Friction and Texture Retention of Concrete Pavements

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University Transportation Center for Alabama
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In the late 1980s and early 1990s, the Alabama Department of Transportation (ALDOT) noticed a decline in skid trailer numbers on concrete pavements shortly after grinding operations. The engineers at the time suspected that the coarse aggregate led to decline of skid trailer numbers and the resulting conclusion led to a ban of carbonate aggregates in mainline concrete pavement that is still in place. Recently, ALDOT has decided to reexamine the ban on carbonate aggregates in mainline concrete pavement and perform a comprehensive and detailed study of the issue.

It appears that there is sufficient data to begin using carbonate aggregates in mainline concrete pavements in Alabama provided each source is properly and fully tested. Both the researchers and ALDOT have provided sets of recommended specification language to this end.</td></tr>
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Executive Summary

In the late 1980s and early 1990s, the Alabama Department of Transportation (ALDOT) noticed a decline in skid trailer numbers on concrete pavements shortly after grinding operations. The engineers at the time suspected that the coarse aggregate led to decline of skid trailer numbers and the resulting conclusion led to a ban of carbonate aggregates in mainline concrete pavement that is still in place. Recently, ALDOT has decided to reexamine the ban on carbonate aggregates in mainline concrete pavement and perform a comprehensive and detailed study of the issue.

A total of 48 aggregate, grinding, and grooving combinations were tested as part of this extensive research project. Duplicate specimens were made for a total of 96 specimens. Three aggregate sources were examined: a siliceous source, a "hard" limestone source, and a "soft" limestone source. Two blade spacings were examined for grinding operations: 52 blades/foot and 60 blades/foot. Some ground specimens were also grooved. Finally, a set of specimens had the Next Generation Concrete Surface (NGCS) applied to them. The specimens were polished with the National Center for Asphalt Technology (NCAT) three-wheel polishing device (TWPD). At various points through the polishing process, the dynamic friction tester (DFT) and British pendulum tester (BPT) were used to evaluate the initial, intermediate, and final friction values. At the completion of polishing, the texture was characterized with the circular track meter (CTM).

Across the board, the highest performing texture was that with no grooves and 52 blades/ft. Very generally, the initial, final, and retention values for friction increased with increasing siliceous content. However, some of the trends were extremely minor and in a few cases siliceous aggregates caused higher friction loss. There were numerous instances when blended carbonate/siliceous concrete pavement surfaces performed better than sole siliceous concrete pavement surfaces. There were also a few instances where the sole limestone concrete pavement surface performed better than the sole siliceous pavement surface.

Through the course of the study, it was found that CTM testing does not indicate any benefit in increasing siliceous content. Additionally, there is no good correlation between DFT and CTM. Aggregate durability tests (i.e. acid residue, sulfate soundness, LA abrasion, and Micro-deval) are not good indicators of polishing behavior and no correlation between DFT/CTM and aggregate property values was observed.

It appears that there is sufficient data to begin using carbonate aggregates in mainline concrete pavements in Alabama provided each source is properly and fully tested. Both the researchers and ALDOT have provided sets of recommended specification language to this end.

1.0 Introduction and Problem Statement

Background

Carbonate aggregate, namely limestone, usage in portland cement concrete (PCC) and asphalt concrete (AC) mixtures have been historically limited due to observed deleterious effect on friction performance loss over time. As a result of the historical poor performance with respect to the locked wheel friction trailer numbers and traffic incident reports, ALDOT specifications explicitly prohibits the use of carbonate coarse aggregates in the construction of PCC pavements and AC pavements for surface courses (Alabama Department of Transportation Standard Specifications for Highway Construction 2018). Since AC pavements are usually constructed in multiple lifts, ALDOT allows up to 100% of carbonate coarse aggregates in non-surface course lifts. While two-lift paving technology exists for PCC pavements, its use outside of Europe is minimal and the US primarily uses single lift slipform paving for PCC construction. Thus, the entire mixture design for a PCC pavement must contain non-carbonate coarse aggregates. The vast majority of the quarries in the state of Alabama produce carbonate aggregates, specifically limestone and dolomite. Both PCC and AC construction projects must generally obtain non-carbonate aggregates from a few limited sources within Alabama or import the aggregate from out-of-state or even out-of-country.

A prior study at The University of Alabama found that limestone from the state of Alabama was being used by two border states of Alabama (Florida and Mississippi) in their PCC pavements and was widely regarded as being the best performing limestone for friction resistance (Klenke et al. 2015). Another study at Auburn University has shown that some limestone, obtained from quarries in the state of Alabama, performed comparable to some gravel aggregates in terms of friction resistance (Kandhal and Bishara 1992). This could mean the use of carbonate aggregates from certain sources could be used safely in pavements the same way non-carbonate aggregates are used. Using a blend of aggregates or carefully choosing diamond grinding and grooving textures can be potential solutions to improve the skid trailer friction numbers of PCC pavements. In turn, this could increase the feasibility of using local carbonate aggregates in the state of Alabama and reduce the cost of PCC, and indirectly AC, pavements.

Objectives and Scope of Research

The objective of this project is to investigate two potential solutions to address the loss of friction in concrete pavements comprising of local carbonate aggregates in the state of Alabama: (1) blending non-carbonate and carbonate coarse aggregates, and (2) optimizing diamond grinding and grooving textures during pavement rehabilitation.

Task 1: Material Selection and Specimen Fabrication

One of the significant tasks of this study was to select the coarse aggregates to fabricate the laboratory specimens. This task consisted of following components:

- Three types of coarse aggregates comprising a range of hardness from Alabama were selected: one non-carbonate (siliceous) and two carbonate sources (one “hard” and one “soft”).
- A siliceous river sand was selected as a fine aggregate for all mixtures.
- Properties of the coarse aggregates were tested using LA abrasion test, micro-deval test, sodium sulfate soundness test, acid insolubility test and X-ray diffraction (XRD).
- Laboratory-scaled pavement specimens of 20 in. x 20 in. x 3.5 in. were fabricated using the selected coarse aggregates with a textured top surface.

Fresh concrete properties and hardened concrete properties were measured to ensure that ALDOT specifications were met.

Task 2: Apply Textures

As a part of this task, the International Grinding and Grooving Association (IGGA) laboratory grinding apparatus was used to apply several textures to the laboratory specimens. One of the textures applied was the Next Generation Concrete Surface (NGCS). This texture has been implemented in numerous states to reduce road noise and was included as part of this study to examine its friction characteristics.

Task 3: Accelerate Wear and Characterize Friction

The scope of this task included the wear of laboratory specimens and measurement of friction retention at various wear intervals. For this purpose, following tasks were done:

- A three-wheel polishing device, originally developed at the National Center for Asphalt Technology (NCAT) was used to wear specimens.
- Friction retention was measured at various wearing/polishing cycles using Dynamic Friction Tester (DFT), British Pendulum Tester (BPT) and Circular Track Meter (CTM).

Task 4: Data Analysis

In this task, data collected from the previous tasks was analyzed. The components of this task were:

- Data obtained from DFT, BPT and CTM were compared and statistically evaluated.
- Friction retention data was analyzed considering the aggregate properties tested in Task 1.

Task 5: Recommendations and Final Report

In this task, the research team has made recommendations to ALDOT based on the testing data and current state-of-the-practice within Alabama. A national examination of state DOT specifications was also used to guide the recommendations to be consistent and conservative in nature.

2.0 Literature Review

Review of Pavement Friction

Friction between a tire and pavement is not fully understood. It is therefore measured and observed but the mechanisms behind the frictional force are not fully explained. In the literature (Hall et al. 2009; Kummer and Meyer 1966), two main mechanisms and components of pavement friction (Figure 2-1) are discussed: adhesion and hysteresis.

Adhesion corresponds to shear of the molecular bonds formed between pavement and tire when the rubber of the tire is compressed against the unevenness of the road surface resulting in high normal pressure (Hall et al. 2009; Rizenbergs 1968).

When the tire moves over the unevenness of the pavement surface, it is subjected to deformation. This involves energy to compress the tire when it is on top of the protruded area of a pavement surface. However, the rubber of the tire when moved from this protruded area does not regain the whole energy supplied. This results in loss of energy due to the deformation of the tire rubber and is known as the hysteresis component of pavement friction (Hall et al. 2009; Rizenbergs 1968).

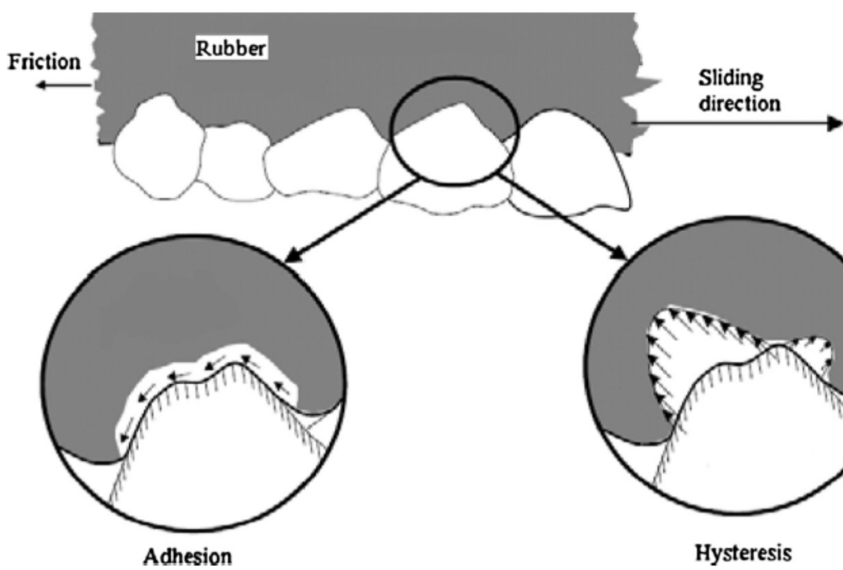


Figure 2-1. Two main components of friction (Hall et al. 2009)

Important variables affecting the tire-pavement friction are pavement surface characteristics, wetness of the pavement, tire properties, environmental factors, normal load, area of contact and sliding velocity of tire with respect to pavement (Rezaei et al. 2011; Rizenbergs 1968). However, sliding velocity, wetness of the pavement, and pavement surface characteristics are most critical as these can be countered through pavement design (geometric and materials) and speed limit regulations.

Sliding Velocity

The adhesion component of friction decreases with increase in sliding velocity of the vehicle (Rizenbergs 1968). This might be due to reduced possibility of molecular bonding between tire and surface. The hysteresis component increases slightly with the increase in sliding velocity due to increase in the rate of deformation of rubber tire on the pavement (Rizenbergs 1968). Overall, the pavement friction decreases with increase in sliding velocity.

Surface Moisture

It is a widely known experience that friction on wet pavement is less than friction on dry pavement (Panagouli and Kokkalis 1998a; Rezaei et al. 2011; Rizenbergs 1968). The loss of friction is mainly due to reduction in the adhesion component of friction since the formation of molecular bond between pavement and tire in the presence of water is difficult (Rizenbergs 1968). Since the hysteresis component of the friction depends on the tire rubber deformation, it remains largely unaffected (Rizenbergs 1968).

Macro- and Micro-Texture

Pavement surface characteristics are mainly identified by the surface texture or the irregularities of the pavement (Panagouli and Kokkalis 1998b; Rezaei et al. 2011; Whitney et al. 2013). Primarily, two types of surface irregularities affect pavement friction: microtexture and macrotexture.

Irregularities of 0.3 mm to 50 mm contribute to macrotexture category of the pavement texture. Mainly, shape, angularity, spacing and distribution of aggregates contribute to macrotexture (Panagouli and Kokkalis 1998b). Hysteresis component of friction derives from macrotexture and plays an important role at higher sliding velocities (Whitney et al. 2013). Therefore, in wet conditions macrotexture is more crucial. The Circular Track Meter mainly measures the macrotexture of the surface (ASTM E2157).

The texture of pavement surface at micro level, that is, irregularities between 0.005 mm and 0.3 mm, contribute to microtexture (Panagouli and Kokkalis 1998b). Texture of aggregates and the texture of cement mortar contribute to the microtexture. Microtexture of the pavement surface contribute to adhesion type of friction and therefore is more relevant at lower sliding velocities (Whitney et al. 2013). Polished aggregates decrease the microtexture and therefore the skid resistance of pavements (Rezaei et al. 2011). Therefore, type of aggregate used is of great importance to the friction retention of the pavement surface. The British Pendulum Tester mainly measures the microtexture of the pavement surface. As the Dynamic Friction Tester measures the coefficient of dynamic friction for a range of sliding velocities, it measures both microtexture and macrotexture.

Review of Geology

The geological composition of the aggregates used in concrete directly impact the macro- and micro texture properties of the surface. Additionally, those aggregates can impact numerous other physical properties such as strength, flexure, modulus, and even polishing resistance. The impact these aggregates have on overall performance shows the importance of identifying and analyzing the properties of the aggregate.

Carbonate Aggregates

Calcium carbonate, generally referred to as limestone, is classified as a sedimentary rock, meaning it is composed of numerous different layers of varying material and texture (Pirsson 1908). It can occur naturally or synthetically in hydrous (monohydrate, hexahydrate, and ikaite) and anhydrous (calcite, aragonite, and vaterite) forms (Clarkson et al. 1992; Vanderdeelen 2012). Calcite is the most commonly found phase of calcium carbonate and contains high calcium percentages. Dolomite, which is similar to calcite, has a 50/50 $Mg^{2+}\ to\ Ca^{2+}$ ratio, whereas pure calcite is considered to have no Mg^{2+}. Dolomite is formed from calcite without chemically affecting the original CO_3^{-2} (Degens 1965).

Calcite and dolomite are both classified as nondetrital sedimentary rocks. Calcite is known to be almost completely soluble when in contact with HCl, whereas dolomite, at room temperature is not completely soluble. This has to do with the size of the Ca and Mg atoms and the strength of the bonds they form with oxygen. Calcium is about 40% larger than magnesium and thus the Mg-O bond has a higher degree of covalency compared to the Ca-O bond (Lund et al. 1973; Singurindy and Berkowitz 2003). When the temperature is increased (Figure 2-2), dolomite is dissolved at a rate two orders of magnitude faster than the rate at room temperature (Lund et al. 1973). Thus, a correctly run ASTM D3042 acid insoluble residue test should yield similar numbers between calcite and dolomite, especially given the fact that titration is not involved and the Ca and Mg ions are not separately characterized.

Typically, carbonates have a hardness value on Mohr's hardness scale of 4 or less, with soft limestones having a smaller value and hard limestones having a value closer to 4. Limestone has a specific gravity of between 2.0 and 2.7 and a porosity of between 0-25% depending on the cemented oolite present (Pirsson 1908). Thus, dolomite, i.e. dolomitic limestone, is generally harder than pure calcite, i.e. limestone. Dolomite is also considered slightly more dense, having a specific gravity of about 2.87 when pure with roughly the same absorption (Pirsson 1908). In general, it has been found that the largest factor on the physical properties of varying type of rocks lies in the microstructural properties they have from their formation. This was determined by a study that investigated the correlation of abrasion hardness to rebound hardness test using a Schmidt hardness test, showing, with reasonable accuracy, that it was possible to predict abrasion hardness from the Schmidt test (Shalabi et al. 2007). For polishing, ALDOT currently uses ALDOT-382, known as the BPN9 test, which is essentially the ALDOT version of ASTM D3319 and nearly equivalent to AASHTO T279 (Table 2-1).

Table 2-1: Comparison of BPN9 Testing Methods.

Parameter	ALDOT-382	ASTM D3319	AASHTO T279
Year of last revision	1994	2017	2018
Aggregate Type	Only Carbonate	Any	Any
Sample Preparation	Follow ASTM D3319	--	Identical to ASTM D3319
Surface Friction	ASTM E303	ASTM E303	AASHTO T278[1]

[1.]Identical to ASTM E303.

