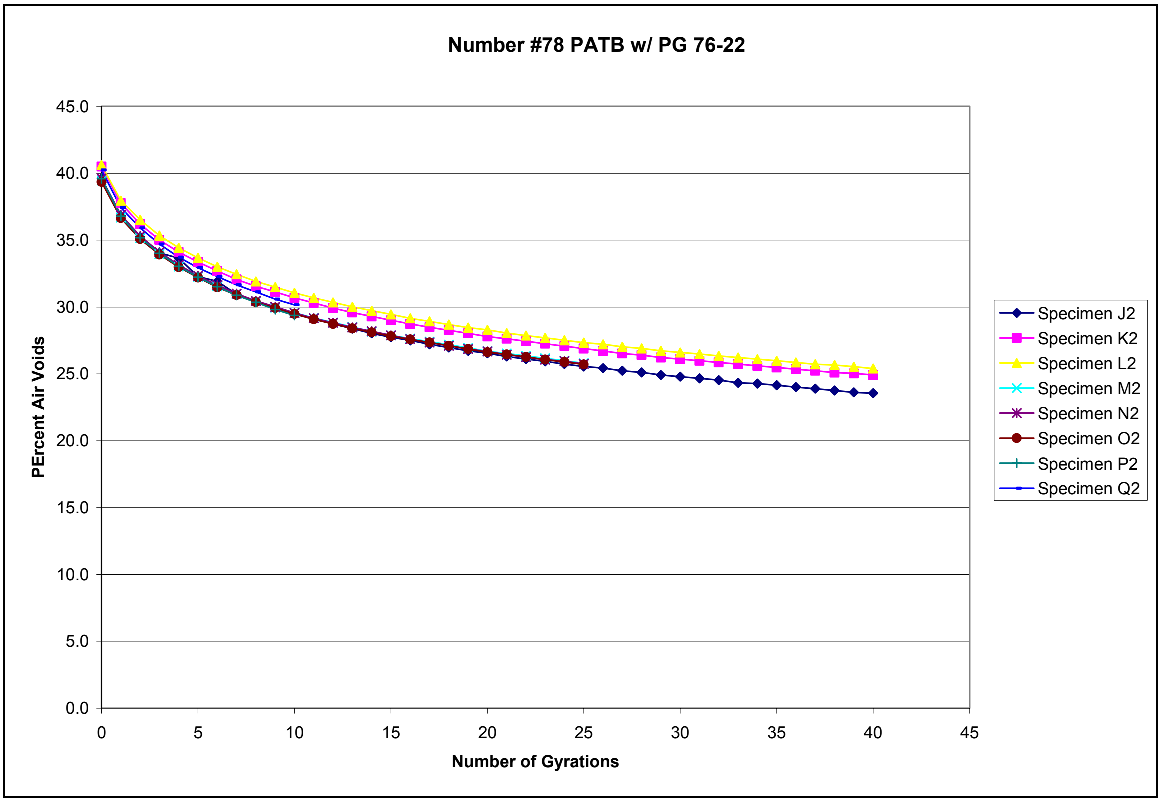

Figure 5.5. Percent Air Voids versus Compaction Effort for Number 78 Stone with PG 76 – 22 Modified Binder.

Permeability was then tested for the samples with number 78 aggregate. Testing was performed in accordance with Florida DOT's method for measurement of water permeability of compacted asphalt mixtures, designation FM 5-565. Four runs were performed on each pill. The average coefficient of permeability as well as the standard deviation were calculated and are shown in Table 5.10. Even though the percent air voids for the number 78 pills are very close to the percent air voids of the number 57 aggregate samples, it was expected that the number 78 aggregate samples would have less permeability. Recall from the equation presented by Lindy and Elsayed, (1995) that as the amount of aggregate in the mixture passing the number 8 increases, the permeability of the open graded mixture will decrease. The number 78 aggregate gradation permits up to 10% passing the number 8 sieve, whereas the number 57 allows up to 5%. Nonetheless, the results in Table 5.10 show that all samples average a permeability coefficient greater than 1,000 ft/day. Recall that a minimum of 1,000 ft/day is recommended for permeable base course recommended by the FHWA (1990) and by Baumgartner (1992).

Table 5.10. Permeability Testing Results for PATB with Number 78 Stone.

Samples	AC Type	% AC	% Air Voids	Permeability (ft/day)	Standard Deviation
A2	PG 64-22	2.00%	24.7	1245	18.52
B2	PG 64-22	2.00%	23.9	1126	9.08
C2	PG 64-22	2.00%	24.4	1211	4.70
D2	PG 64-22	2.00%	24.8	1221	30.59
E2	PG 64-22	2.00%	24.5	1127	36.97
F2	PG 64-22	2.00%	24.7	1173	15.59
G2	PG 64-22	2.00%	27.8	1349	14.68
H2	PG 64-22	2.00%	27.3	1322	17.80
I2	PG 64-22	2.00%	26.8	1242	27.05
J2	PG 76-22	2.00%	23.6	1015	27.73
K2	PG 76-22	2.00%	24.9	1156	18.87
L2	PG 76-22	2.00%	25.4	1122	9.16
M2	PG 76-22	2.00%	25.8	1160	6.95
N2	PG 76-22	2.00%	25.8	1030	3.72
O2	PG 76-22	2.00%	25.7	1145	14.09
P2	PG 76-22	2.00%	29.4	1383	14.74
Q2	PG 76-22	2.00%	30.2	1429	9.98

Figure 5.6 shows the average permeability versus percent air voids for the number 78 aggregate PATB samples. The error bars represent the standard deviations plus and

minus the average permeability value for each respective sample. In general, there is a visible relationship between permeability and percent air voids as demonstrated by the respective R squared values. As the percent air voids increases, the permeability increases as well. The samples prepared with PG 64-22 binder have a smaller range in average permeability than those prepared with modified PG 76-22 binder. The samples with PG 64-22 binder also do not have average permeability values below 1,100 ft/day. Whereas, two samples with PG 76-22 modified binder do have average permeability coefficients close to 1,000 ft/day. All in all, the most important information shown in Figure 5.6 is that not one of the samples prepared had permeability values less than 1,000 ft/day.

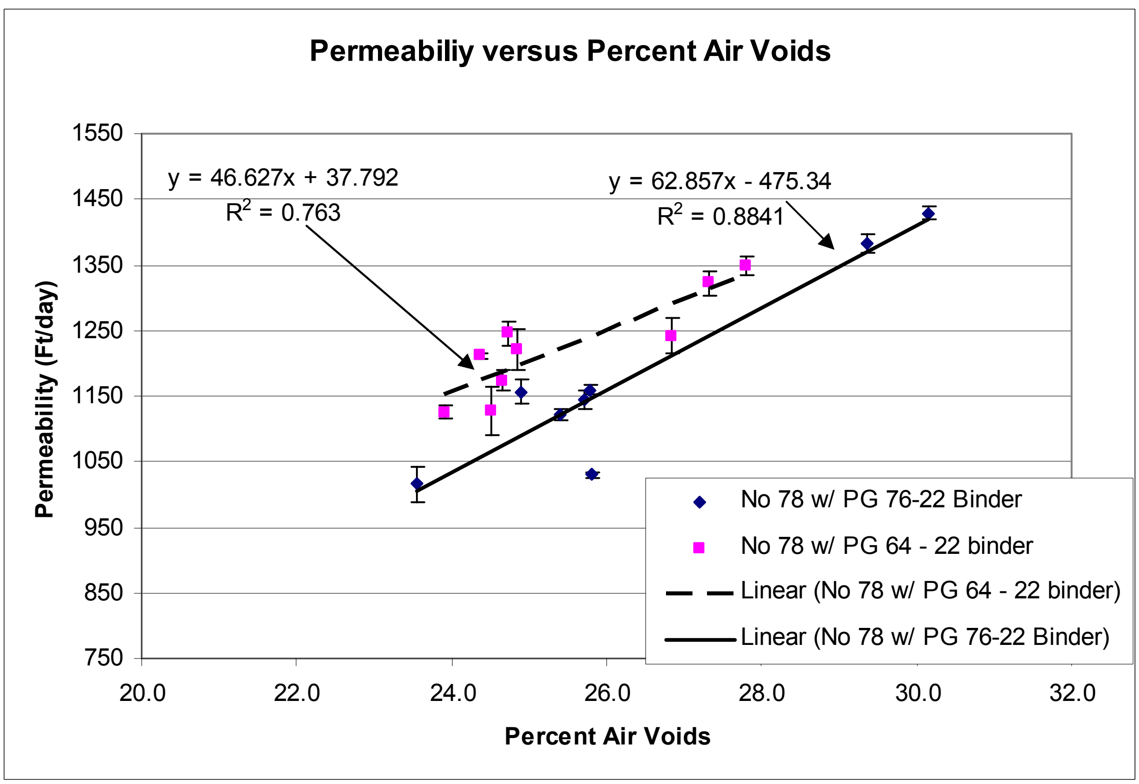

Figure 5.6. Permeability versus Percent Air Voids for PATB with Number 78 Stone.

Once permeability testing was completed, the samples were tested for stability in accordance to AASHTO T245. Again the samples were prepared for testing by heating in a warm bath at a temperature of 60°C for duration between 30 and 40 minutes. The samples were removed from the bath, loaded into the Marshall apparatus and subjected to load until failure. During the load application, stability was plotted against flow for each sample. Table 5.11 shows the maximum stability and flow at maximum stability for each sample prepared with number 78 aggregate and PG 64-22 binder. The stiffness index is also reported in Table 5.11 as is the quotient of maximum stability divided by flow at the point of maximum stability. The results show that all samples subjected to 40 or 25 revolutions in the gyratory compactor, the flow at which maximum stability was reached ranges from 15 to 20 in/in. This means that the stability curve did not resemble a straight line increasing in stability until maximum permissible flow was reached. These samples had maximum stability achieved at a much lower flow value and leveled off until maximum flow was reached. The maximum stability values are not very high (ranging from 525 to 800 pounds) and are very similar to many of the results for the samples prepared with number 57 aggregate. The values of the stiffness index, however, are much higher because maximum stability was reached at a lower flow. This leads one to believe that the number 78 aggregates could provide a more stable platform for construction traffic.

Those samples subjected to 10 revolutions did not have very good results. Considering that the asphalt content for all samples reported in Table 5.11 was held constant at 2.0%, it would be expected that those samples with higher air void content would have less stability. The results in Table 5.11 support this assumption. The samples subjected to 10 revolutions had higher average content and reported much lower stiffness index values than those samples subjected to more compaction effort.

Table 5.11. Stability Testing Results for Number 78 PATB with PG 64 – 22 Binder.

Samples	AC Type	% AC	Revolutions	% Air Voids	Maximum Stability (lbs)	Flow @ Max Stab (in/in)	Stiffness Index
A2	PG 64-22	2.00%	40	24.7	525	20	26.25
B2	PG 64-22	2.00%	40	23.9	800	20	40.00
C2	PG 64-22	2.00%	40	24.4	625	20	31.25
D2	PG 64-22	2.00%	25	24.8	800	15	53.33
E2	PG 64-22	2.00%	25	24.5	800	20	40.00
F2	PG 64-22	2.00%	25	24.7	625	20	31.25
G2	PG 64-22	2.00%	10	27.8	200	20	10.00
H2	PG 64-22	2.00%	10	27.3	300	35	8.57
I2	PG 64-22	2.00%	10	26.8	525	35	15.00

Table 5.12 shows the results of stability testing for those samples consisting of number 78 aggregate and PG 76-22 modified binder. These results show that these samples resemble number 57 aggregate PATB in the sense that maximum stability was reached at the maximum permissible flow. However, the values of maximum stability are much higher than those prepared with number 57 aggregate, which results in higher stiffness index values. The stiffness index values are rather comparable to those results in Table 5.13. Modifed binder has less resistance to flow during this testing procedure, but may possess better stiffness and resistance to rutting during construction. It is also clear to see that samples with higher percent air voids have much lower stiffness index values. Samples P2 and Q2 that were subjected to 10 revolutions had stiffness index values

around 10, which were much lower than any of the stiffness index values for the other samples.

Table 5.12. Stability Testing Results for Number 78 PATB with PG 76–22 Binder.

Samples	AC Type	% AC	Revolutions	% Air Voids	Maximum Stability (lbs)	Flow @ Max Stab (in/in)	Stiffness Index
J2	PG 76-22	2.00%	40	23.6	1450	25	58.00
K2	PG 76-22	2.00%	40	24.9	1350	35	38.57
L2	PG 76-22	2.00%	40	25.4	1450	35	41.43
M2	PG 76-22	2.00%	25	25.8	950	35	27.14
N2	PG 76-22	2.00%	25	25.8	1175	35	33.57
O2	PG 76-22	2.00%	25	25.7	950	35	27.14
P2	PG 76-22	2.00%	10	29.4	375	35	10.71
Q2	PG 76-22	2.00%	10	30.2	350	35	10.00

Since the asphalt content for all of the samples prepared with number 78 aggregate was held constant at 2.0%, it was possible to investigate the stiffness of the mixture versus percent air voids. Figure 5.7 shows the scatter plot of the stiffness index versus percent air voids. It is clear to see that for both types of mixtures used the stiffness of the samples decreases as the percent air voids increases; this trend is expected. As the amount, size, and interconnectivity of the air voids increases, the interlock between aggregates decreases resulting in lower strength or stability. From the data presented Figure 5.7, it can be said that by decreasing the target percent air voids in the sample from 30% to 25% yields in a drastic increase in stiffness and stability. This ultimately presents the underlying issue involved in the research. Air voids clearly dictate the permeability of the sample as shown in multiple graphs and seem to also have some influence in stability as shown by the regression equation as respective R squared values in Figure 5.7.