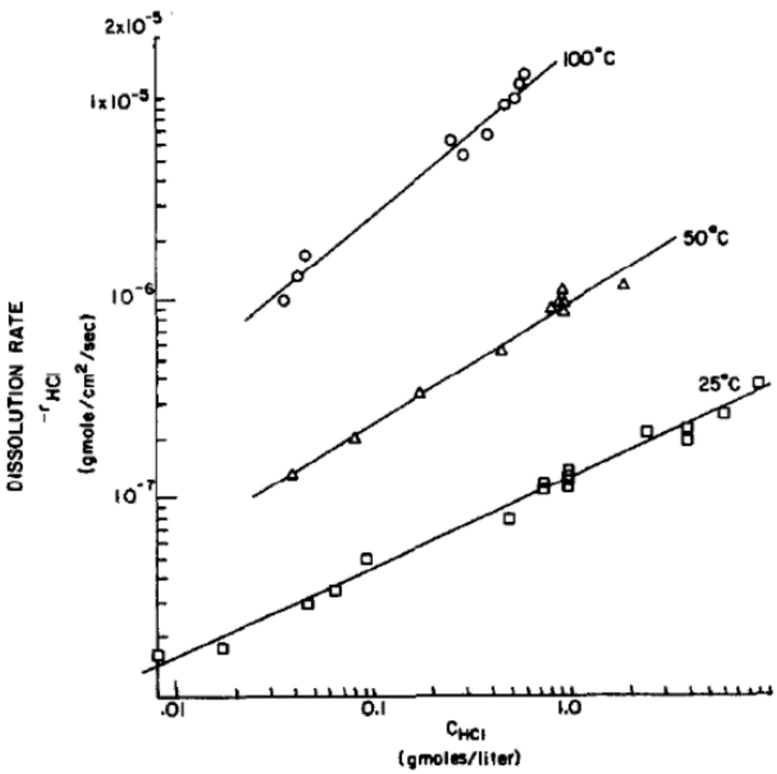


Figure 2-2. Dissolution rates for dolomite in HCl at various concentrations (x-axis) and temperatures (Lund et al. 1973)

Siliceous Aggregates

Granite, a type of siliceous aggregate, is classified as an igneous rock. In general, igneous rocks are comprised of oxides, including, but not limited to: silica, alumina, iron oxide, ferric, ferrous, magnesia, lime, soda, and potash. Granite, in particular, is classified as an equigranular phanerite structure containing quartz and feldspar and is normally lighter in color. Because of this crystalline make up of coarse grains, it is typically very rough in texture, unlike many carbonates. Granite, like other igneous rocks, is known to be almost completely insoluble to acid. Its specific gravity is not significantly different from carbonates, ranging from 2.61-2.75

generally, and normally has a unit weight of around 165 lbs/ft^3. Granite is known for its very strong structural properties as well as having very low porosity (Pirsson 1908). In terms of hardness, granite normally ranges between 6-7 on a Mohr's scale making it significantly harder than carbonates (Roger and Richard 1948).

Previous Carbonate Polishing Studies

Limestone is a very common aggregate source for much of the U.S., especially the southeastern portion. A study in 2005 demonstrated that some form of limestone was used as an aggregate in 37% of the concrete mixed in the United States (Gransberg and James, 2005). Research has shown that limestone has poor performance compared to granite, trap rock, and sandstone in terms of polish resistance (Rado 2009). This polishing behavior can lead to reductions in pavement friction retention. This has led some states, such as Alabama, to limit the amount of limestone aggregates in mainline pavement surface(Klenke et al. 2015). This can cause logistical and economic issues for states that contain a large number of limestone quarries, such as Alabama (Klenke et al. 2015). The FHWA, in Technical Advisory T 5040.36 recommends the following:

The fine aggregate fraction of the aggregate concrete matrix provides microtexture. As such, the fine aggregate should be wear and polish resistant. A minimum 25% of the fine aggregate for concrete should be siliceous material.

The aforementioned advisory only addresses the fine aggregate portion, not the coarse aggregate portion. FHWA Technical Advisory T 5080.17 discusses PCC mixture design and no coarse aggregate properties related to polishing are described.

Sources Within Alabama

The state of Alabama has a significant supply of limestone in various portions of the state (Kandhal and Bishara 1992). As previously mentioned, siliceous aggregates are often used in place of limestone. However, siliceous aggregate has some disadvantages, such as being susceptible to water damage in AC pavements and having low strength and stability in PCC pavements. Previous testing conducted at Auburn University have shown that some limestone sources in Alabama performed equivalent to some gravel aggregates in terms of friction resistance (Kandhal and Bishara 1992). This opens the possibility that carbonate aggregates from certain sources could be used safely in pavements the same way siliceous aggregates are used. However, there must be a method to consistently characterize carbonate aggregates to accurately select those that will perform well. In the study conducted at Auburn University, the researchers were able to loosely correlate the performance of an aggregate in an acid insoluble residue test. It was reasoned that the lower carbonate percentage in the aggregate would have better friction retention properties. Nevertheless, it was determined that this correlation was not strong enough to make a standard recommendation (Kandhal and Bishara 1992). This type of correlation is further confounded by the presence of dolomite. As previously mentioned, at room temperature, dolomite is less soluble in an acid solution than calcite. However, at the elevated temperatures seen in the acid insoluble residue test (ASTM D3042), the dolomite becomes as soluble or even more soluble than calcite. Kandhal and Bishara did not directly account for this solubility discrepancy in their evaluation of the acid residue test for aggregate qualification.

In the same study, when looking at locally produced aggregates, the authors provided a very detailed look at the geological make up and test results of aggregates from various quarries and regions in the state of Alabama. A comparison of these numbers for the region where the reported “better performing” limestone came from (Calera) showed that of the four sources of limestone listed in the study, one was shown as 100% calcite, one was shown as 48% calcite and 51% dolomite, one was shown as 85% dolomite and 13% calcite, and the fourth was shown as 100% pure dolomite. It should be noted that the authors analysis method to determine calcite/dolomite contents were extremely qualitative and the numbers presented could be significantly different. The authors primarily utilized a 10% acid efflorescence test and petrographic microscopy following the Folk classification system (Folk 1959) to evaluate the amount of calcite/dolomite. While there is nothing inherently wrong with the data, it is imprecise due to the subjective nature of the analysis. X-ray diffraction (XRD) provides a significant improvement in the quantification of the calcite and dolomite phases. Nevertheless, the results from the 1992 study show that on average, the tested limestones has a BPN9 value of 31 with a range between 24 and 36 while the baseline gravel average was also 31 with a range between 27 and 34 (Kandhal and Bishara 1992). If carbonate aggregates are thought to experience a higher degree of polishing in the field, it is apparent from this early study that the BPN9 value alone is not a clear indication of the observed field performance.

A prior study at The University of Alabama examined the use of carbonate aggregates in pavement and sent a questionnaire to several Southeastern states to determine the use of said aggregates in PCC pavement. It was found that there were two states that border Alabama, Florida and Mississippi, that used limestone from the state of Alabama, specifically quarries in Calera and Tuscumbia, in their PCC pavements. These same DOTs noted that the limestone from Alabama was widely regarded as being the best performing limestone for skid resistance in PCC pavements (Klenke et al. 2015).

Sources Outside of Alabama

In the 2000’s, several roads in the state of Pennsylvania started to experience rapid friction loss. The Pennsylvania Department of Transportation launched a study to determine if the use of a particular source of limestone aggregates, Vanport limestone, had been the cause of this deterioration. They first examined what other DOT’s used as standards regarding limestone testing and noted that several DOT’s require a certain percent of any aggregate used to be insoluble per the acid insoluble residue test (Rado 2009), which correlates with the suggestions and research of a previous study done in Alabama (Kandhal and Bishara 1992). The Pennsylvania Department of Transportation decided to look at multiple testing methods and use them to determine how likely it was that their aggregate was the cause of the poor state of the roads. In their conclusion, they noted that while regular limestone was the worst performer in all of these various test, sandy limestone (not 100% carbonate) can in many cases outperform granite aggregate (siliceous) (Rado 2009).

Some researchers have tried to offset this poor performance of certain limestone sources by blending multiple aggregates into the concrete mixes. One study showed that combining limestone and gneiss (carbonate and siliceous mix), can actually offset the poor skid resistant properties of the limestone when tested with micro-deval, polished stone value (PSV) method (BS 812), and Wehner & Schulze tests (Senga et al. 2013). In another study that examined different aggregates of various geological complexion, it was shown that the micro-deval test had very consistent and repeatable results with the field performance of those respective aggregates for concrete and hot-mix asphalt mixes in field studies (Fowler et al. 2006). The micro-deval reliability was also seen in another study (Rezaei et al. 2011). Another study (Rado 2009) found that the British Pendulum Test (BPT) was a good indicator of aggregate performance for friction retention.

Several researchers have investigated the roles that micro-texture and macro-texture have in skid resistance and aggregate performance. Micro-texture of a pavement can be considered the angularity and mineral properties, whereas macro-texture is overall pavement coarseness (Ergun et al. 2005). Since it has been concluded that slip speed is a major factor in skid resistance, macro-texture is considered to have a larger impact on skid resistance, although micro-texture cannot be ignored. Regardless, both can be used to analyze the friction characteristics of pavements (Ergun et al. 2005). A study looking to determine some non-geological causes of poor friction resistance showed that in some cases, densely graded asphalt mixes improved macro-texture enough to help improve friction resistance (Rezaei et al. 2011). The same study did however show that PFC (Permeable Friction Courses) mixes were more influenced by micro-texture characteristics (Rezaei et al. 2011).

Some researchers have related non-geological factors to the skid resistance of pavements in general. These factors can help improve skid resistance which could make limestone and other carbonate aggregates a viable and safe option for pavements. For example, researchers in 2005 determined that there was a linear type response of the skid resistance of limestone aggregates pavement sections and temperature when subjected to polishing, citing data that showed that when temperature increased, the skid resistance would decrease and that limestone sections lost their friction at a much faster rate than gravel sections (Bazlamit and Reza 2005). Another study showed that the hardness of the aggregate could cause polishing, theorizing that single mineral rocks with low hardness would perform poorly when tested for skid resistance. They tested their theory and found good correlation with that theory when testing limestone with a Wehner/Schulze test (Kane et al. 2013).

A different study looked at numerous different geological rocks used in pavement sections to see if surface treatment and gradation had any impact on skid resistance. Lab and field results showed that surface treatment actually did lead to higher friction numbers in sections; densely graded mixes performed slightly better but not to the same level as surface treatments (Rezaei et al. 2011). One of the more surprising non-geological factors that can impact skid resistance is the cementitious material used. Two different research studies, (Yoshitake et al. 2016) and (Asi 2007), investigated the use of fly ash and slag improving skid resistance of pavements. The first study sought to find the skid resistance of limestone aggregates in recycled concrete pavements with 40% fly ash and a lower w/cm ratio used; this was shown to increase the skid resistance to be similar to that of siliceous aggregates, meaning recycled limestone concrete, with the addition of fly ash, could greatly improve the friction (Yoshitake et al. 2016). The other study looked to examine what factors would be the largest impact on skid resistance between asphalt content, mix design, or aggregate gradation or quality. The results of this study showed that adding 30% slag to the mix design increased skid resistance significantly, while increasing asphalt content actually would lower the skid resistance (Asi 2007).

As mentioned previously, it has been theorized and tested that limestone aggregates that are not 100% calcite, such as dolomitic limestone, can perform better in pavements. Dolomitic limestone is a rather common geological occurrence in the Southeast, especially in Alabama (Kandhal and Bishara 1992). Some studies looking at the performance of different types of limestones have looked at the performance of dolomite and dolomitic limestone. The most extensive and prominent was a study in 1991 looking at abrasion resistance of concrete (Laplante et al. 1991). This study showed that dolomitic limestone performed much better than typical limestone in aggregate testing. In many cases, traditional limestone had very high loss of friction properties, but dolomitic limestone was able to perform almost identical to granite in all of these categories (Laplante et al. 1991) . This is very important because granite is generally considered a very hard and skid resistant aggregate. This could indicate that instead of pure limestone being used, dolomite or dolomitic limestone could be used and produce much safer results for roadways.

DOT Specifications for Carbonate Aggregates

Current ALDOT Specifications

This section outlines the current specifications regarding carbonate and/or limestone coarse aggregate usage in both asphalt and concrete pavements in Alabama (Table 2-1). Interestingly, there is a significant difference between asphalt and concrete pavement specifications in Alabama. Asphalt pavements can incorporate carbonate aggregates dependent on traffic levels or British Pendulum Testing (BPN) results after a 9-hour polishing procedure (ALDOT-382). For concrete pavements, limestone is outright banned from use in mainline paving regardless of any specific aggregate property whereas in certain applications, up to 50% carbonate aggregate can be used for an asphalt pavement. At the time this report was written, there was only a single source of approved limestone with a BPN9 value greater than 35.

Table 2-2. Current ALDOT specifications regarding carbonate aggregate usage in paving operations

Pavement	Specification	Language
Asphalt[1]	401.02(b)	"The use of carbonate stone…shall be restricted as follows": • 500 vehicles or less per day – No restrictions apply • More than 500 but less than or equal to 1,000 vehicles per day – Carbonate stone shall not be used in the final application [wearing surface] • Over 1,000 vehicles per day – Carbonate stone shall not be used in any application
Asphalt[2]	403.02(a)	Similar restrictions as 401.02(b) with exception of: • 501 to 5,000 vehicles per day – Carbonate stone shall be limited to a maximum of 30% of the blended gradation
Asphalt[3]	409.02(b)	Identical to 401.02(b)
Asphalt[4]	423.02(c)	Removes average daily traffic (ADT) restrictions and specifies BPN9 test for maximum carbonate percentages allowed: • BPN9 ≤ 25, maximum of 30% carbonate stone • BPN9 26 through 28, maximum of 35% carbonate stone • BPN9 29 through 31, maximum of 40% carbonate stone • BPN9 32 through 34, maximum of 45% carbonate stone

		• BPN9 ≥ 35, maximum of 50% carbonate stone
Asphalt[5]	424.02(c)	Identical to 423.02(c)
Concrete	450.02(b)	"The coarse aggregate for mainline and ramp pavement shall be granite, sandstone, quartzite, or gravel with a specific gravity greater than 2.550 (specific gravity requirement applies to gravel only). Gravel with a specific gravity less than or equal to 2.550 and limestone will not be allowed."

[1]Surface Treatments
[2]Micro-surface Seal Coat
[3]Triple Layer Surface Treatment
[4]Stone Mastic Asphalt
[5]Superpave Wearing Surface

Current National Specifications

A survey of the remaining 49 state DOT specifications has shown that while some identify potential issues with limestone aggregates, nearly all of them require testing for use in concrete pavements (Table 2-2 and Figure 2-3). Alabama, Idaho (703.02(a) and 703.02(c), 2018 Idaho DOT Specifications), New Jersey (901.06.01, 2018 supplemental update to 2007 New Jersey DOT Specifications), and South Carolina (SC-M-501) are the only four states with an outright ban on limestone aggregate use in pavements. There are several states that have no specifications related to aggregate polishing in concrete pavements: Alaska, Delaware, New Hampshire, and Utah. Of these states, New Hampshire (401.2.1.4, 2016 New Hampshire DOT Specifications) and Utah (02786.2.3(b) and 02741.2.2(b), 2017 Utah DOT Specifications) have polishing specifications for asphalt pavements. Alaska and Delaware have no polishing specifications at all. They, and the other listed states, may address polishing through engineering controls or some other method but it is not explicitly listed in the standard/special specifications.

Table 2-3. National specifications for aggregate qualification for pavements with respect to polishing susceptibility. Unless otherwise noted, the listed specification refers to limits from LA Abrasion (AASHTO T-96) testing.