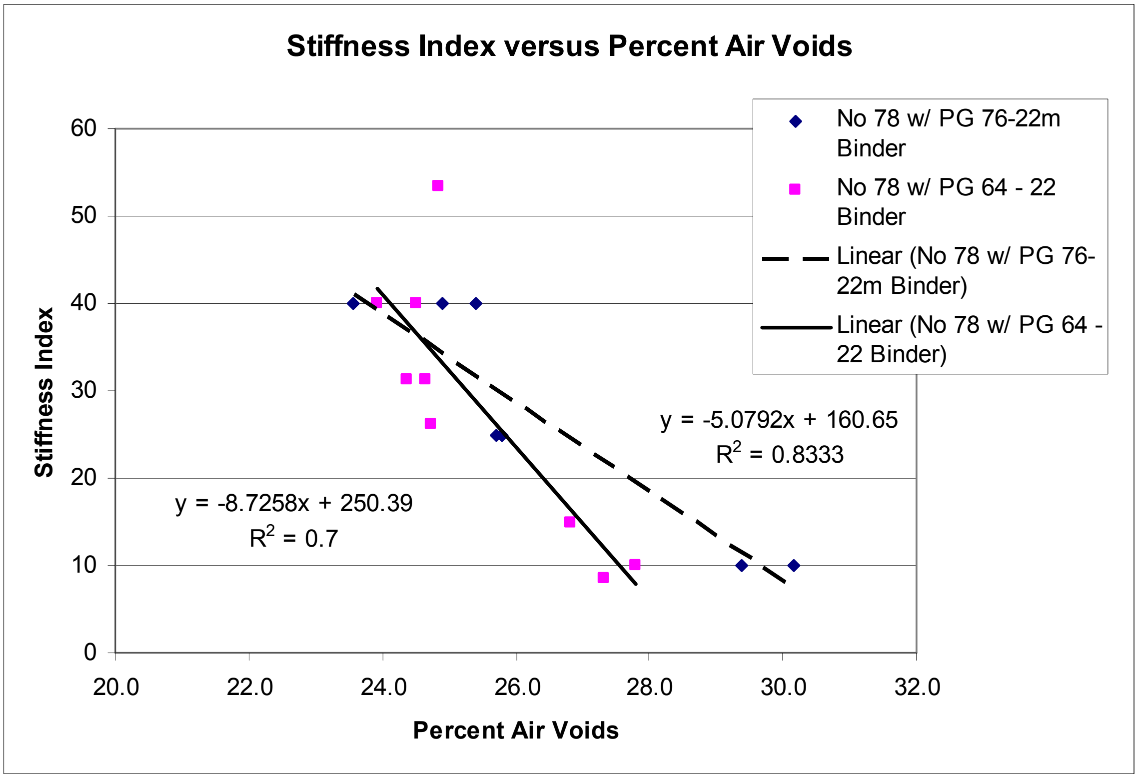

Figure 5.7. Stiffness Index versus Percent Air Voids for No. 78 Aggregate PATB.

Discussion of Results

A total of four HMA mixtures were tested in order to quantify the characteristics of current and possible PATB mixtures. Mixture 1 consisted of number 57 limestone aggregate with 2.0% PG 64-22 binder. Mixture 2 comprised of number 57 aggregate with 2.0% PG 76-22 polymer modified asphalt cement binder. Mixture 3 was a finer number 78 limestone aggregate mixed with 2.0% by weight PG 64-22 binder. The final mixture consisted of number 78 limestone aggregate with 2.0% PG 76-22 modified binder. All samples were produced with 2.0 asphalt content in order to minimize asphalt draindown and provide a consistent basis of comparison.

For each mixture type, multiple samples were compacted at three different compaction efforts. This was done in order to determine the necessary compaction effort required to achieve 30% air voids. Table 5.13 shows a compaction comparison for the four PATB mixtures. The first and most important statistic from the table is that the mixtures with number 57 aggregate required 20 revolutions to attain 30% air voids, whereas the samples comprised of number 78 aggregate only required 10 revolutions. It can be concluded that the aggregate gradation dictates the compaction effort needed and that the binder employed does not. Furthermore, PATB produced with number 78 aggregates would be easier to compact in the field because it requires less compaction effort. In general, the number 57 aggregates are much larger and harder to seat and compact, which creates shoving and rutting issues during construction. The finer number 78 aggregates, it is believed, would not have these adverse qualities during construction.

Table 5.13. Compaction Comparison of PATB Mixtures.

Mixtures	Aggregate Gradation	Binder	Target Compaction Effort (Revs)	Average % Air Voids	Standard Deviation (%)
1	No. 57	PG 64 - 22	20	27.6	1.057
2	No. 57	PG 76 - 22	20	30.3	0.659
3	No. 78	PG 64 - 22	10	27.3	0.481
4	No. 78	PG 76 - 22	10	29.8	0.555

The most important engineering property of PATB is the permeability of the layer. Recall that the FHWA (1990) recommends that an open graded asphalt treated base course should have a minimum coefficient of permeability of 1,000 ft/day. Mixtures

1 and 2, consisting of number 57 aggregate, had good permeability results that ranged from 1,425 ft/day to 1,675 ft/day. These calculated coefficients of permeability are very similar to the permeability coefficients of the PATB cores taken from Interstate 20. Mixtures 3 and 4 had average coefficients of permeability slightly lower than those of mixtures 1 and 2. This was expected since the skeleton of the number 78 aggregate PATB had more particles passing the number 8 sieve. This can also be explained by the composition of the air voids. Voids in the number 78 samples are smaller and probably less interconnected. Likewise, the voids in the number 57 aggregate samples are larger and probably more connected. This would provide larger and better connected escape routes for the entering water, which would yield higher permeability coefficients. Nonetheless, mixtures 3 and 4 are probably suitable in terms of permeability since their permeability values still exceeded 1,000 ft/day.

Figure 5.8 shows the plot of average permeability versus percent air voids for samples comprised of all four mixtures. It is clear to see that there is a relationship between permeability and air void content. The R squared value of 61.27% based on the raw data of the permeability testing means that there is a fairly decent relationship between permeability and percent air voids. As the percent air voids increases, the respective average permeability increases. These results support the observations presented in the literature review, supporting the fact that percent air voids have a major influence in the permeability of the compacted asphalt mixture.

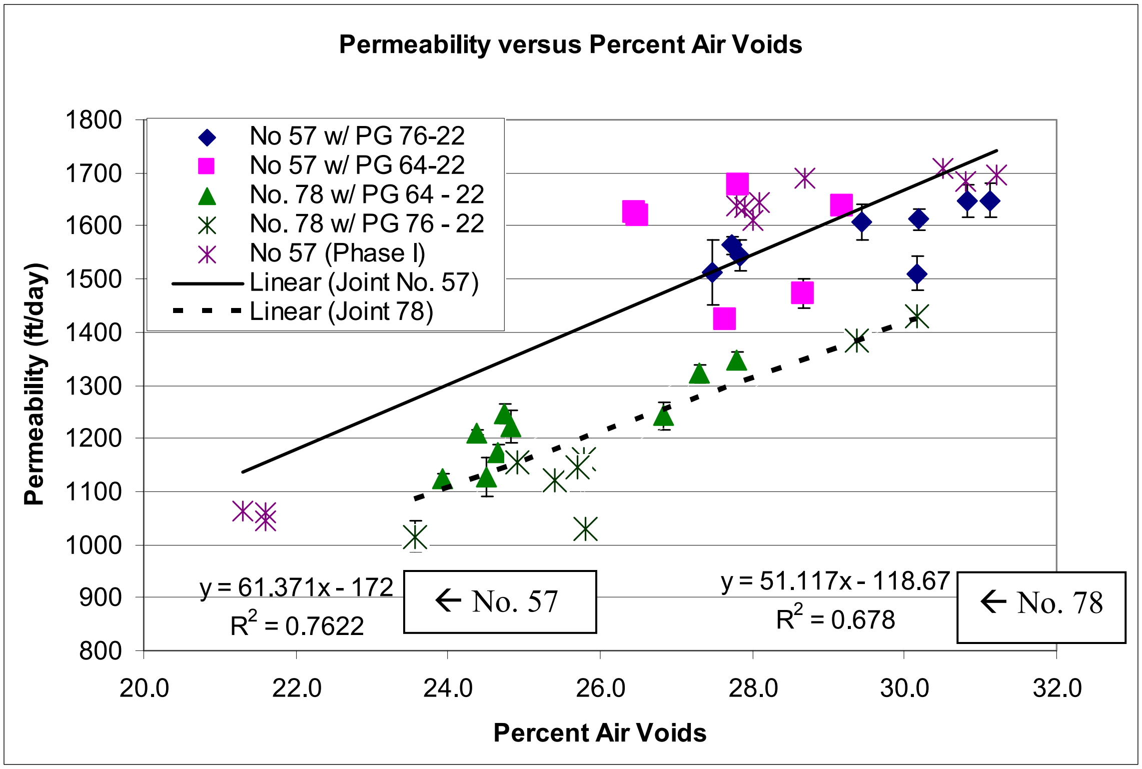

Figure 5.8. Permeability versus Percent Air Voids for Different PATB Mixtures.

The other engineering characteristic of interest in this research is stability. Stability of the PATB mixtures varies based on aggregate gradation, asphalt content and compaction effort. Compaction effort plays a role since it ultimately affects the air void content of the sample. The results are presented in Table 5.14. On average, the best options to produce the best stability are mixture 3 compacted to 25 revolutions and mixture 4 compacted to 40 revolutions, for they have the highest stiffness index values. Samples that were subjected to lower compaction efforts, 15 revolutions or less, had very small stiffness index values or crumbled before testing could be performed. These samples had higher air void contents, which in turn made the sample less durable and more susceptible to excessive flow.

Visual inspection of the compacted cores also provided some idea of its stability. Six of the nine samples produced with mixture 1 crumbled before stability testing could be performed. Samples consisting of mixtures 1 and 2 behaved like the cores taken from pavement and pills produced with plant mixed PATB, in the sense that they crumbled at room temperature, and had very little chance of surviving the hot bathing required for stability testing. None of the samples made with mixtures 3 and 4 crumbled at room temperature or in the hot bath. This fact simply leads to the conclusion that number 78 aggregate PATB is a better more suitable option as PATB material.

Table 5.14. Stability Comparison of PATB Mixtures.

Mixture	Aggregate Gradation	Binder	Compaction Effort	Average % Air Voids	Average Maximum Stability (lbs)	Avg. Flow @ Max. Stability (in/in)	Average Stiffness Index
1	57	PG 64 - 22	40	28.5	583.3	35.0	16.7
2	57	PG 76 - 22	40	27.7	883.3	34.2	25.8
2	57	PG 76 - 22	20	30.1	650.0	35.0	18.6
3	78	PG 64 - 22	40	24.3	650.0	20.0	32.5
3	78	PG 64 - 22	25	24.7	741.7	18.3	41.5
3	78	PG 64 - 22	10	27.3	341.7	30.0	11.2
4	78	PG 76 - 22	40	24.6	1416.7	31.7	46.0
4	78	PG 76 - 22	25	25.8	1025.0	35.0	29.3
4	78	PG 76 – 22	10	29.8	362.5	35.0	10.4

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

The first objective of this research was to devise a laboratory procedure in order to produce samples similar to the PATB produced in the field prior to the construction of subsequent layers. Through trial and error, plant mixed PATB heated to 200°F can be handled and compacted appropriately. Compaction with a gyratory compactor can produce samples similar to cores obtained from the field. Aggregate breakdown is an issue of concern, but compaction with the gyratory compactor creates similar amounts of aggregate breakdown that are observed in the field. It is important, however, to account for the imperfect cylinder geometry of the PATB samples when calculating the percent air voids during compaction by using a correction factor. Variability in measured air voids is not caused by removal of the samples from the mold.

The second objective of this research was to make modifications to the specified PATB material employed by ALDOT as a drainage layer. Four different mixtures were prepared and tested for air voids, permeability and stability. The results indicate that the ALDOT's 4th Division made a sound decision in changing the binder from PG 64–22 to a modified PG 76-22 binder. The PATB consisting of number 57 aggregate and PG 64–22 binder often crumbled upon removal from the compaction mold or in the hot bath required for stability testing. Those samples comprised of number 57 aggregate and modified PG 76–22 binder tested well for air voids and permeability, but did not have very good stability results.

It is concluded that PATB consisting of number 78 gradation limestone and PG 64–22 binder would be a viable option for ALDOT. Even though the results of permeability testing indicate that the number 78 aggregate PATB would have lower permeability, it appears that it is sufficient. When compared to the number 57 aggregate samples, the number 78 aggregate samples required half the compaction effort to achieve 30% air voids. This leads to the conclusion that number 78 PATB would be easier to compact in the field and have less issues with shoving and rutting.

Another positive point in selecting number 78 aggregate with PG 64–22 binder is the total material cost to ALDOT. Modified polymer asphalt costs more than the unmodified PG 64–22 binder. The number 78 aggregate also has a smaller nominal maximum aggregate size. This may allow the department to put down the PATB in two lifts of two inches instead of one lift of four inches. By using two lifts, the result may be a more stable and consistent material to support construction traffic paving the subsequent layers.

Although the laboratory results strongly recommend the use of number 78 gradation limestone with PG 64–22 binder, it is recommended that ALDOT construct a trial section for several reasons. The first reason is to see if construction can produce a drainage layer with the positive permeability and stability characteristics. This needs to be tested for both PATB layers constructed with one lift and PATB layers constructed with two lifts. Secondly, research needs to be performed concerning the performance of the PATB layer with time. In essence, ALDOT needs to know how the PATB layer with number 78 gradation limestone changes in time. Since the number 78 gradation has smaller air voids with less interconnectivity, it needs to be determined if clogging or densification will be a serious issue with time. If clogging becomes an issue, then the drainage layer cannot performed as designed and will ultimately be the source of many distresses related to water trapped within the pavement section.

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APPENDIX A

SUMMARY OF PATB CONSTRUCTION FROM OTHER STATE AGENCIES

Many states across the Southeast region of the United States include specifications regarding the construction of permeable asphalt treated base courses in their respective publications. The nomenclature of the PATB may vary from state to state. For example PATB is referred as Open Graded Asphalt Base Course in Arkansas. Other variations include Asphalt Drainage Course (Mississippi). The specifications in the following summary are from the specification literature published by the following states: Arkansas, Florida, Louisiana, Mississippi, and Tennessee.

The Arkansas State Highway and Transportation Department Specifications for Open Graded Asphalt Base Course

The Arkansas State Highway and Transportation Department (AHDT), similar to ALDOT, uses an open graded asphalt base course in order to facilitate the drainage of water through pavement structures. It consists of a mixture of crushed stone and asphalt binder. AHTD sets forth specifications for the liquid asphalt binder, aggregate, and construction practices that relate to the construction of the open graded drainage layer.

Asphalt Binder

The AHDT states that all asphalt binders must meet the requirements set forth in AASHTO M 320 Table 1. All requirements (flash point temperature, viscosity, dynamic shear, creep stiffness, etc..) must be met for the type of performance grade binder employed in the construction of the open graded drainage layer. The AHDT, however, drops the direct tension requirement for asphalt binders in its specifications. Similar to ALDOT, the AHDT mandates the use of heat – stable anti – stripping additive if stripping problems arise. The specifications detail that the additive is to be added at a rate of 0.5 to 0.75 percent by weight of the asphalt binder used and is to be added to the binder just before the liquid binder is added to the mix. The AHDT uses four different designs of open graded asphalt base course. The asphalt content range for the four design types range from 1.5 to 4.0 percent for Types I, II, and III and ranges from 2.5 to 3.0 percent for Type IV. The asphalt content is one of the two properties tested for acceptance during construction.

Aggregate

Gradation of the aggregate in the open graded base course is the second and final testing requirement necessary for acceptance. The aggregate gradation is dependent upon the open graded asphalt base course selected for construction. Table A1 presents the gradations, as well as, the asphalt content for the four design types. Figure 2 charts the different gradations.

It can be said that Type A1 contains mostly larger aggregates with 100 percent of the aggregates larger than the No. 4 sieve. Type IV gradation is most similar to that of the No. 57 stone gradation employed by ALDOT. This also holds truth for asphalt content. For these purposes, Type IV open graded asphalt base course will be compared to ALDOT's gradations and requirements. Type II gradation is most similar to that of Type I, for it mandates the use of larger aggregates, as much as 50 percent greater than ½ inch (12.5 mm). Type III is very comparable to Type IV in terms of aggregate gradation, but allows for a wider range of asphalt content, 1.5 – 4.0 percent compared to 2.5 – 3.0 percent.