State	Concrete Spec.	Asphalt Spec.	Notes
Arizona	1006-2.03(c)	404-2.02(c)	
Arkansas	501.02(c)	409.01	
California	90-1.02(c)(2)	39-2.02(a)(4)(e)	
Colorado	703.02	703.04; 703.05	Asphalt: max 18% micro-deval
Connecticut	M.03.01(1)(b)	M.04.01(1)(b)	
Georgia	800.2.01(a)		
Hawaii	703.02	703.09	All aggregate must be basalt
Idaho	703.02(a); 703.02(c)	703.05	Concrete: no limestone in wearing surface
Illinois	1004.01(a)		Asphalt: high ESAL HMA must contain high % dolomite
Indiana	904.03(a)		Polish resistant test: ITM-214
Iowa	4115.02	4127.02	
Kansas	1102.2(a)	1103.2(b)	
Kentucky	805.03.02		
Louisiana	1003.01.2.3		Friction rating system: 1003.01.2.4
Maine	703.02	703.07	Both: max 18% micro-deval
Maryland	901.01		Asphalt: carbonate must have min 25% insoluble residue
Massachusetts	M2.01.0		
Michigan	902.11		
Minnesota	3137.2(d)(1)	3139.2(c)(1)	Asphalt: no carbonate from Platteville Geological Formation
Mississippi	703.03.2.2	703.06.1	

Missouri	1005.2.1.2	1002.2.1.2	
Montana	701.01.2(d)	701.03.1	
Nebraska	1033.02.3(b)(6)	1033.02.4(a)(7)	Asphalt: max 80% limestone in surface (1028.02.3(b))
Nevada	706.03.01	705.03.01	
New Hampshire	N/a	401.2.1.4	
New York	703.02		
North Carolina	1014-2(d)	1012-1(b)(6)	
North Dakota	802.01(c)(2)	816.02	
Ohio	703.02(b)(2)		Both: Guidance available for polishing
Oklahoma	701.06	703.04(b)	
Oregon	0269.20(d)	00744.10(a)(2)	
Pennsylvania	703.2		
Rhode Island	M.01.10		
South Dakota	820.2(b)	880.2(b)	
Utah	N/a	02786.2.3(b); 02741.2.2(b)	Asphalt: Uses BPN9
Vermont	704.02(b)	704.10(a)(2)	
Virginia	203.03(c); 248.02(a)		
Washington	9-03.1(1)	9-03.8(1)	
Wisconsin	501.2.5.4.3	460.2.2.1	

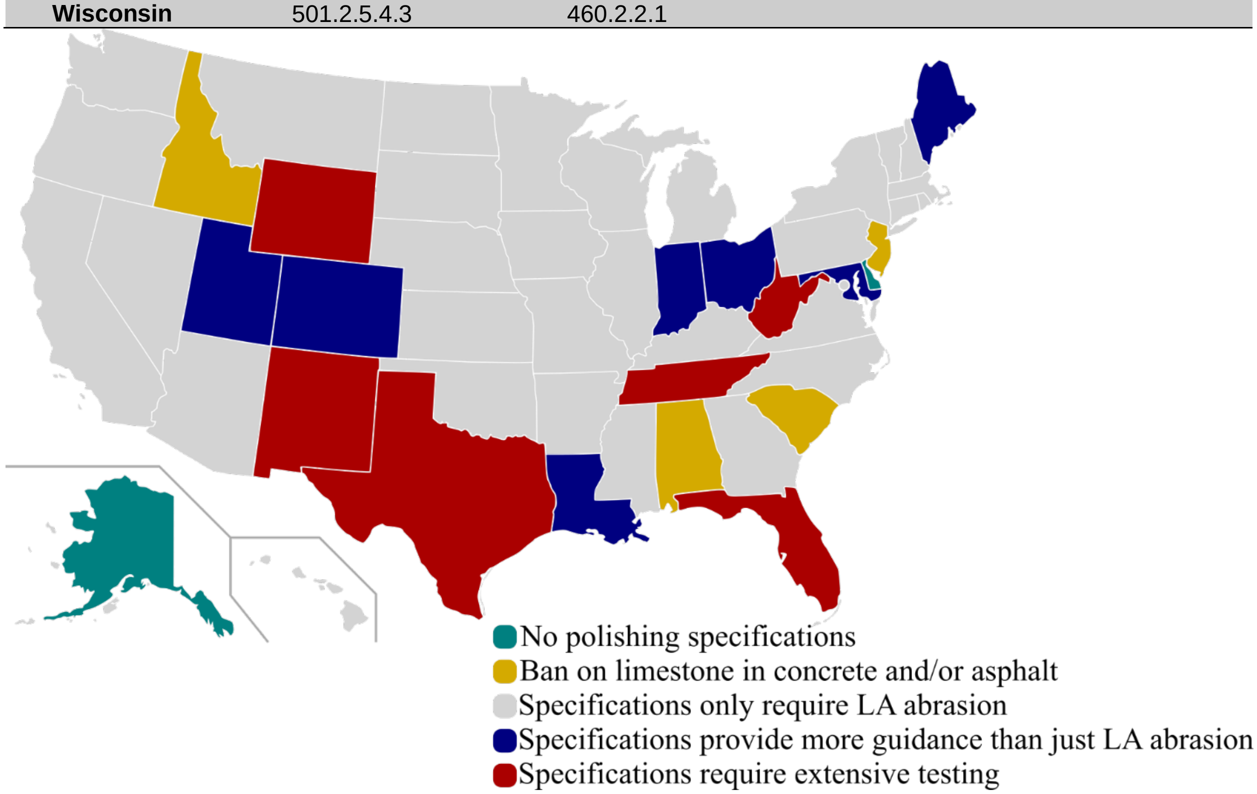


Figure 2-3. Map showing state DOT specifications related to polishing

Notably absent from Table 2-2 are the states of Florida, New Mexico, Tennessee, Texas, West Virginia, and Wyoming. These six states have extensive guidance on how to specify and mitigate polishing problems as a function of their standard specifications. Other states may use engineering controls to mitigate polishing, but the six aforementioned states explicitly outline how this is done in a standardized fashion. It may be possible to use these six states as a model moving forward to systematically address polishing of both concrete and asphalt pavements.

Florida Specifications The Florida DOT has unique specifications addressing the use of limestone aggregate. It is apparent that their specifications were developed with input from a geological perspective. Both concrete and asphalt pavement aggregates must have a max LA abrasion loss of 45% (901-1.3). In addition, when limestone is used for either concrete or asphalt pavement as part of a friction course (901-2.3), the "crushed limestone shall have a minimum acid insoluble content of 12% (FM 5-510)". Furthermore, for limestone and dolomitic aggregates used in asphalt pavements (901-2.3):

Pre-Cenozoic limestones and dolomite shall not be used as crushed stone aggregates either coarse or fine for asphalt concrete friction courses, or any other asphalt concrete mixture or surface treatment serving as the final wearing course. This specifically includes materials from the Ketone Dolomite (Cambrian) Newala Limestone (Mississippian) geologic formations in Northern Alabama and Georgia

New Mexico Specifications The New Mexico DOT uses a calculated parameter called the aggregate index to systematically evaluate different aggregate sources. The aggregate index, *AI*, is calculated according to 910.2.4 (2007 specification book) and is as follows:

$$AI = \frac{1}{3}\sqrt{LA^{2.2} + SL^{3.0} + A^{4.0}}$$

where *LA* is the LA abrasion loss percentage, *SL* is the magnesium sulfate soundness loss percentage, and *A* is the absorption capacity of the aggregate, as a percentage. Aggregates used in concrete pavements (509.2.4.2.2) and asphalt pavements (423.2.2.1.1) must have an aggregate index value less than 25. It is not immediately clear why absorption capacity is considered in this system but it is suspected that it is to limit the freeze-thaw damage potential in a similar fashion as the magnesium sulfate soundness test.

Tennessee Specifications The Tennessee DOT goes beyond LA abrasion testing and utilizes additional testing for riding surfaces. The full specification is found in 903.24 and a summary is presented below in Table 2-3. In addition, any aggregate shall be preapproved for use by the Division of Materials and Tests. The aggregates must maintain satisfactory performance in the field to remain an approved source.

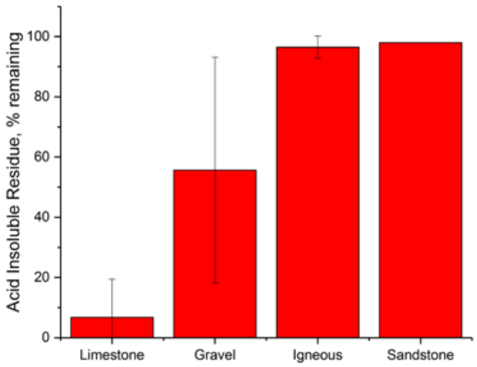
Table 2-4. Testing specifications for aggregates used in riding surfaces in Tennessee

Property	Test Method	Type I (all roads)	Type II (all roads)	Type III (15,000 ADT max, excluding interstates)	Type IV (5,000 ADT max)
SiO_2, % min	ASTM C25	40%	30%	20%	10%
CaCO3, % max	None listed	32%	--	--	--
Acid Insoluble, % min	ASTM D3042	50%	35%	25%	--
BPN9, min	AASHTO T279	30	30	25	22

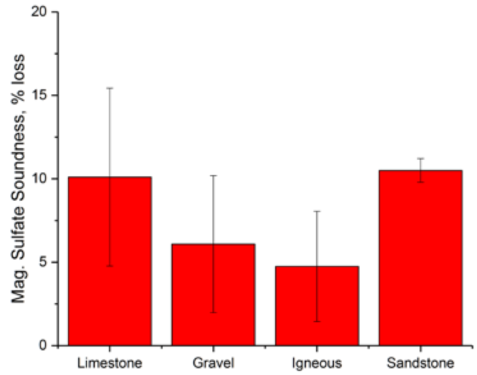
Texas Specifications The Texas DOT goes a bit further beyond what Tennessee does and has an entire material producer list that is essentially pre-qualified for various applications. For example, with concrete pavements, the TxDOT prequalified list, Concrete Rated Source Quality Catalog (CRSQC), divides the coarse aggregate sources into 15 geological categories and then characterizes them with five different test methods. The five methods are: LA abrasion, micro-deval, magnesium sulfate soundness, coefficient of thermal expansion (COTE), and acid insoluble residue. Interestingly, TxDOT does not use the BPN9 test (AASHTO T278/T279) to characterize aggregate.

The published catalog is updated bi-annually and while it does not exempt the listed source from further quality assurance testing, it provides a readily accessible listing of all potentially viable aggregate sources for concrete pavements. TxDOT also uses the same list method for asphalt pavements but removes the COTE test requirement. The data from a COTE test is only useful when designing continuously reinforced concrete pavements (CRCP).

Looking at the material producer list, the average values for the different aggregate types and test methods are shown in Figure 2-4. The general trends are as expected for the different types of aggregates. The acid insoluble residue values for gravel vary significantly since the gravel category encompasses 10 of the 15 geological aggregate types. If the average values are put in the context of other DOT specifications, namely the max 40% LA abrasion loss and max 18% micro-deval loss, nearly all the sources listed would meet other state specifications.



(a)



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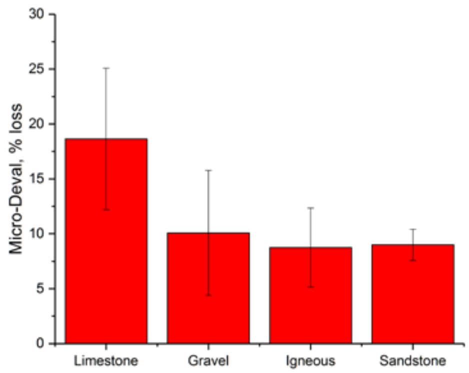

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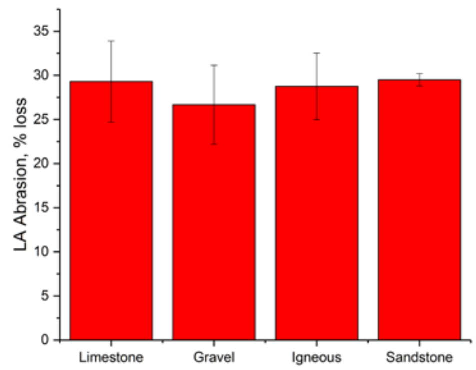


(d)

Figure 2-4. Charts of aggregate properties from the Texas Concrete Rated Source Quality Catalog

West Virginia Specifications The West Virginia DOT explicitly outlines, in the standard specification book, what are considered skid resistant aggregates for concrete and asphalt pavements. For both concrete and asphalt, the max LA abrasion is 40% (703.1.3). When using limestone, both concrete and asphalt pavements must use a source that contains a minimum of 10% acid insoluble residue (703.1.5.1). If dolomitic limestone is used for either concrete or asphalt pavements, the dolomite source “shall contain a minimum of 10% elemental magnesium” (703.1.5.2).

Wyoming Specifications The Wyoming DOT allows the state engineer to specify polish resistant aggregates for certain projects. For projects that do not have the requirement for polish resistant aggregates, the aggregates used in concrete pavements must have a max LA abrasion value of 40% (803.2.2) while asphalt pavement aggregates can have max LA abrasion values between 35-40% depending on type (803.5.5). When polish resistant aggregates are specified (803.6.2), the following parameters (Table 2-4) are used. The skid number requirement is based on at least five years of field data but only for pavements carrying traffic exceeding 3.5 million ESALs.

Table 2-5. Wyoming specifications for aggregate polishing beyond LA Abrasion testing

Test Method	Description	Specified Value
ASTM D3042	Acid Insoluble, % min.	70
AASHTO T279	BPN9, min.	32
AASHTO T242	Skid number, min.	40

Friction Texture Application and Retention

There are several different methods and specifications from agency to agency and among researchers to determine a roadways texture retention and texture application. Some agencies do not even have specifications for grinding or tining, and some leave it up to the engineer to determine. There also is not always a universal method for quantifying texture retention, with each state and agency using various methods. These methods typically are similar but change slightly to try and give better accuracy than the previous method. The varying methods in specifications and test methods means it is vital to understand these methods when looking at methods and previous studies.

Test Method Background

There are various testing methods used by individual researchers and state department of transportations to evaluate texture retention for pavements. In terms of true friction resistance, the most widely recognized and commonly practiced is the British Pendulum method (Mahmoud and Masad 2007). This method is used by numerous DOT's and is widely used in many studies as a bench mark for new texture retention methods. The British Pendulum testing apparatus is simply a pendulum device on a frame with a rubber slider on the head of the pendulum. The pendulum arm freely swings about the horizontal axis of the frame. There is a spring connected to the rubber slider inside the pendulum arm connecting to a lever mechanism inside the head of the pendulum. The test works on a loss of energy principle when the pendulum arm swings over the test surface. The kinetic energy is reduced in the pendulum arm and then continues to swing up until it reaches its max position at the end of its swing. The end position is detected by a drag pointer that is mounted and moves along the arm during the swing. Friction is determined by the amount of energy lost during the swing, this is reported as a PVT (pendulum test value) or a BPN (British Pendulum Number). A higher PVT value indicates a higher friction coefficient (Hiti and Ducman 2014). However, some studies have shown that the British Pendulum could vary highly in its results, especially when used on coarse aggregate surfaces (Mahmoud and Masad 2007), (Pin et al. n.d.).

With respect to texture measurements, there are non-contact testing methods that can be used for analysis, such as an AIMS (Aggregate Imagining System) that uses imaging software to interpret the surface characteristics of pavement (Mahmoud and Masad 2007). There have been similar methods developed in other countries as well, such as a method developed in Canada that works by taking a series of stereo photographs with a 35mm reflex camera that analyzes the surface based on macro and micro-texture properties to help determine the texture (Holt and Musgrove 1982). There is also a non-destructive method in Australia called the Twin Laser Profilometer that utilizes lasers capable of taking surface roughness measurements at highway speeds to help determine the macro-texture characteristics of the roadway (Mackey n.d.).

There is also various contact related methods that have been developed by researchers to address different needs or to attempt to improve on the British Pendulum test. Another popular test used by various department of transportations is a Locked-Wheel test, which works with a tow-behind trailer that measures the friction resistance of a locked tire travel at 40mi/hour (Klenke et al. 2015). A dynamic friction tester (DFT) is another popular test that operates by measuring the torque on 3 small pads (Klenke et al. 2015; Rado and Kane 2014). Additionally, there is also a side-force test that again works on measuring skid resistance, this time on perpendicular corning with two tires (Klenke et al. 2015). A couple of spot test texture retention methods include the sand patch test, where the mean texture depth is determined by the diameter of a set of glass beads spread into a circle (Horne and Buhlmann 1983; Klenke et al. 2015), as well as the circular texture meter (CTM) which uses a non-contact laser, matching the path of a DFT test, and determines texture depth (Rado and Kane 2014).

In Australia there is a 3-wheel system called the Gripster test that operates under the same principle as the locked wheel test (Mackey n.d.). There has also been an empirical method developed called the Huang-Hilbert Transformation (HHT) that uses an empirical formula to take decomposed residue from a roadway and use a finite element system to solve for the texture loss, their findings were found to be comparable to those of the DFT and CTM tests on the same specimens (Rado and Kane 2014).

Current ALDOT Specifications and Test Methods

In the state of Alabama, longitudinal grinding specifications for concrete roadways are specified to be 60 blades per foot spaced evenly, with the ridges being a minimum of 1/32” higher than the groove. ALDOT specifies that any grinding equipment must be a self-propelled unit with diamond blades and designed for grinding portland cement pavements. There is no specification by ALDOT for aggregate hardness when grinding surfaces.

ALDOT, along with numerous other state DOT’s, typically use the locked-wheel friction trailer method to determine roadway friction. These DOT’s also typically use profilographs and inertial profiler to measure the smoothness of the pavement. For aggregates, ALDOT uses LA abrasion and sodium sulfate soundness test (5 cycles of sodium sulfate) to determine aggregate quality for use in pavements.

Current Regional Specifications

The Southeast U.S. has a variety of requirements from their respective DOT’s for aggregates allowed and texturing of their pavements. Figure 1 shows a breakdown of how different states all over the country, and especially the Southeast, specify their surface texturing. A numerical breakdown of the grinding specifications is provided in Table 2-6, showing the specifications for all Southern Association of State Highway and Transportation Officials (SASHTO) states with exception of TN, NC, and VA due to their specifications not being published.