Table A1. Gradations for AHDT Open Graded Asphalt Base Course

Requirements for AHDT Open Graded Asphalt Base Course					
Sieve (Square Mesh Type)		**% Passing by Weight**			
		Type 1	**Type 2**	**Type 3**	**Type 4**
3 inch	75 mm	100			
2.5 inch	63 mm	95 - 100			
2 inch	50 mm		100		
1.5 inch	37.5 mm	30 - 70	75 - 90		
1 inch	25 mm				100
3/4 inch	19 mm	0 - 15	50 - 70	100	90 - 100
1/2 inch	12.5 mm			90 - 100	
3/8 inch	9.5 mm	0 - 2			20 - 55
No. 4	4.75 mm		8 - 20	0 - 15	0 - 10
No. 8	2.36 mm			0 - 3	0 - 5
No. 100	0.150 mm		0 - 5		
Asphalt Content (%)		1.5 - 4.0	1.5 - 4.0	1.5 - 4.0	2.5 - 3.0

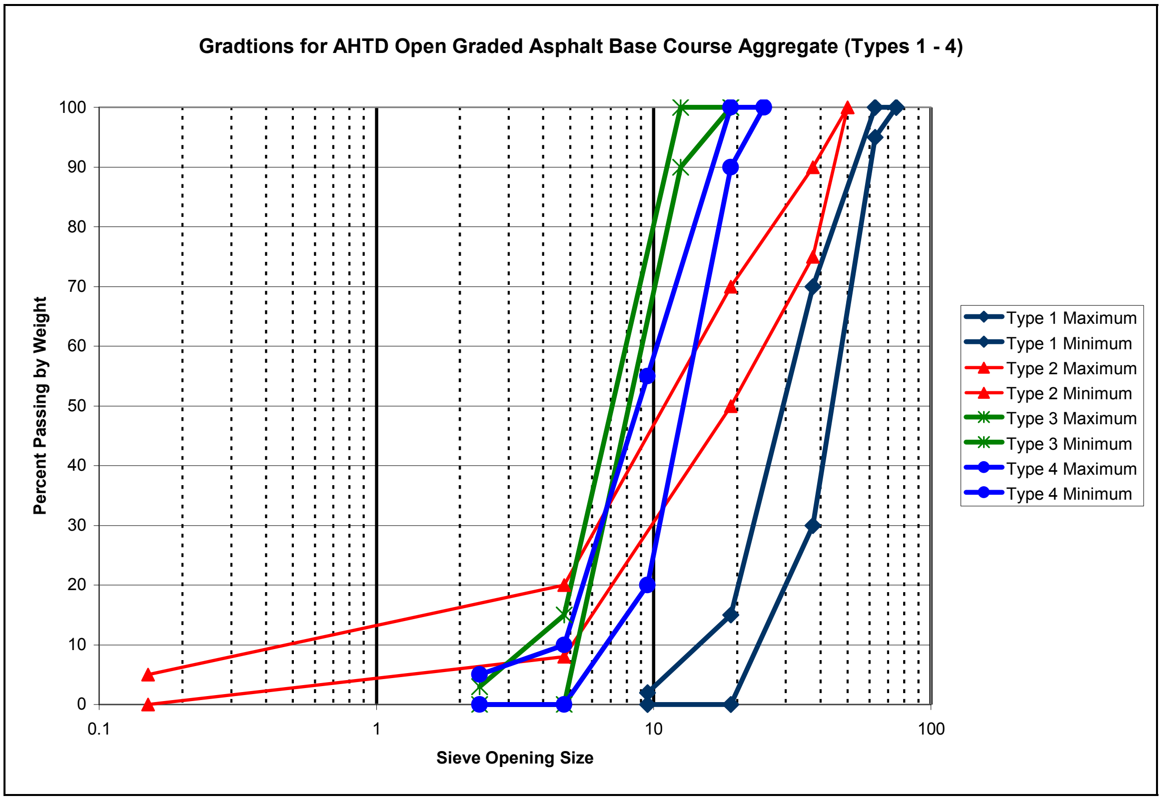

Figure A1. Gradation Composition for AHDT Open Graded Asphalt Base Courses

The coarse aggregates used to construct the open graded asphalt base courses shall comply with a number of properties. The mineral aggregates must not contain an excess of soft material or deleterious materials. The coarse aggregates (greater than the No. 8 sieve) should possess a minimum percent of fractured faces of 98% on one face and 80% on two faces. Durability testing (LA Abrasion, AASHTO T96) must have no more than 40% loss by weight. For soundness requirements, the AHDT requires that a 88% minimum value must be exhibited by coarse aggregates after 5 cycles of sodium sulphate soundness testing (AASHTO T 104). Flat and elongated particles must not exceed 10% by weight of the aggregate.

Construction Requirements

The open graded asphalt base course mixture must not be miced or placed when the surface temperature is below 40° F. Other temperature requirements include that the aggregate must be between the temperatures of 275 and 325° F when the asphalt binder is mixed. Paving must not commence or continue when the mixture is 25 degrees above the approved mix design temperature measured in the paving machine. Compaction only occurs when the temperature of the mixtures has cooled between 100 to 280° F.

Compaction practice varies with the weight of the vehicle used. A steel wheeled 2 axle tendem roller with a weight between 3.0 and 5.0 tons must coverage a complete forward and backward pass over the same section of the base course when the appropriate temperature of the mix is met in order to avoid pavement displacement. Compactors weighing between 7.0 to 11.0 tons need only make one forward pass on the base course at compaction temperature. Vibratory rollers meeting weight requirements may be employed as long as the vibratory unit is not in use.

Hauling traffic is not allowed to travel upon the open graded asphalt base course. Hauling trucks must remain on the shoulder during the construction of the open graded asphalt base course and the subsequent layers. The mixture for both cases must be conveyed to the paving machines.

The Florida Department of Transportation Specifications for Asphalt Treated Permeable Base

The Florida Department of Transportation (FDOT) uses an asphalt treated permeable base (ATPB) course and outlet pipes beneath concrete pavements. FDOT uses a combination of crushed stone and asphalt cement to construct the ATPB. They set forth specifications for construction requirements, asphalt binder, coarse aggregate, and geotextile fabrics in its 2003 Specification Manual.

Asphalt Binder

FDOT states in its specifications that it targets an asphalt content in the range of 2.0 to 3.0 percent by weight of total mixture. The designated performance grade binder that is used for ATPB is PG 67 – 22. Alike ALDOT and AHDT, the binder must meet the binder must meet requirements presented in AASHTO Section M320 Table 1. Additional binder requirements for the PG 67 – 22 binder used in FDOT ATPB include those requirements presented in AASHTO T-240 and T-104. These requirements address mass loss (maximum of 0.5% for all grades) and the presence of flammable liquid (standard naphtha is to be negative for all grades). Smoke points shall be a minumum of 125°C for the binder. Unlike ALDOT, which advises the addition of an anti- stripping agent in the blend if potential stripping issues arise, FDOT requires a heat stable anti-stripping additive to be blended with the binder at the terminal at a rate of 0.5 percent by weight of asphalt binder. These specifications also call for 95% binder coverage of all aggregate.

Coarse Aggregate

FDOT allows that coarse aggregate used in the ATPB is stone, slag or gravel. Two gradations are currently used, a Grade No. 57 or a Grade No. 67. The gradation for FDOT ATPB is displayed in Table A2 and shown in the chart featured in Figure A2. The Grade No. 57 gradation is identical to the gradation specified by ALDOT.