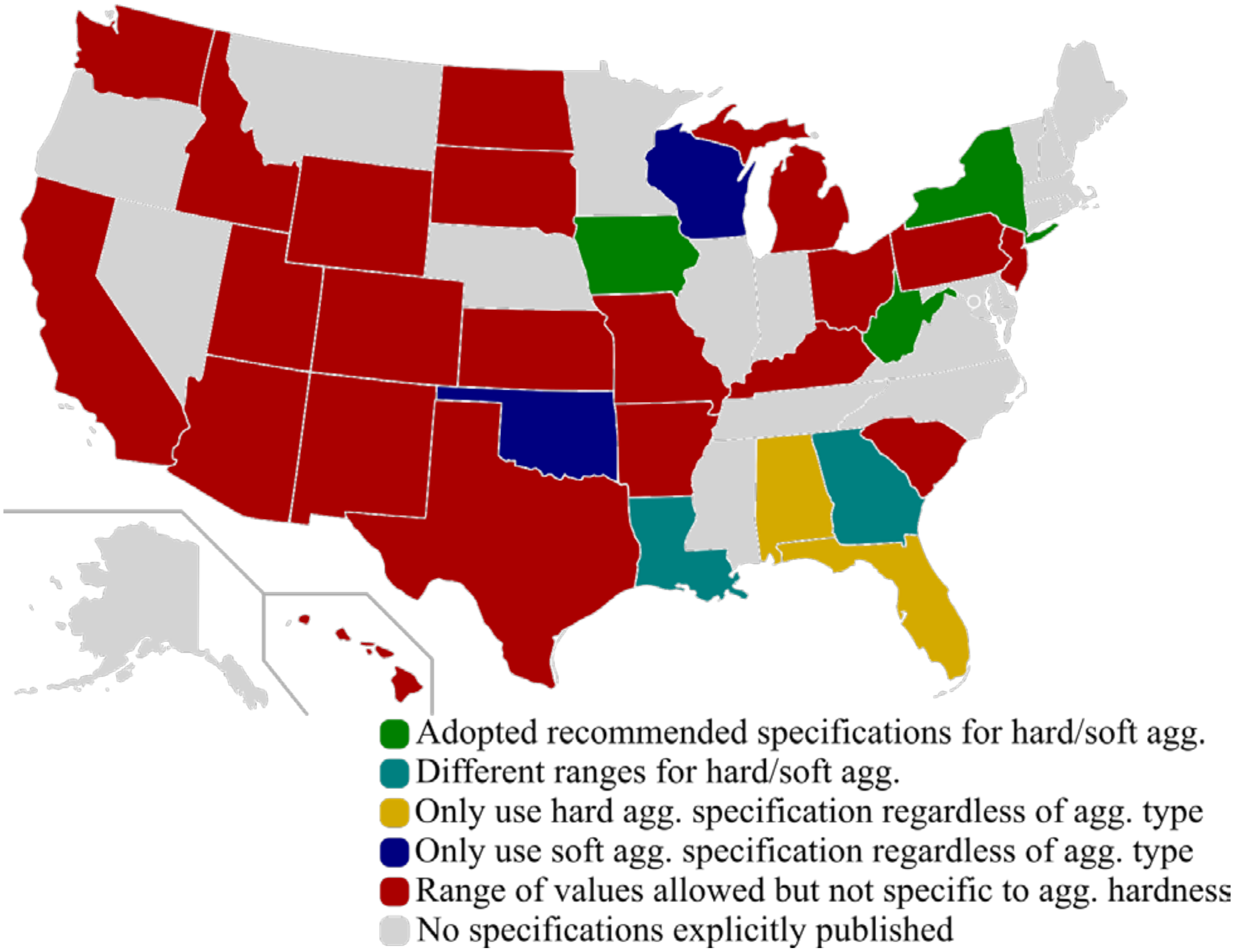


Figure 2-5. Map of state diamond grinding specifications

Table 2-6. Summary of grinding and grooving specifications of southeastern states.

State	Agg.	Land Width (in) Min	Land Width (in) Max	Groove Depth (in) Min	Groove Depth (in) Max	Grooves per foot Min	Grooves per foot Max
Alabama		0.076		0.031		60	
Arkansas		0.087	0.103	0.031		53	57
Florida		0.076		0.031		60	
Georgia		0.076	0.087	0.031	0.094	57	60
Kentucky[1]		0.080	0.125	0.031	0.063	49	59
Louisiana		0.076	0.095	N/a	N/a	55	60
Mississippi		N/a	N/a	0.063		N/a	N/a
South Carolina[1]		0.060	0.125	0.063		49	65
West Virginia[1]	Limestone	0.090	0.120	0.031	0.094	50	56
	Gravel	0.080	0.110			52	59
Texas		0.076	0.117	N/a	N/a	50	60

[1]State specifies groove width which has been converted to a blade per foot for purposes of this table.

In the state of Texas, there is no restriction on aggregate for concrete pavements other than it pass the minimum requirements for thermal expansion, but no reference to hardness or any other value is made unless the aggregate is being used with surface treatments. If used in a surface treatment, the aggregate must meet LA abrasion, micro-deval, sodium sulfate, and gradation requirements. Tining, carpet drag, and grinding are acceptable texturing methods for pavements in Texas. For tining, the tine must be approximately 1/32" thick and 1/12" wide spaced at 1" center to center for transverse tines, and spaced at ¾" for longitudinal tines. Grinding specifications say that is must be circular diamond blades capable of grinding at least 3' width longitudinally each pass with no damage done to the concrete (TxDOT 2004).

The state of Florida has similar specifications to Alabama for grinding, requiring diamond grinding to be 60 blades per foot evenly spaced and the groove height 1/32", and smoothness is tested using a profilograph. However, Florida does allow the use of limestone and dolomite in their concrete pavements providing they have a max loss of 45% on an LA abrasion test and a max loss of 12% on a sodium sulfate test, and are not from a disapproved source specifically listed in the specification manual ("FDOT Standard Specifications For Road And Bridge Construction" 2017).

In their 2017 standards, the state of Mississippi also allows limestone providing it meets the specified requirements. The state of Mississippi does not have any published requirements or regulations on diamond grinding of pavements (Klenke et al. 2015; "Mississippi Standard Specifications for Road and Bridge Construction" 2017).

Tennessee limits carbonate aggregates to only their classified "Type I" roads and can only have a maximum of 32% calcium carbonate content. The state of Tennessee has no published specifications on record for diamond grinding of concrete (TDOT 2015).

The Georgia Department of Transportation allows the use of limestone and dolomite, their "Group I" aggregate classifications, provided that they meet the required wear and soundness requirements for the grading specified in the pavement. These Group I aggregates, however, are not allowed in micro-surfacing mixes. Georgia allows transverse tining of their roadways, however have no published specifications regarding diamond grinding of pavements ("Standard Specifcations Construction of Transportation Systems" 2013).

Brief Overview of Different Textures

In a study the Federal Highway Administration, stated the importance of pavement texture for various aspects of roadway performance. The purpose of the study was to help determine what finishing methods would help provide better pavement friction retention performance. There are numerous different methods of finishing roadways, including broom finish, wire brush finish, burlap drag finishes, and more. It was found that deeper texturing methods improve frictional performance by providing more access for water to escape the roadway instead of pooling up. Transverse grooving of hardened pavements also was found to lead to better performance since water had a quicker escape route, and that longitudinal grooving improves lateral friction (Balmer n.d.).

Finishes are applied in numerous different ways for concrete pavement roadways. Immediately after curing, burlap drag finishes, wire brush finishes, or a broom finish can be applied to the fresh concrete. Finishes may be applied after the setting of the roadway concrete by grinding grooves and tracks into the pavement. There are also new methods used to improve texture, such as a Next Generation Concrete Surface, which is also a type of finished texture applied by grinding(Balmer n.d.).

Diamond grinding is the process of planing the roadway with closely spaced diamond cutting disks that form parallel grooves. These grooves can be longitudinal (with the direction of traffic) or transverse (perpendicular to traffic). Starting with a study done at NASA in 1969 (Horne 1969), longitudinal and transverse grooving both have been shown to bring skid resistance of wet pavements to acceptable numbers, although transverse can be slightly safer since longitudinal can cause vehicle shimmying (Descornet 2000). Diamond grinding can be a cost-effective way to resurface roadways; the same roadway can be reground 2-3 times and grinding may be done on just one lane can be done if necessary. However, grinding does reduce the slab thickness and can cause slab cracking if the contractor is not careful with the depths. The FWRA 2004 recommended dimensions for diamond grinding design was: groove width: 0.125", depth: 0.125"-0.25", and spacing of 0.75" (Caltrans Division of Maintenance 2007).

Transverse diamond grinding has been experimentally shown to reduce the amount of water required to initiate hydroplaning, and that the large widths and close spacing of the grooves can improve the frictional pavements characteristics (Ong et al. n.d.), (Fwa et al. 2006). Studies have shown that transverse grooves typically produce much better results than longitudinal grooves in pavement. However, when diamond grinding was used to introduce the texture, longitudinal grinding was able to achieve and sometimes surpass the performance of transverse tining or grooving (Li et al. 2016). Another advantage of diamond grinding is it can be done on any existing roadway surface, and no previous texturing has any impact on the performance of diamond grinding (Buddhavarapu et al. 2017). Diamond grinding was proven to increase pavement friction numbers from approximately 42 to around 80, a 90% increase, and that diamond grinding can reduce accidents by approximately 42% (Rao et al. 1999a). Another study proved that diamond grinding pavements can provide around a 30% increase in skid number values, due to the better water drainage the grooving allows (Chen and Hong 2015). Overall, diamond grinding is shown to be a good method for increasing skid resistance by increasing macrotexture properties and is proven to have excellent longevity on pavements and can actually lengthen the life of a roadway significantly (Rao et al. 1999b; a).

Longitudinal/Transverse Tining

Tining was first developed to reduce noise in roadways and is one of the most common texturing methods for high speed roadways. A tining texture is applied by taking a metal rake and raking it across the pavement either longitudinal or transversely. The spacing of the tining texture is normally between 10-40mm with spacing between 3-6mm and a depth of 3mm (Neithalath et al. 2005).

Broom/Turf Drag

A broom or burlap drag finish is using either a broom or a piece of wet burlap to drag across wet concrete to introduce texture. These finishes are usually very shallow (Balmer n.d.).

Next Generation Concrete Surface (NGCS)

The Next Generation Concrete Surface (NGCS) texture was developed at Purdue University along with ACPA and IGGA originally to help create a less noisy roadway surface. Multiple studies showed that a NGCS surface produced the quietest ride in terms of on-board sound intensity (OBSI) (Klenke et al. 2015; Scofield 2017).

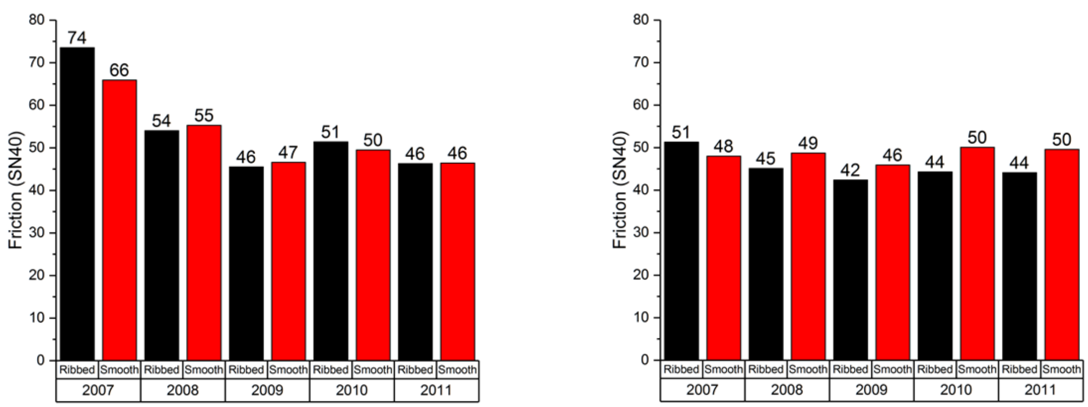
The NGCS texture is similar to diamond grinding, but it cuts deeper grooves (1/8" or deeper) at a wider spacing interval (around ½") as shown in Figure 2-6. The NGCS increases the macrotexture of the pavement and allows more water to escape the roadway through the deeper grooves, helping to improve friction. NGCS is still a relatively new texture method but so far has been shown to improve friction (and road noise) more efficiently than just diamond grinding (Klenke et al. 2015; Scofield 2017).

The NGCS texture has been shown to improved friction performance, even improving on diamond grinding methods. A test section of MnROAD on I-94 was textured with conventional diamond grinding (CDG) and NGCS. The site was tested yearly with a skid trailer and the results (Figure 2-7) show that the NGCS, while having a much lower initial friction value, maintained consistent friction over the course of the study.

In a section of I-70 in Kansas that was tested with multiple surfacing methods, including CDG, longitudinal grooving, tining, and burlap dragged finishes, the NGCS was shown to hold its friction over time as well longitudinal grooving and better than other kind of textures after 7 years, as shown in Figure 2-8 (Scofield 2017). Compared to values recorded from the National Center for Asphalt Technology (NCAT) test track of asphalt samples (with various aggregates) loaded with 10.1M ESALs, it can be concluded that concrete surfaces, especially the NGCS, can hold their friction as well, if not better, as some asphalt mixes (Figure 2-9) and other concrete surface types (Heitzman et al. 2015; Scofield 2017).

Interestingly, in a study on I-355 in the Chicago area, the stopping distance of the NGCS was found to be longer than that of comparable CDG and transverse tining textures (Scofield 2017). For both the wet and dry surface tests, the SUV stopping distance was longest with the NGCS as shown in Figure 2-10. It is possible that the contact area of the tire-pavement interaction is smaller with the NGCS which leads to a higher rate of activation of the anti-lock brake system.

Figure 2-6. Example of NGCS texture characterized by deep grooves with shallow intermediate grooves applied in the longitudinal direction. From (Scofield 2017).



(a) (b)

Figure 2-7: Results from locked wheel friction testing with both ribbed and smooth tires on I-94. Conventional diamond grinding (a) and NGCS (b) results on same test section within MnROAD. The 2007 results represent the as-constructed friction values. Adapted from (Scofield 2017).

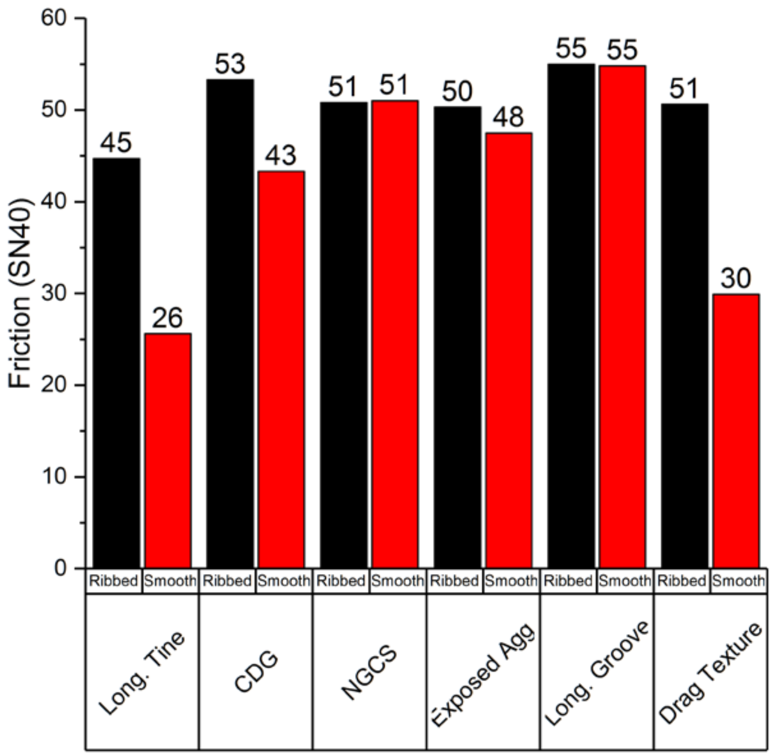


Figure 2-8. Skid trailer friction results of a test section of I-70 in Kansas after seven years of traffic. Adapted from (Scofield 2017).