Table A2. Gradation Specifications for FDOT ATPB Aggregates

Requirements for FDOT ATRB			
Sieve (Square Mesh Type)		% Passing by Weight	
		No. 57	No. 67
1.5 inch	37.5 mm	100	
1 inch	25 mm	95 - 100	100
3/4 inch	19 mm		90 - 100
1/2 inch	12.5 mm	25 - 60	
3/8 inch	9.5 mm		20 - 55
No. 4	4.75 mm	0 - 10	0 - 10
No. 8	2.36 mm	0 - 5	0 - 5

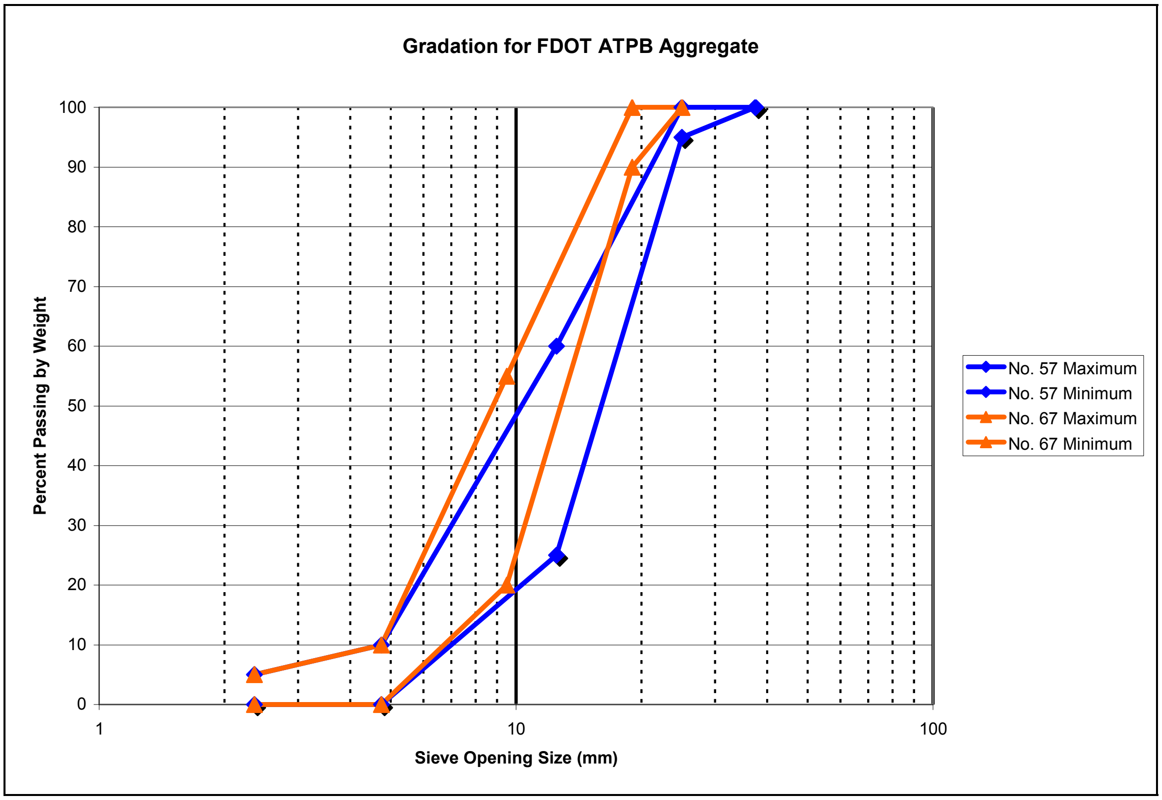

Figure A2. Gradation Composition for Aggregate in FDOT ATPB

The coarse aggregates employed for the construction of ATPB must be relatively free of clay, friable particles and other deleterious substances. Physical properties that the aggregate must comply include durability, soundness, and presence of flat, elongated particles. No more than 45% by weight of aggregate may be loss after Los Angeles Abrasion Testing. Maximum loss for aggregate undergoing AASHTO T104 Sodium Sulfate Soundness testing is 12%. Flat or elongated particles are not to exceed 10% by weight.

Geotextiles

FDOT also requires the use of a goetexile fabric to separate the ATPB course from underlying layers. It is stated in the FDOT specification the geotextiles used are woven or nonwoven fabrics that permit the drainage of water to the underlying subgrade. The fabric is not to be moved or damaged during the ATPB laydown sequence. Material requirements (i.e. handling, exposure, etc…) are identical to those requirements made by ALDOT.

Construction Requirements

FDOT requires a mixture temperature of 230 to 285°F upon laydown of the ATPB. Any ATPB mixture that had been mixed more than two hours prior to construction may not be used. Ambient air temperatures must be above 50° F and rising during the construction period. Compaction takes place as soon as the ATPB has cooled to 190° F, but must be completed before the surface temperature fall below !00°F. FDOT does allow contractors to cool the ATPB with water if necessary. Static wheeled compaction vehicles that can be used must exert an operating weight no more than 140 pounds per linear inch of drum width or a total weight of 8 to 12 tons. The ATPB layer should have a compacted thickness of 4 inches.

The Louisiana Department of Transportation and Development Specifications for Permeable Asphalt Treated Base

The Louisiana Department of Transportation and Development (LDOTD) uses a combination of aggregate, polymer modified asphalt binder, and anti-stripping additive to construct permeable asphalt base courses. LDOTD specifies requirements for the asphalt binder, coarse aggregate and additive, as well as, construction practice in its 2000 specification manual.

Asphalt Binder

The polymer modified asphalt binder used for the PATB in the state of Louisiana is Performance Grade PG76 – 22m. The binder must meet requirements shown in Table 1002-1 in the LDOTD Manual. These specifications are similar to those present in AASTHO Section 320M Table 1. Asphalt binder is added to the coarse aggregate at a rate of 2.0 to 4.0 by weight of the total mixture. At least 90% coating of the aggregate by the asphalt binder must be achieved for approval. Anti-stripping additive is to be added to the asphalt at a rate of 0.5% by weight of asphalt. Additional additive may be incurred if 90% coating is not achieved but should not exceed 1.2% by weight of asphalt binder.

Aggregate

LDOTD uses 100% crushed stone aggregates for its PATB. The aggregates must originate from a pre-approved source and must have a soundness loss of no more than 15% when subjected to sodium sulfate soundness testing (AASHTO T 104). Durability is also specified. Aggregate must not have more than 40% abrasion loss as per Los Angeles Abrasion testing (AASHTO T96). LDOTD uses the following gradation shown in Table A3. The gradation chart is shown in Figure A3.

Table A3. Gradation for LDODT PATB Aggregate

General Compostion for LaDOTD Permeable Base Aggregates		
Sieve (Square Mesh Type)		**% Passing by Weight**
1 inch	25 mm	80 - 100
3/4 inch	19 mm	90 - 100
3/8 inch	9.5 mm	20 - 55
No. 4	4.75 mm	0 - 10
No. 8	2.36 mm	0 - 5

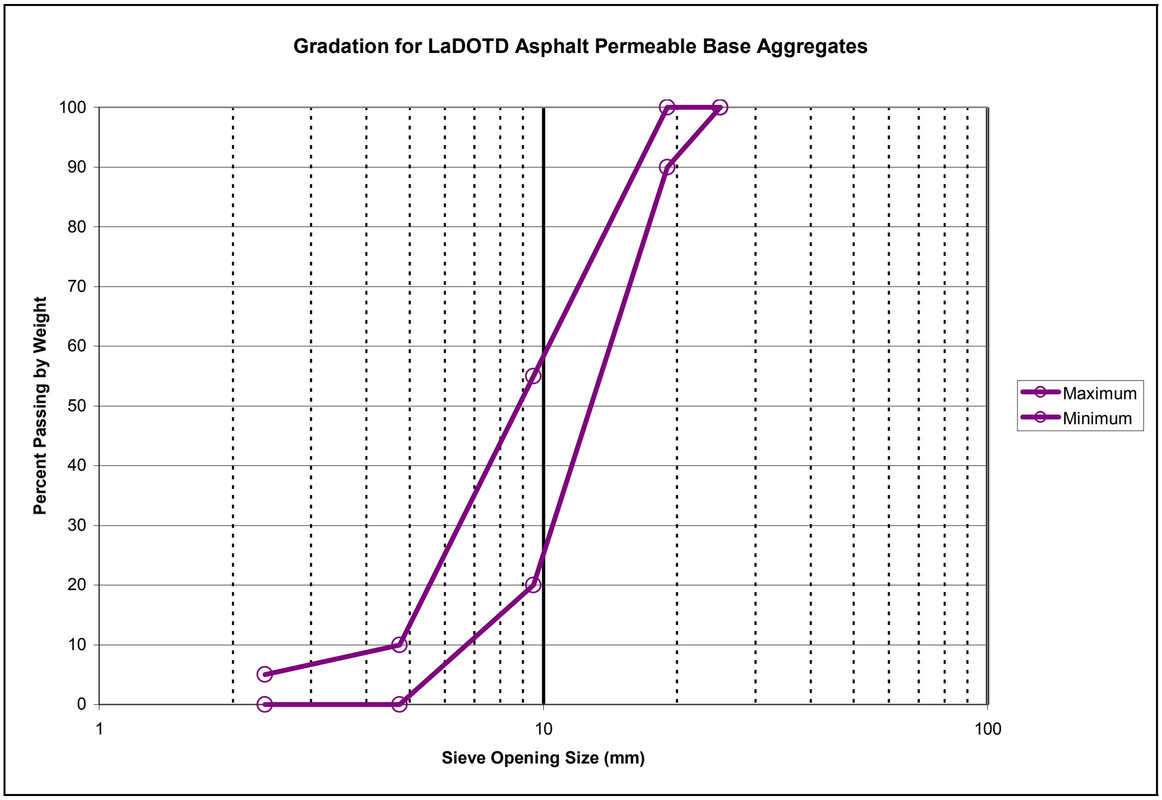

Figure A3. Gradation Composition for Aggregate in LDOTD PATB

` Similar to ALDOT's No. 57 crushed stone gradation, LDOTD's gradation calls for the inclusion of some fine particles (particles passing the No. 4 sieve). 90% of the aggregate blend is coarse aggregate.

Construction Requirements

Permeable asphalt base constructed in the state of Louisiana are paved at a temperature of 200 to 260 °F. Compaction commences at an ATPB temperature of 160 to 100°F. One to three passes of a 5 to 10 ton smooth steel wheel roller satisfies complete compaction. The permeable base is to be protected from the elements and the intrusion of fines and mud until the paving of the subsequent layer. Traffic does not travel upon the base course. Vehicles contributing to paving operations are permitted to drive on the course as long as they enter and exit as close to the paving as possible. Subsequent layers are to be placed no more than 15 working days after the construction of the PATB.

The Mississippi Department of Transportation Specifications for Asphalt Drainage Course

The Mississippi Department of Transportation (MDOT) uses an asphalt drainage course consisting of crushed aggregate and asphalt cement. It is constructed upon properly prepared surface in accordance to all of MDOT's specifications including: aggregate, asphalt cement, composition, and construction requirements.

Asphalt Binder

Liquid asphalt binder is added to the aggregate for the asphalt drainage course at a rate of 2.5% by weight. MDOT allows a tolerance of 0.4% which yields an acceptable range of asphalt content of 2.1 to 2.9 percent by weight of mixture. MDOT also specifies that hydrated lime may be employed as an additive in order to reduce stripping. Lime is to be added at a rate of 1.0% by weight of total aggregate. It is also stated in the MDOT specifications that the performance grade binder that is to be used is PG 67-22, unless otherwise stated by the construction plans. Any type of binder selected for the asphalt drainage course, whether it is PG 67-22 or any other variation of performance grade asphalt binder, must conform to the items set forth in AASHTO M-320 Table 1.

Coarse Aggregate

MDOT uses a No. 57 blend crushed limestone, sandstone, or granite to provide the physical build up of its asphalt drainage course. The aggregate gradation and chart are shown in Table A4 and Figure A4 respectively, As shown, this gradation demands very little fine aggregate (0 to 5% passing the No. 4 sieve) and is identical to the gradation blend used by the Florida Department of Transportation.

Table A4. Gradation for MDOT Asphalt Drainage Course Aggregate

General Compostion of MDOT Asphalt Drainage Course		
Sieve (Square Mesh Type)		% Passing by Weight
1.5 inch	37.5 mm	100
1 inch	25 mm	80 - 100
1/2 inch	12.5 mm	25 - 60
No. 4	4.75 mm	0 - 10
No. 8	2.36 mm	0 - 5

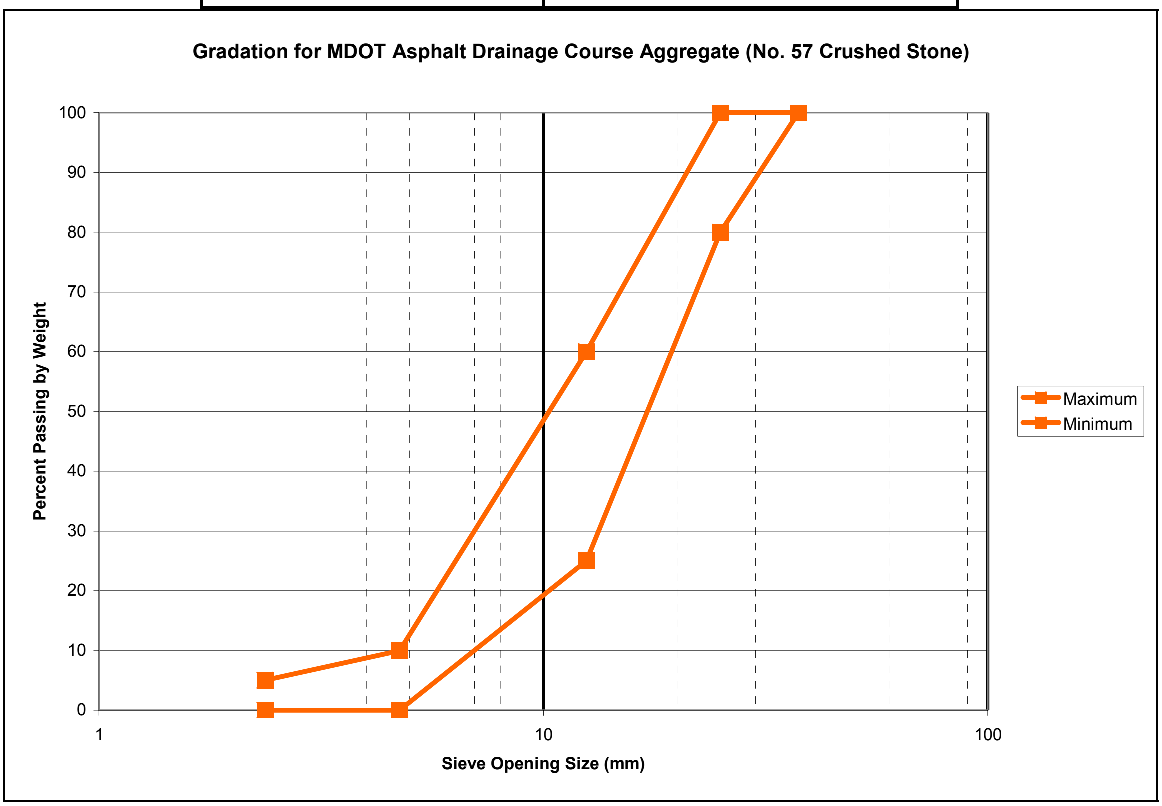

Figure A4. Aggregate Gradation in MDOT Asphalt Drainage Course

The coarse aggregate used in Mississippi must pass certain physical and characteristic properties. The aggregate in the asphalt drainage course must not be contaminated with fines or deleterious materials. The percentage of wear of the aggregate after standard Los Angeles Abrasion testing (AASHTO T96) must not exceed 45%. Sodium sulfate soundness results (AASHTO 104) are not to exceed 20%. The temperature of the mixture of aggregate, binder, and lime is to fall in the range of 220 to 250°F.