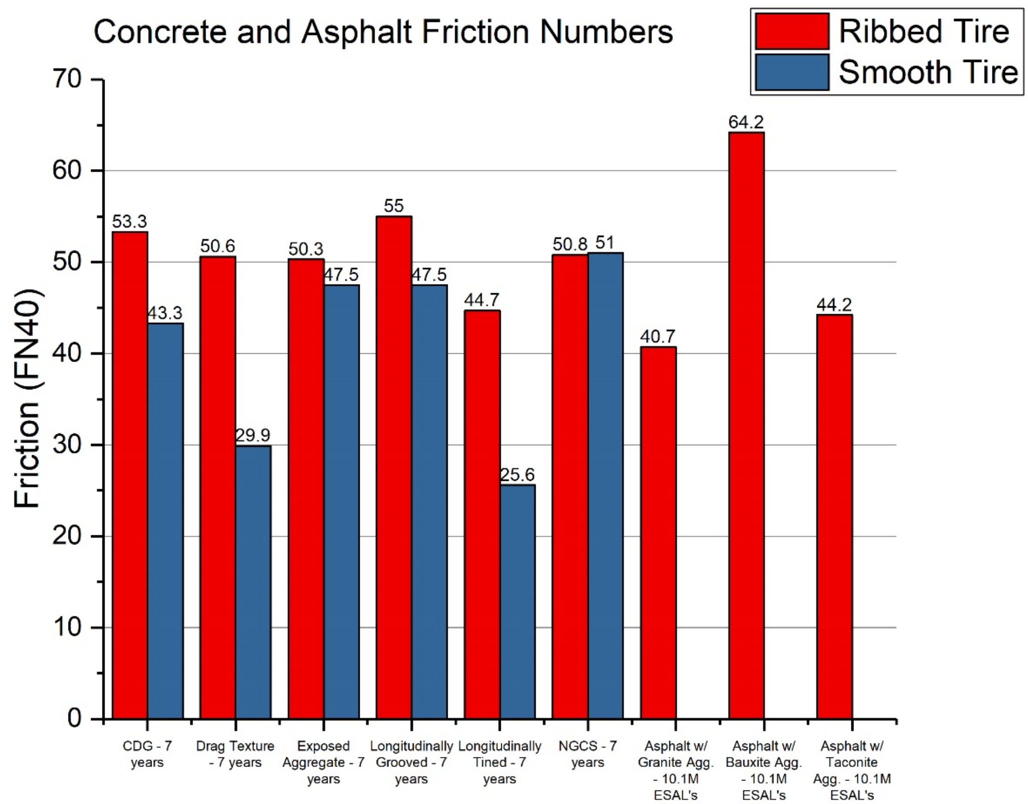


Figure 2-9. Friction number values from both concrete and asphalt pavements. Concrete data is from (Scofield 2017) and asphalt data is from (Heitzman et al. 2015) which is a result of testing at the NCAT test track facility.

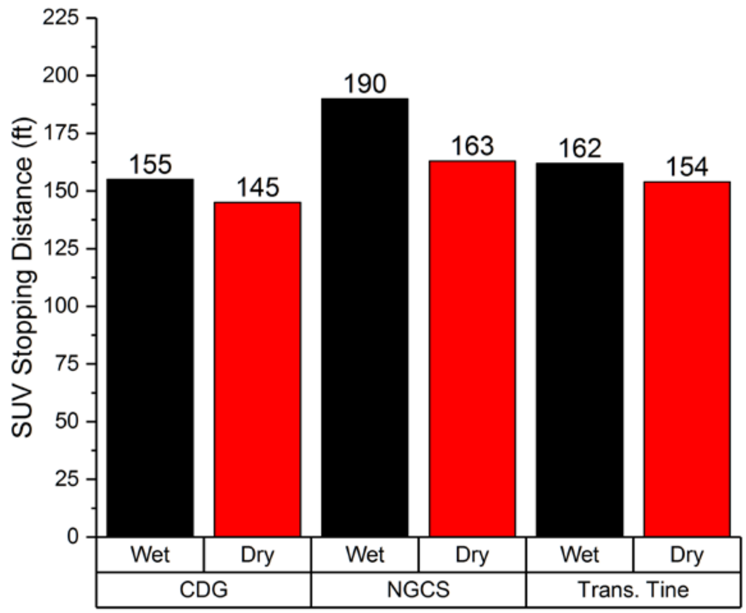


Figure 2-10. Stopping distance of an instrumented SUV on I-355 in Chicago under wet and dry conditions. From (Scofield 2017).

3.0 Experimental Methodology

In this study, concrete pavement laboratory samples were tested for surface frictional properties. Nine (9) different aggregate blends were used in the fabrication of these samples and five (5) different types of textures were applied to these samples. The samples were then used test surface frictional properties. The broad outline has been given in Figure 3-1.

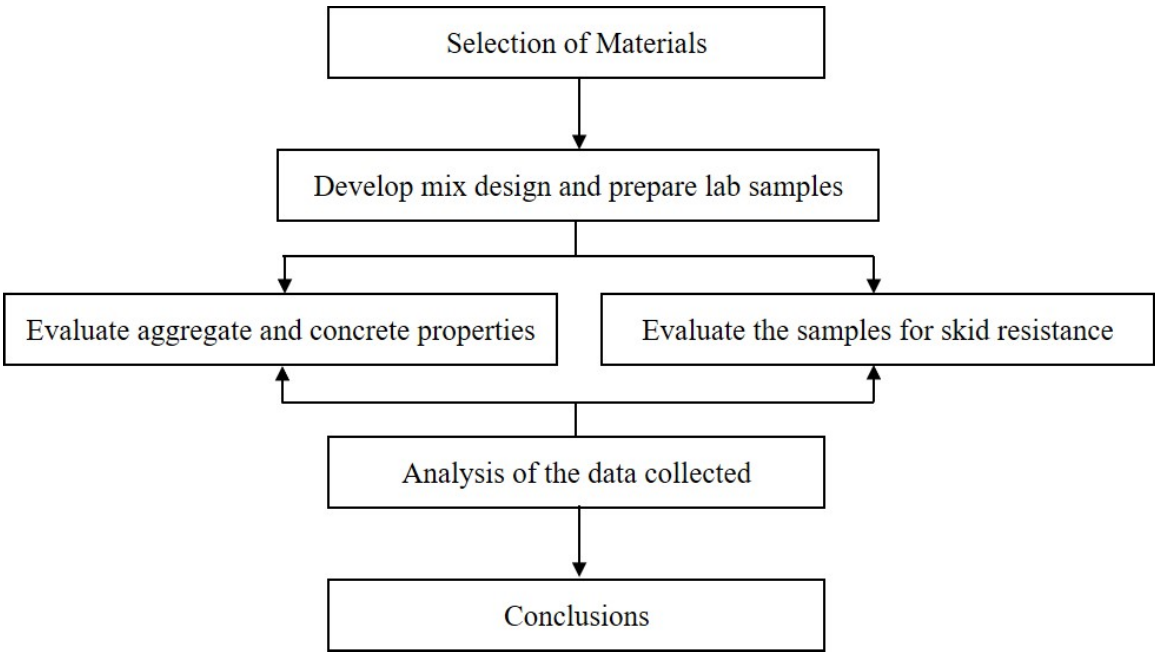


Figure 3-1. Outline of experimental program used for this study.

Selection of Materials

Three common coarse aggregate types used for concrete were chosen from ALDOT approved quarries in the state of Alabama. The three aggregates chosen for this study were: a soft limestone, a hard limestone, and a granite. The sand used for all mixtures was a silica sand from an ALDOT approved quarry. Standard Type I/II portland cement was used for all mixtures with no additional supplementary cementitious materials.

Mixture Design Process

To determine the interaction of various aggregates with each other and their influence on physical properties of concrete, nine (9) different blends of limestone and granite aggregates were used in our experimental mixes. Multiples of each mix were then made for the different texturing applied for each blend.

These nine blends were (any remaining percentages were comprised of the granite aggregate):

- 100% Granite (100Si)

- 100% Hard Limestone (100HLS)
- 100% Soft Limestone (100SLS)
- 75% Hard Limestone (75HLS)
- 75% Soft Limestone (75SLS)
- 50% Hard Limestone (50HLS)
- 50% Soft Limestone (50SLS)
- 25% Hard Limestone (25HLS)
- 25% Soft Limestone (25SLS)

Water-cement ratio of 0.45 (maximum for Alabama pavements) was used along with a cement content of 564 pcy. Coarse aggregates volume of 11.85 ft^3 was chosen based on ALDOT approved mixes for a previous project. Due to a very low cement content of 564 pcy in the mixes, a large amount of superplasticizer was used in each mix, typically adding around 3.0 fluid ounces per 100lbs of cement. Additionally, an amount of 0.1 fluid ounces per 100lbs of cement of air entraining admixture was used to attain the required air content numbers for the ALDOT to consider the mix design usable for roadway concrete. Both the superplasticizer and air entertainer used in this study were from ALDOT approved sources. Nominal maximum aggregate size used was 1 inch and the gradation met the ALDOT standards.

Evaluation of Material Properties

The tests conducted on the materials and concrete used in this study are shown in Figure 3-2.

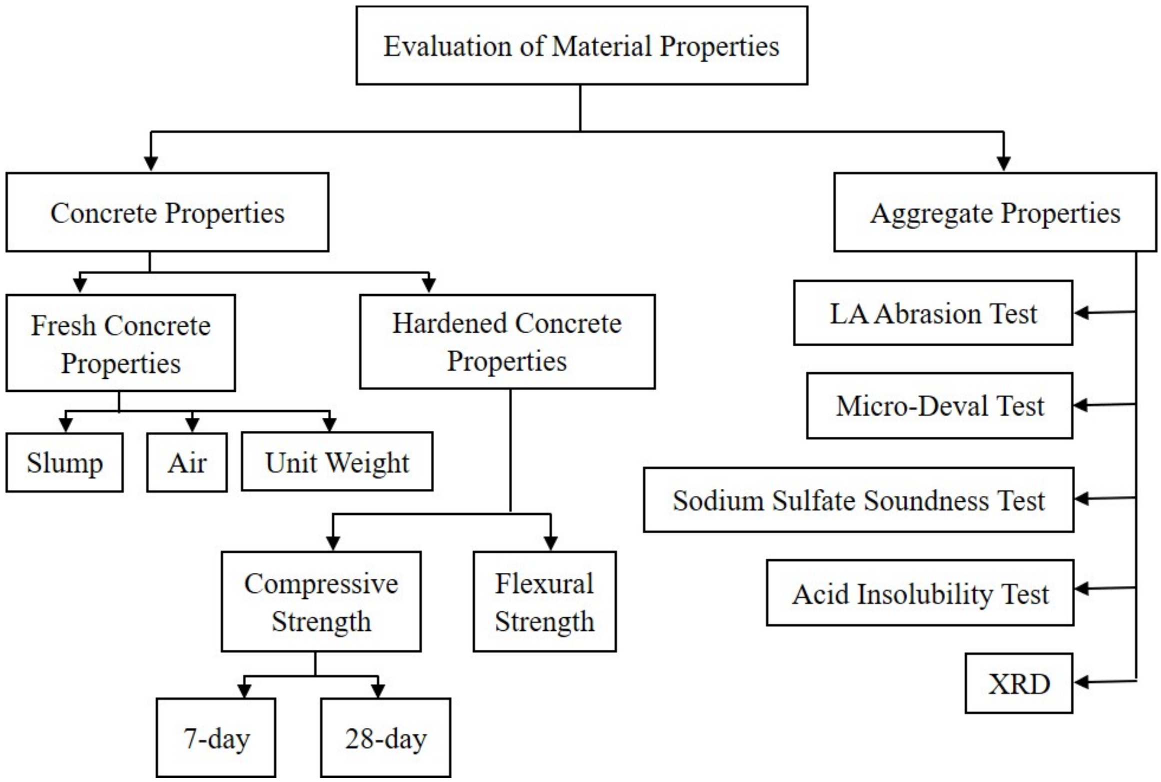


Figure 3-2. Material characterization process used in this study.

Concrete Properties

Each mix design used in this project had a test mix that was used to test the fresh concrete properties as well as the hardened concrete properties to ensure the compliance of these mixes with ALDOT. When relevant, the appropriate ALDOT, ASTM, and/or AASHTO standard test methods are listed.

Fresh concrete data was obtained for each trial mix and slab mix to make ensure each mix was in accordance with ALDOT standards for slump, unit weight and air content. The slump was determined in accordance with AASHTO T119. The unit weight was determined in accordance with AASHTO T121. The air content was determined in accordance with AASHTO T152 as required in ALDOT-170.

Hardened concrete properties were measured using 4 inch by 8 inch cylinders, 6 inch by 12 inch cylinders, and flexural strength beams.

Aggregate Properties

The three aggregates used in this study are different and therefore have various different properties that directly impact the performance of the concrete. Geological and physical properties of aggregates will have an impact on surface texture retention properties. Since the geological properties are rather unchanging, physical properties of the aggregates were tested in this study.

The LA Abrasion test was performed since it is a commonly used test to determine aggregate polishing. This test method uses large steel ball bearings and a very large rotating drum to determine the amount of crushed aggregate when subjected to a certain number or rotations in the drum. This test was performed at the ALDOT Materials Testing Facility in Montgomery, AL in accordance with AASHTO T-96.

The Micro-Deval test is a test that works under the same general principle of LA abrasion, but in much smaller bins and smaller ball bearings. The drum in this test is filled with water and aggregate. It is then rotated a specific number or rotations and at a certain speed, based on the aggregate gradation. The material is then sieved and weighed to determine the amount of loss the aggregate suffered. This test was done at the ALDOT Materials Testing Facility as well in accordance with AASHTO T-327.

The Sodium Sulfate Soundness test was also performed since it is another popular DOT test for freeze-thaw resistance. Sodium sulfate test was performed in compliance with ASTM C88-13. In this test, each aggregate was submerged in a sodium sulfate solution for approximately 16-18 hours and then dried to an oven dry weight, and then re-submerged. Following the recommendations followed by many DOTs, 5 cycles of the test were run and then sieved and weighed the aggregate to determine what percentage of aggregate had been broken down by the solution.

Acid insoluble content was another very important test performed to determine the amount of material resistant to acid in each aggregate type. This allowed in determination of the amount of carbonate in each aggregate type. This test used 6N hydrochloric acid and was performed in compliance with ASTM D3042.

The final aggregate test performed was an x-ray diffraction (XRD) test for true aggregate identification. For this test, each aggregate was ground to a very fine powder passing a #200 sieve. The powder was then scanned at a sufficient rate to allow for accurate and efficient analysis of the signal. A Bruker D8 Discover XRD device was used for all testing. This test allowed in the determination of the exact crystalline structure and mineral make up of each aggregate type used in this study.

Evaluation of Slabs for Surface Frictional Properties

The concrete surface textures were evaluated by using 20 in. X 20 in. X 3.5 in. concrete slabs. Procedure of testing for is shown in Figure 3-3.

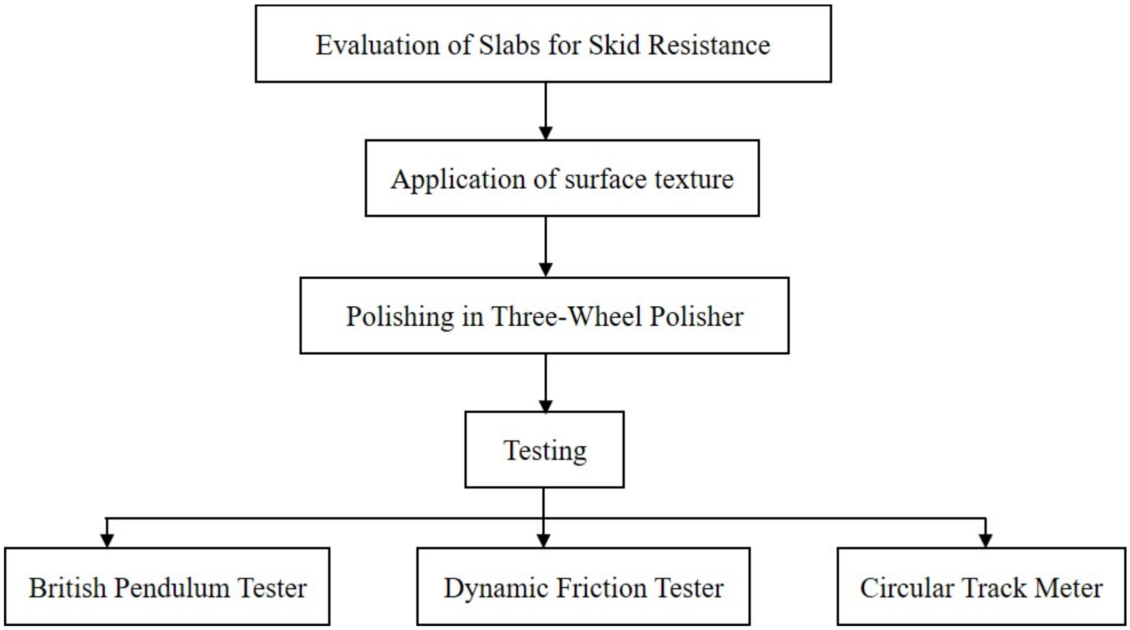


Figure 3-3. Process for evaluating slab skid resistance.