Construction Requirements

MDOT first addresses weather limitations when it introduces construction requirements for asphalt drainage course. The base course cannot be construction if the air or surface temperature falls below 40°F. The surface on which the asphalt drainage course is laid must be properly prepared to plan. Construction cannot commence if the underlying surface is wet or frozen.

Compaction is completed by vehicles weighing between 8 and 12 tons. Vibratory (only set in static mode) or steel wheeled tandem rollers compact the asphalt drainage course. Compaction is considered sufficient after 1 to 3 passes of the respective rollers. Compaction only occurs when the temperature of the mixture ranges between 100 to 150°F. The contractor is not permitted to use water to assist in the cooling process.

Similar to the aforementioned specifications of other states, MDOT's specifications are very adamant in restricting the operation of vehicles on the asphalt drainage course. The only vehicles that are allowed on the drainage course are those responsible for the construction of the overlying layers. Long hauling vehicles are prohibited. Those trucks delivering paving materials enter and exit as close to the paver as possible. MDOT is unique by stating that twisting and turning traffic is not allowed.

The contractor is also responsible for protecting the asphalt drainage course from the intrusion of clogging material.

The Tennessee Department of Transportation Specifications for Bituminous Treated Permeable Base

The Tennessee Department of Transportation (TDOT) constructs permeable base course in its roadways to facilitate the drainage of water out of pavement structures. The mixture consists of aggregate and asphalt concrete binder. TDOT has various specifications for aggregate, asphalt binder, and construction technique to ensure the proper construction of the bituminous treated permeable base.

Asphalt Binder

The current specifications used by TDOT call for the use of PG 64-22 performance grade asphalt binder in its bituminous permeable base course. The binder is applied at a rate of 1.5 to 3.5 percent by weight of total mixture. Asphalt is to visibly cover the aggregate completely. These binders applied in the mixture must conform to those requirements presented in AASHTO M320 Table 1. If stripping problems are encountered, anti-stripping agents are to be added to the bituminous treated permeable base mixture. TDOT targets a mixing temperature of the polymer modified performance grade binder with the stone aggregates at a temperature range of 150 – 170°F unless otherwise designated by the binder manufacturer..

Aggregate

TDOT uses a gradation of crushed stone, granite, gravel, or slag that consists of merely 0 – 4% fines. The information presented in Table A5 and Figure A5 shows that TDOT allows the uses of large aggregates, contrary to many its Southeastern DOT counterparts. Up to 30% of the gradation by weight can be retained on the 1-1/2" sieve. Physical requirements for the aggregate used include that sodium soundness loss must not exceed 9% (AASHTO T104) nor may it consist of more than 5% of soft or nondurable particles. 75% of those aggregates greater than the No. 4 sieve must have a minimum of two fracture faces.

Table A5. Gradation for TDOT Bituminous Permeable Base Course Aggregate

General Compostion of TnDOT ATPB		
Sieve (Square Mesh Type)		**% Passing by Weight**
2 inch	50 mm	100
1- 1/2 inch	37.5 mm	70 - 100
3/4 inch	19 mm	55 - 80
No. 4	4.75 mm	0 - 11
No. 100	0.150 mm	0 - 4
No. 200	0.075 mm	0 - 3

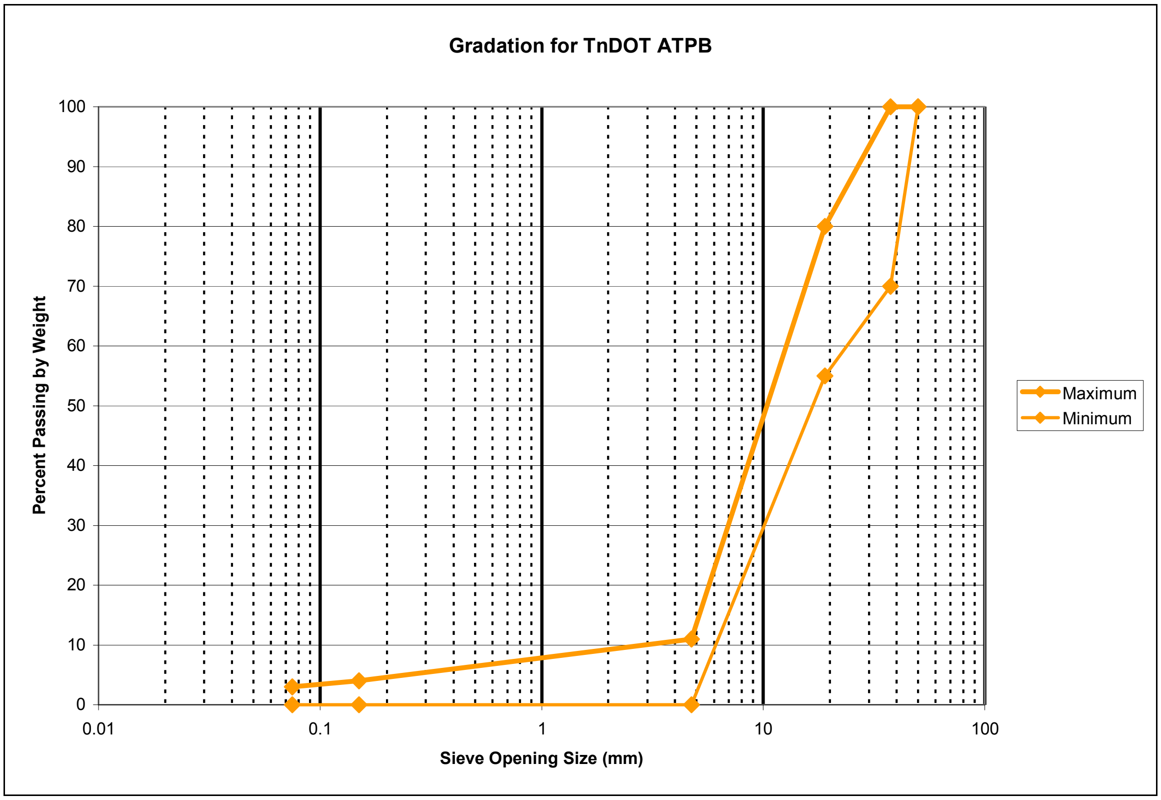

Figure A5. Aggregate Gradation in TDOT Bituminous Treated Permeable Base Course

Construction Requirements

TDOT regulates that the contractors protects the bituminous treated permeable base course from severe weather conditions, especially freezing rain and ice, as well as, the contamination of dust, dirt and other fine grained materials. Protection is required for the whole duration between lay down of the base layer and the construction of the subsequent layer. This duration must not exceed 30 calendar days. Traffic is also limited on Tennessee's permeable base courses. Traffic needed to construct the subsequent layer is allowed to travel upon the base course as long as entering and exiting occurs as close as possible to the paving operation. Lay down of the permeable base may not be done during the period between November 1st and April 1st of the calendar year.

Unlike the previous state agencies' specifications investigated, Tennessee does not have a specific specification for compaction and traffic control on the bituminous treated permeable base course. However, TDOT does have specifications that regulate the equipment to be used for the construction of TDOT pavements. In its specification section entailing equipment used for regular bituminous surface or base course construction, TDOT instructs that steel wheeled tandem or tridem type rollers must weigh at least 7.25 tons. It also declares that vibratory rollers may be used if approval by the field engineer. Considering that all of the aforementioned agencies prohibit the use of vibratory compaction, it can be assumed that the only rollers used for the compaction of Tennessee bituminous treated base course are steel wheeled fulfilling weight requirements.