As previously mentioned, 9 different blends of granite and limestone were used for the evaluation of in this study. Each blend then had 5 different surface textures applied and within the same surface texture, two slab replicates were used to estimate the variability in the data collected. Therefore, this experimental program consisted of ten concrete slabs for each concrete mix design with 9 mix designs totaling to 90 concrete slabs. 6 control slabs of 100% granite, 100% hard limestone and 100% soft limestone mixes were also tested to compare with the diamond ground surfaces. Thus, a total of 96 slabs fabricated and tested. Table 3-1 summarizes the test matrix for this part of testing.

Table 3-1. Testing outline and number of specimens for each combination. The mixtures are identified with the following abbreviations: SI (siliceous), HLS (hard limestone), and SLS (soft limestone). When a mixture has only a percentage of limestone listed, the remaining percentage is SI.

Mix Design	Texture						Total
	Control	60 blades/ft	60 blades/ft, grooved	52 blades/ft	52 blades/ft, grooved	NGCS	
100% SI	2	2	2	2	2	2	12
100% HLS	2	2	2	2	2	2	12
100% SLS	2	2	2	2	2	2	12
75% HLS	0	2	2	2	2	2	10
50% HLS	0	2	2	2	2	2	10
25% HLS	0	2	2	2	2	2	10
75% SLS	0	2	2	2	2	2	10
50% SLS	0	2	2	2	2	2	10
25% SLS	0	2	2	2	2	2	10
						Grand Total	96

Application of Surface Texture

The mill-scale grinding machine shown in Figure 3-4 was used to provide grinding and grooving surface textures on concrete slabs. The shaft of this machine is stacked with closely spaced blades. One such fully stacking shaft of the grinding machine is shown in Figure 3-5. Blades are stacked in different ways to achieve different surface textures. In this study, five different surface textures were applied to the pavement specimens: by using different blade-spacer stacking:

- 60 blades/ft (60)
- 60 blades/ft with grooves (60G)
- 52 blades/ft (52)
- 52 blades/ft with grooves (52G)
- Next Generation Concrete Surface (NGCS)

To achieve these textures, different blade-spacer stacking configuration was used on the shaft of grinding machine. Figure 3-6 shows the diamond grinding and grooving operations. The stacking configurations used in this study for the above-mentioned five textures is summarized in Table 3-2. The subsequent subsections discuss these textures in detail.

Table 3-2. Blade stack dimensions used to create surface textures. All blades had a thickness of 0.125 inches.

	Grinding		Grooving	
Texture	Spacer, in	Depth, in	Spacer, in	Depth, in
60 blades/ft	0.11	0.25	N/a	N/a
60 blades/ft, grooved	0.11	0.25	0.63	0.125
52 blades/ft	0.13	0.25	N/a	N/a
52 blades/ft, grooved	0.13	0.25	0.63	0.125
NGCS	0.04	0.03	0.45	0.125

Figure 3-4. Grinding machine used to apply texture to slab specimens. Shown with a fully stacked shaft.

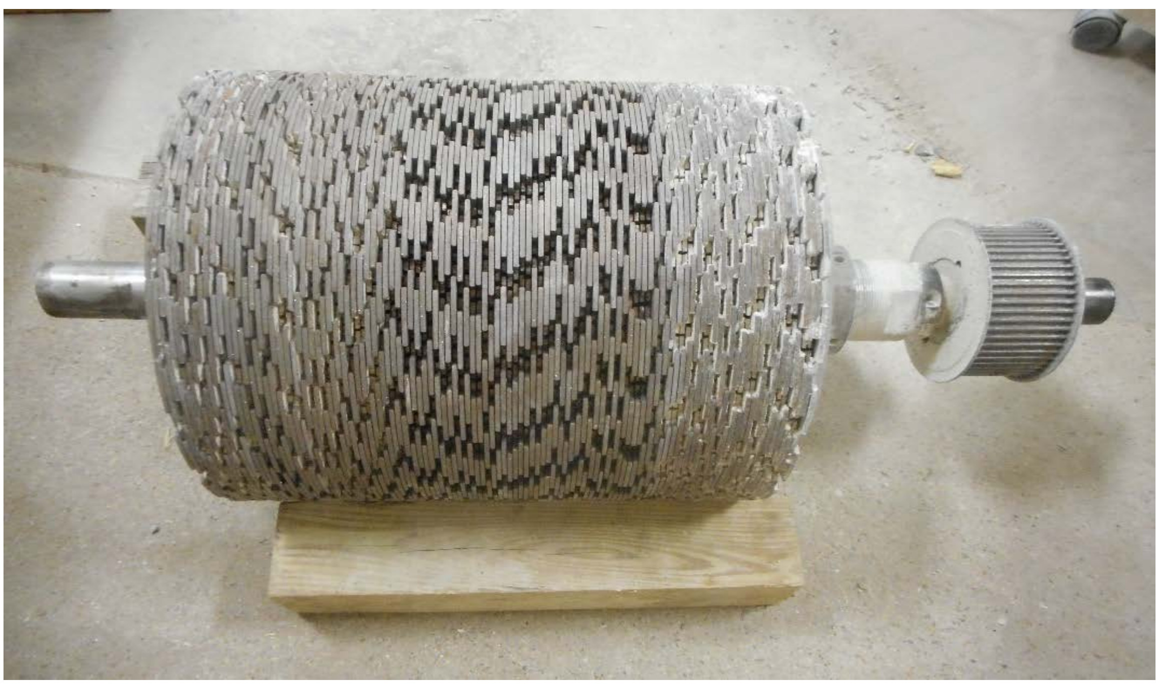


Figure 3-5. Fully stacked shaft close-up with blades and spacers.

Figure 3-6. Before and after images of texture application process. The image on the left shows slabs with as-is texture and the image on the right shows slabs with the applied texture.

In the diamond grinding process, paste content to a depth of 1/4 inch from the surface was removed to expose the coarse aggregates. The speed of grinding was controlled by an electric wench and maintained a constant rate of 2 inches/min to both produce the desired concrete surface texture and to minimize the excessive stress on spacers, diamond blades, and the belt connecting the motor and the shaft. Out of the five textures used in this study two fall in diamond grinding only category, 60 blades/ft and 52 blades/ft.

Diamond grooving is a two-phase job, firstly the slabs were ground by the diamond grinding process and then grooved to a depth of 1/8 inch to drain off the surface water. For grooving, 0.630 in. width of spacers were used, and speed of grooving was 5 inches/min. As little surface is being cut and only smaller depth is removed, diamond grooving is faster than diamond grinding. Two types of surface textures used in this study fall in this category, 60 blades/ft with grooves and 52 blades/ft with grooves.

Like diamond grooving, NGCS is also a two-phase job. Firstly, the slabs were flush ground to a depth of 1/32 in. by using diamond blades with 0.040 in. spacers. Then, 1/8 in. deep grooves spaced at 0.450 in. were applied to the slabs.

Polishing

A Three-wheel polishing device (TWPD) developed by National Center for Asphalt Technology (NCAT) was used to wear the specimen surface similar to abrasion from the traffic loading in the field. This device, as shown in Figure 3-7, consists of three wheels which traces a circular path. For this study, the device was modified based on a previous study (Whitney et al. 2013) by using 2 inch wide polyurethane wheels and applying a total load of 225 lbs. on the wheels. The slabs were then exposed to 160,000 polishing cycles at a polishing speed of 40 cycles/min with a continuous flow of water on the slabs surface to remove the debris. It has been previously shown

(Whitney et al. 2013) that extending the test beyond 160,000 cycles provides little useful data. of the slab surfaces were tested after 10,000, 40,000, 100,000 and 160, 000 polishing cycles. Figure 3-8 shows picture of a slab surface after these testing intervals.

Figure 3-7. Three wheel polishing device (TWPD) developed by NCAT which was used for this study. The NCAT version was modified as described in (Whitney et al. 2013).

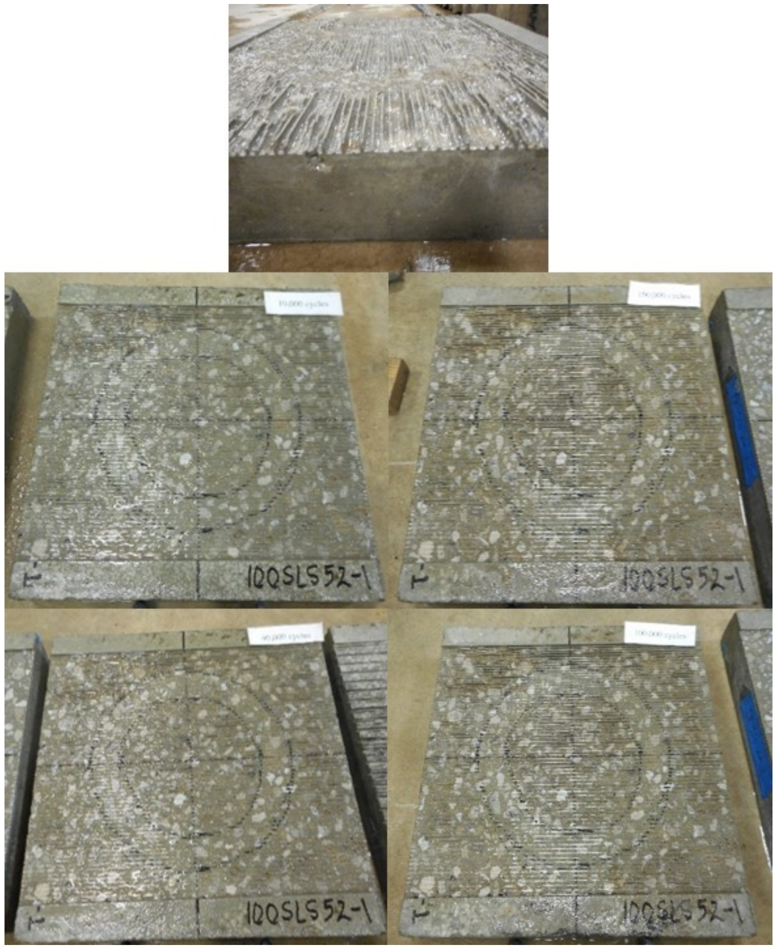


Figure 3-8. Example of textured slab being polished to different polishing intervals; top: surface texture; top-left: after 10k cycles; bottom-left: after 40k cycles; bottom-right, 100k cycles; top-right: 160k cycles.

Friction and Surface Texture Testing

In this study, of the slabs was measured using 3 devices: British Pendulum Tester (BPT), Dynamic Friction Tester (DFT) and Circular Track Meter (CTM). In the field conditions, diamond grinding creates a texture that is parallel to the direction of the travel. However, TWPD travels in a circular path and the other two devices used in this study, DFT and CTM, also test on the circular polished path. The DFT and CTM results therefore might not be comparable to the field conditions. Thus, BPT was used to test at two locations where the texture is parallel to the circular polishing path (Figure 3-9).

BPT has an arm which has a rubber slider that drops from a known height and measures the energy lost when the arm strikes the test surface. Due to the small size of the specimen a T-shaped frame was used for the feet of BPT to rest. Also, because of small test surface available, the standard test method (ASTM E303) used for the measurement of frictional properties was modified by reducing the contact length from 5 in. to 3 in. and using 1.25 in. rubber sliders instead of 3 in. rubber sliders. A correlation between standard test method and modified method was developed by testing five different surfaces. A good linear correlation was seen between the two methods as shown in Figure 3-10. Modified BPT readings were taken at 10,000, 40,000, 100,000 and 160,000 cycles. To minimize variability in the data, three readings were taken at each of the two locations where the texture is parallel to the polishing wheels.

Figure 3-9. British pendulum tester (BPT) setup on a ground and polished slab specimen.

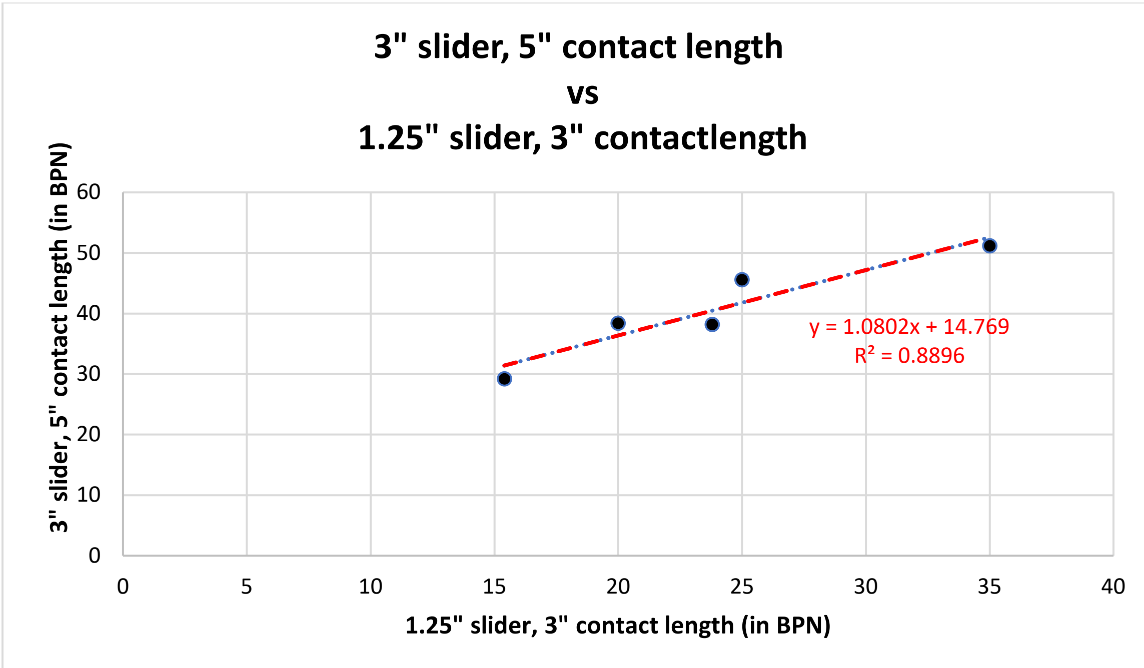


Figure 3-10. Correlation between standard BPT and the modified test method with a reducer slider and contact length.

ASTM E1911 was used to determine frictional properties of concrete slabs using the DFT. The DFT, as shown in Figure 3-11, has three rubber sliders mounted on a rotating disc that travel the same path as the TWPD wearing track. The rotating disc is attached to a spring and is spun to a speed of 80 km/h. Once the rotating sliders are spinning at 80 km/h, they are applied to a wetted pavement surface. The differential slip in the spring balance was measured and is used to obtain the coefficient of friction of the surface. The DFT measures the coefficient of friction at varying slip speeds ranging from 0 to 80 km/h on a pavement.

In this study, DFT was used to measure the frictional properties of the surface at 10,000, 40,000, 100,000, 160,000 polishing cycles. To reduce the variability in the data, five readings were taken at each testing interval.

To measure the mean profile depth (MPD) of the specimen surface, CTM (Figure 3-12) was used in this study. Similar to the DFT, this is a laboratory-scale profiler also travels the same path as the wearing wheels of the TWPD. The CTM has a laser-displacement sensor mounted on a rotating arm and measures the macrotexture profile along the circumference of the circle. The measured macrotexture profile is divided into eight equal segments and the MPD for each segment is in accordance with ASTM E 1845-09. The reported MPD values in this study are the average MPD from all segments. In this study, CTM was used to measure the MPD value of the surface at 160,000 polishing cycles. Three readings were taken for each slab to reduce variability due to testing.

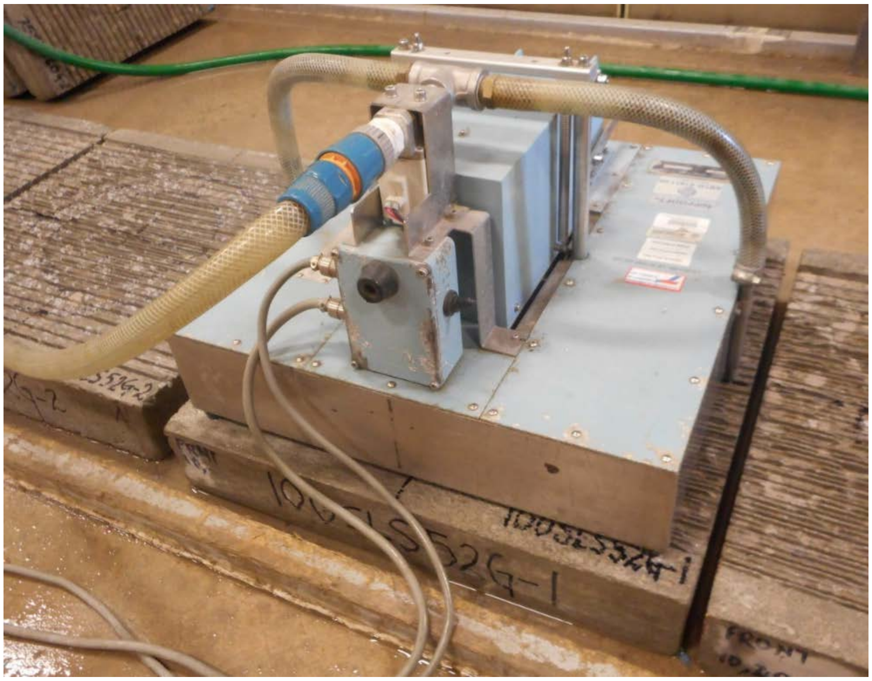


Figure 3-11. Dynamic friction tester (DFT) setup on a ground and grooved slab specimen.

Figure 3-12. Circular track meter (CTM) setup on a ground slab specimen.

4.0 Results and Analysis

Aggregate Properties

The results obtained from the four tests (Acid insolubility test, Sodium sulfate soundness test, LA abrasion test and Micro-Deval test) conducted in this study on aggregates are summarized in Table 4-1.

Table 4-1. Results from aggregate durability characterization tests.

	Acid Residue, % insoluble	Sulfate Soundness, % retained	LA Abrasion, % loss	Micro-deval, % loss
SLS (Soft Limestone)	8.17%	99.37%	37.16%	12.97%
HLS (Hard Limestone)	0.61%	99.44%	30.14%	8.08%
SI (Granite)	98.62%	99.47%	47.96%	9.31%

The XRD testing of each aggregate sample allowed us to determine the chemical composition of the aggregates used in our study. Based on the diffraction spectra of the aggregate samples at various scanning phase angles, as indicated by the peaks in the signal recorded by the machine (Figure 4-1), the three aggregate types were found to be:

- Granite- Quartz
- Hard limestone- Dolomite
- Soft limestone- Calcium carbonate with some dolomite (dolomitic limestone)

Based on this, it was found that the hard limestone used in this study was actually dolomite and the soft limestone was a type of dolomitic limestone. The granite source used in this study is not a pure granite (i.e. quartz) but likely a type of gneiss given the presence of albite.

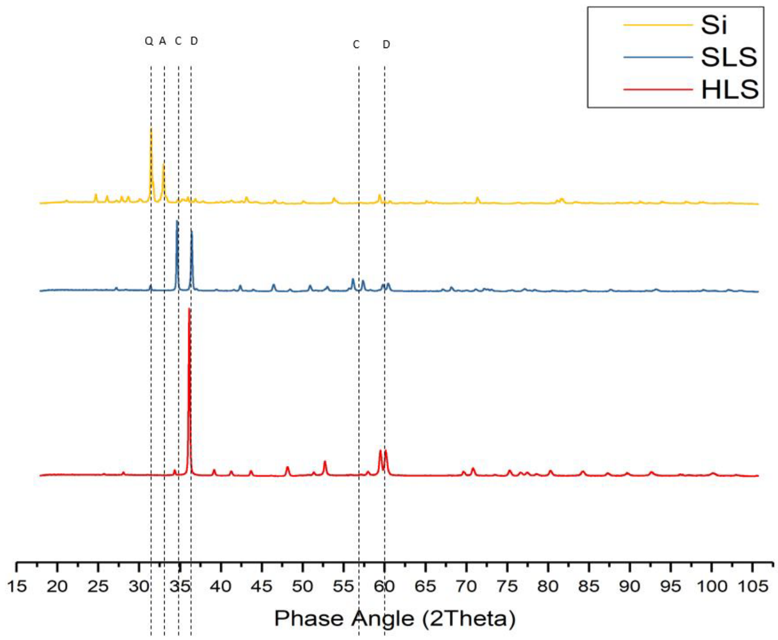


Figure 4-1. X-ray diffraction (XRD) spectra for all three coarse aggregates used in this study. (Q) denotes quartz, (A) denotes albite, (C) denotes calcite, and (D) denotes dolomite.

Fresh and Hardened Concrete Properties

The fresh concrete properties for each mixture tested are shown in Figure 4-2, Figure 4-3, and Figure 4-4. The slump values change between aggregate sources due to the differing gradations. The volume of coarse aggregate for all mixtures was constant but the actual gradation of each source was different. As previously mentioned, an air-entraining admixture was used for all mixtures (Figure 4-3).

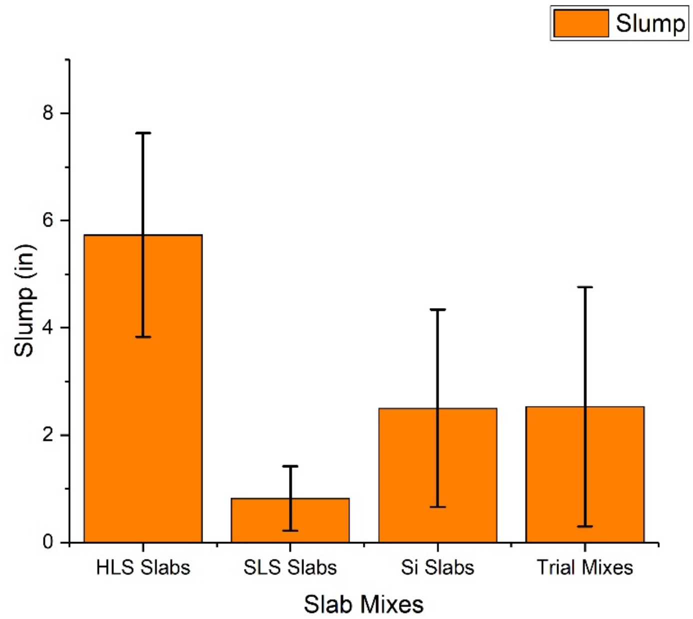


Figure 4-2. Slump values for all mixtures tested. The differences between aggregate sources is due to the differing gradations of the sources.

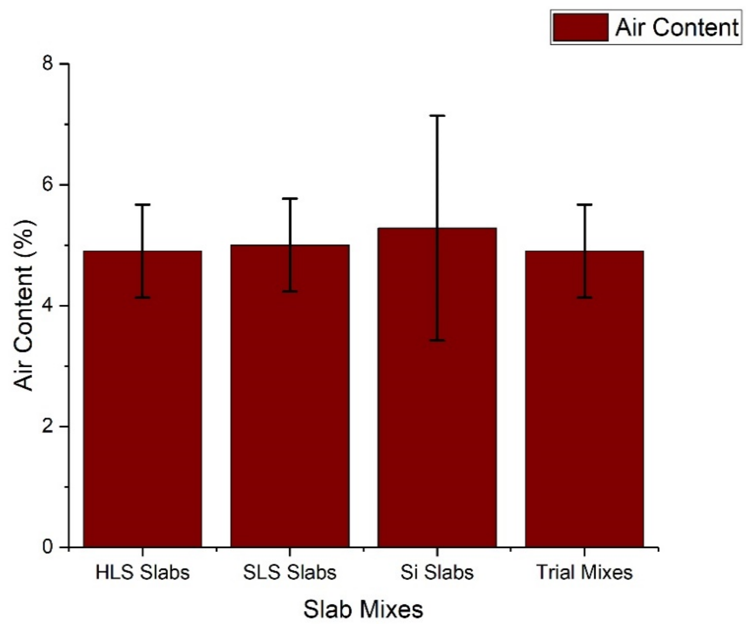


Figure 4-3. Air content values for all mixtures tested. As mentioned previously, an air-entraining admixture was used for all mixtures.

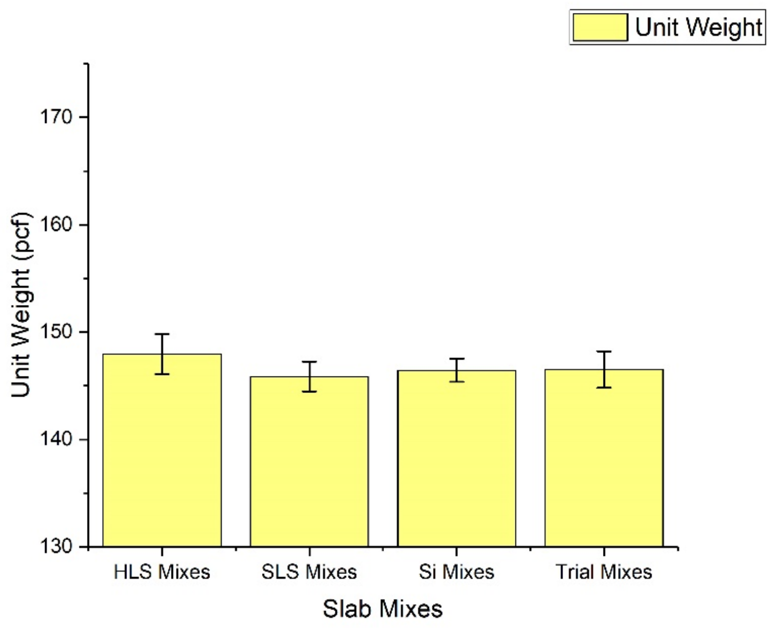


Figure 4-4. Unit weight values for all mixtures tested.

The hardened concrete properties were also characterized for all mixtures at 28 days as seen in Figure 4-5, Figure 4-6, and Figure 4-7. While there was some variability between certain mixtures, all mixtures met ALDOT paving specifications.

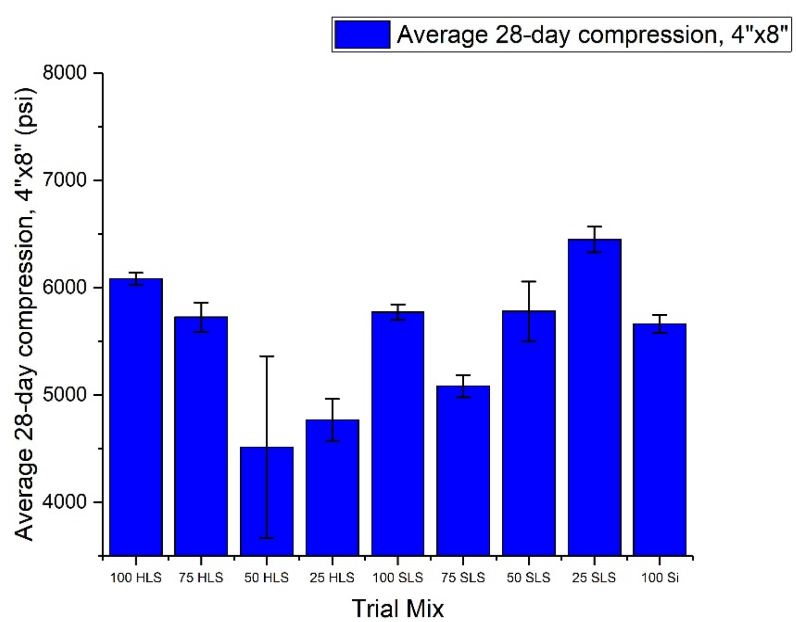


Figure 4-5. Average 28-day compressive strength for all mixtures tested. Specimens were 4" by 8" cylinders and were moist cured until the time of testing.

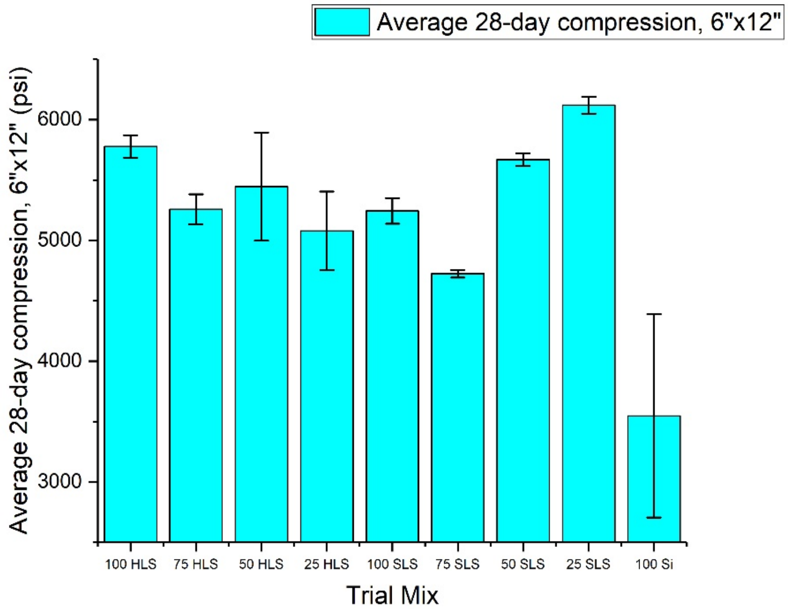


Figure 4-6. Average 28-day compressive strength for all mixtures tested. Specimens were 6” by 12” cylinders and were moist cured until the time of testing.

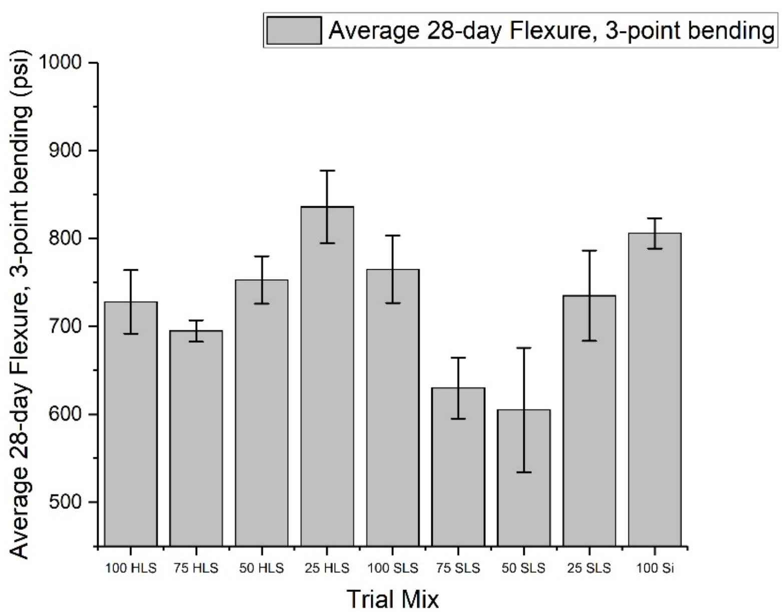


Figure 4-7. Average 28-day flexural strength for all mixtures tested.

Analysis of Polishing

A total of 96 slabs were analyzed for in the laboratory. As previously mentioned, the naming scheme is the percentage of the shown aggregate (e.g. 50SLS indicates 50% soft limestone with the remaining coarse aggregate being granite) and the next number and/or letter indicates the blade spacing and whether grooving is present (e.g. 50SLS-52G indicates 50% soft limestone with grinding at 52 blades/ft and grooving, G, present). One slab (75SLS-52) was damaged during handling, therefore, could not be analyzed. One slab (100SLS-52G) was extra, therefore was tested and used in the data analysis.

British Pendulum Testing

The BPT data obtained from the modified BPT test method is tabulated in Appendix A. The BPN values obtained from the replicate slabs were averaged in the data analysis. A significant amount of scatter is seen in the data. One example of the scatter in the BPT data is shown in Figure 4-8 for the Next Generation Concrete Surface (NGCS) texture with the number of polishing cycles for all the nine aggregate blends used in this study. A non-linear logarithmic regression model was used to fit the data. It can be noted from the plot that for some aggregate blends, the BPN values were increasing with increase is polishing indicating the increase in friction with more polishing. These erroneous results might be due to a very small testing area of 1.25 in. x 3 in. on the slab, which might not be a good representation of the whole slab. Therefore, it can be said that BPT is not a valid assessment tool for quantifying the polishing of pavements in the laboratory.

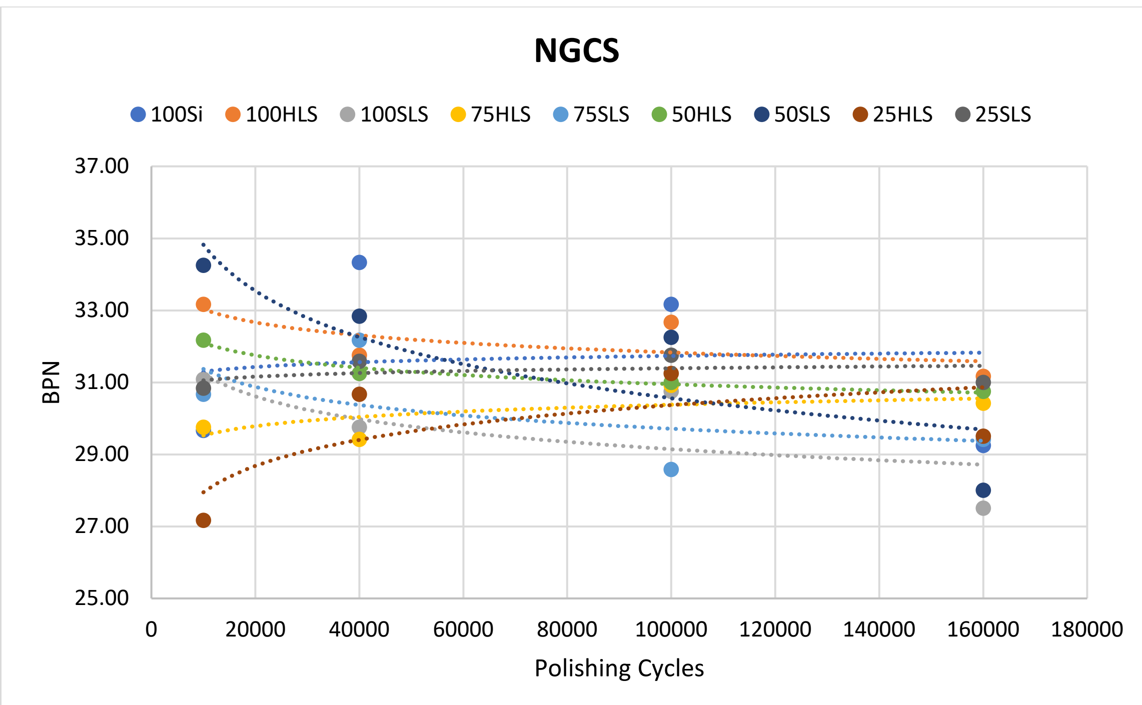


Figure 4-8. Variation of BPN values for NGCS specimens at various polishing cycles and aggregate blends.

Dynamic Friction Testing

The DFT data is tabulated in Appendix B. The dynamic friction value obtained from the DFT at 60 km/h was used to analyze the data. The replicate slabs were averaged for the data analysis. Slabs marked in grey color in the table were not used in the data analysis as the DFT was faulty during their testing.

The DFT data appears to be statistically better than the BPT data. One reason may be that the DFT operates in a circular movement that follows the polishing path while the BPT is a linear measurement. Also, DFT testing area is much larger than the BPT test area to produce better results. Figure 4-9 through Figure 4-13 shows the variation of DFT60 values (Dynamic friction at 60 km/h) with the polishing cycles for 5 different textures analyzed in the study. A non-linear logarithmic regression model was used to fit data. A good trend of decreasing friction values with increasing polishing cycles can be observed for all the slabs except one (25SLS-60). Hence, DFT can be assumed as a good tool for quantifying the polishing of laboratory specimens.

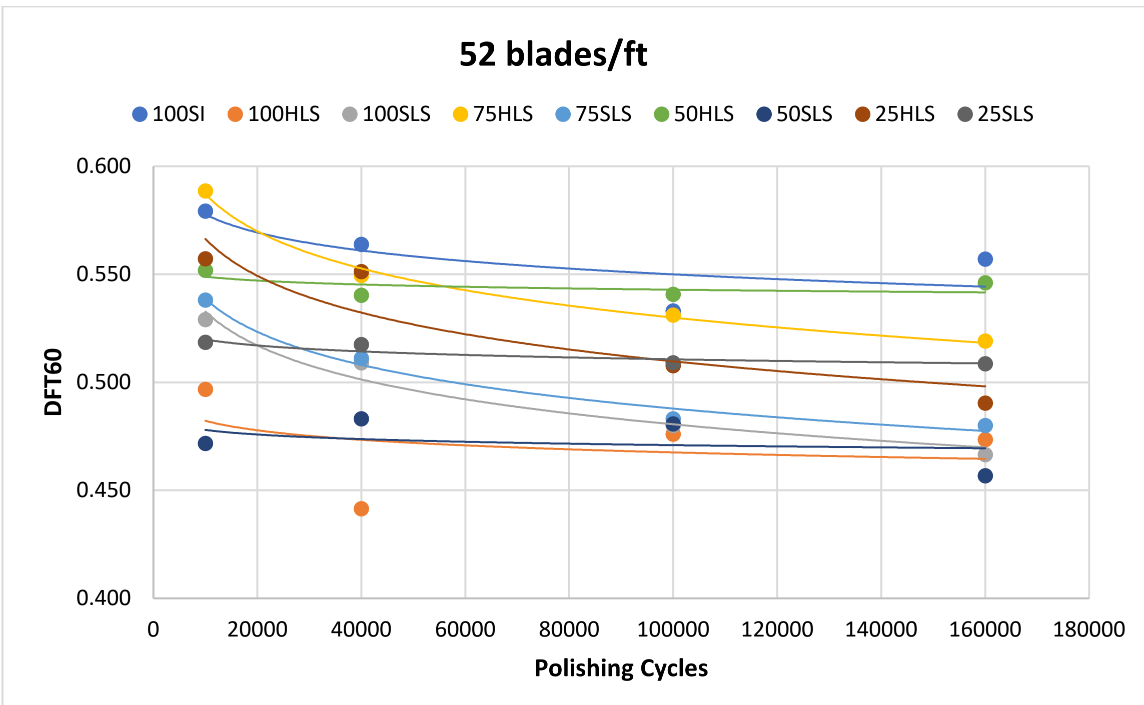


Figure 4-9. Variation of DFT60 values for various polishing cycles and aggregate sources with the grinding texture applied with 52 blades/ft.

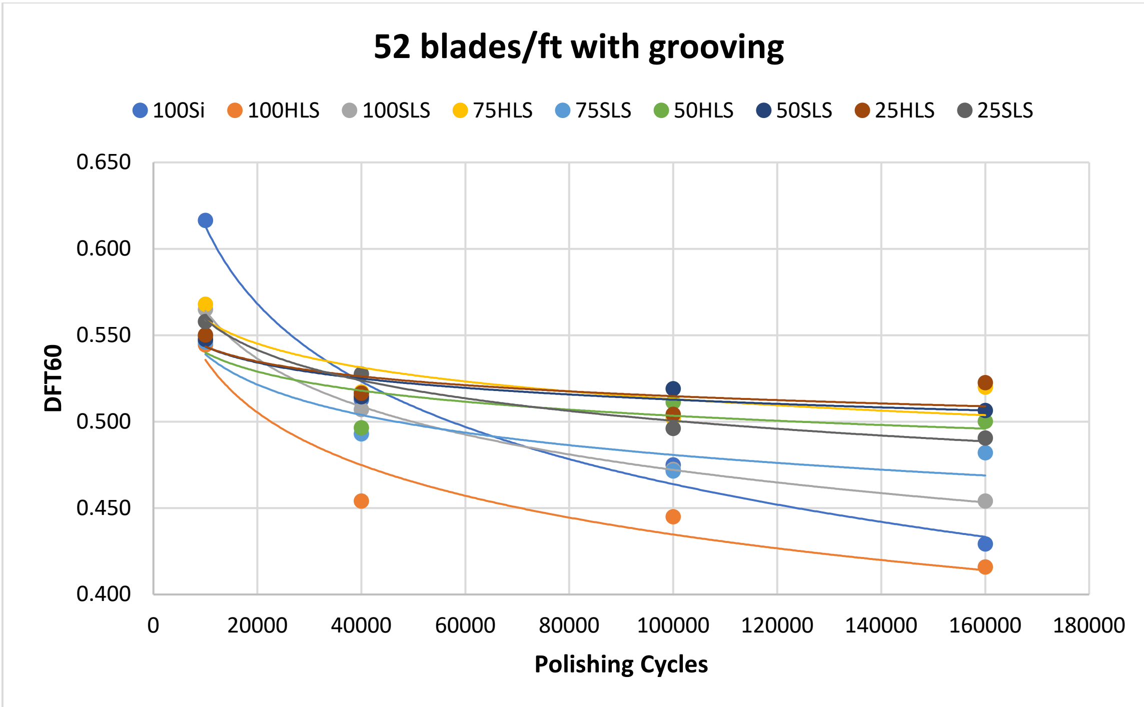


Figure 4-10. Variation of DFT60 values for various polishing cycles and aggregate sources with the grinding texture applied with 52 blades/ft and grooving.

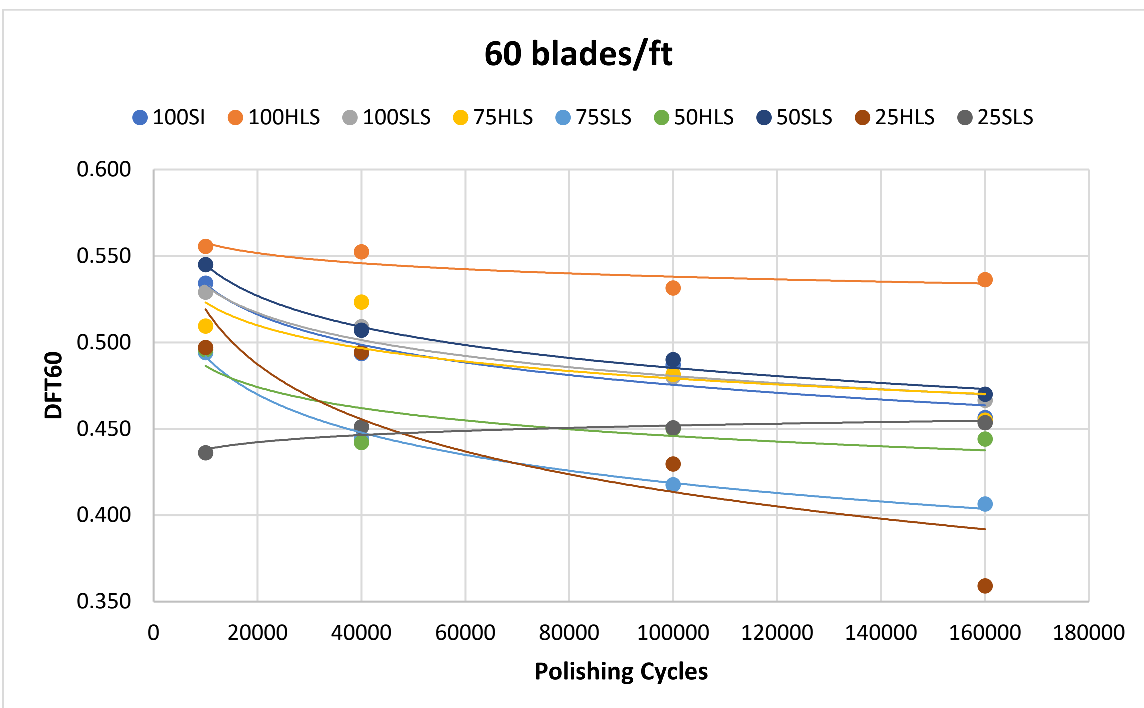


Figure 4-11. Variation of DFT60 values for various polishing cycles and aggregate sources with the grinding texture applied with 60 blades/ft.

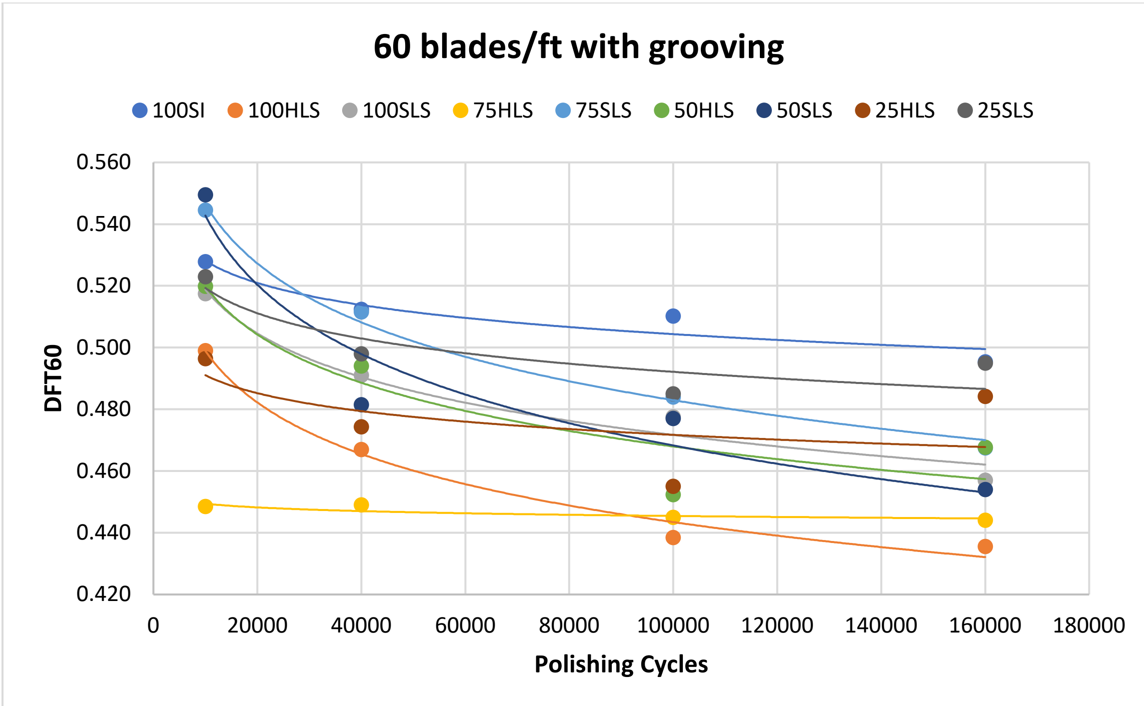


Figure 4-12. Variation of DFT60 values for various polishing cycles and aggregate sources with the grinding texture applied with 60 blades/ft and grooving.

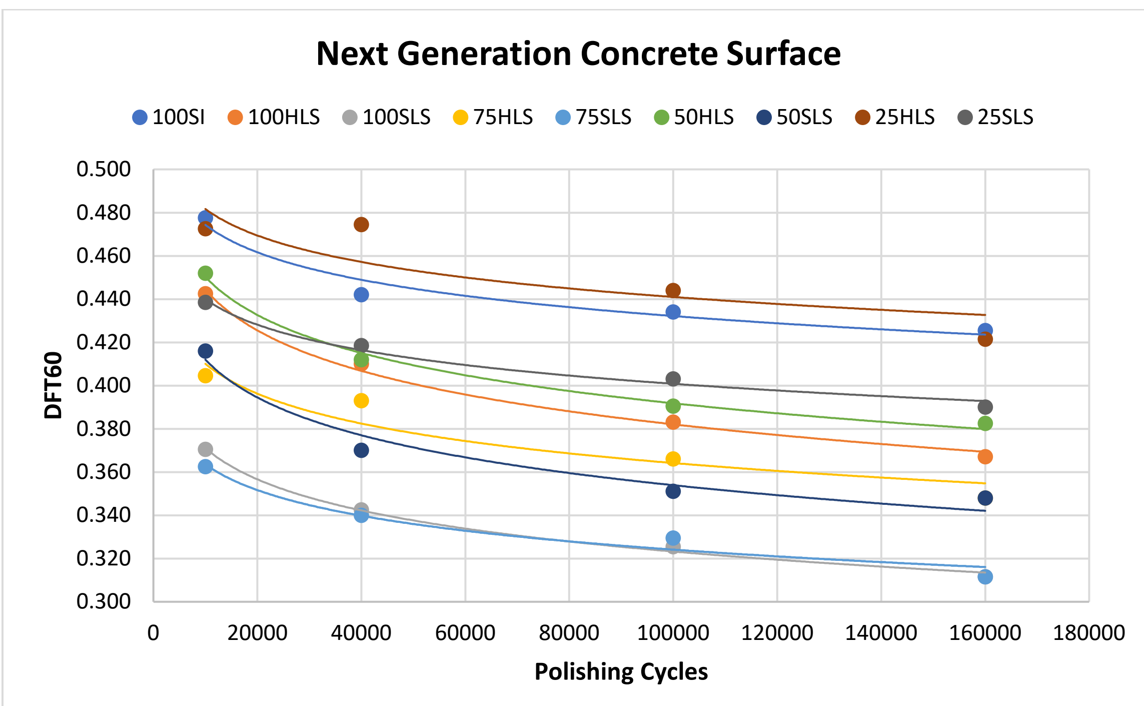


Figure 4-13. Variation of DFT60 values for various polishing cycles and aggregate sources with the NGCS texture.

Initial Friction

In this study, initial friction of the specimens has been assumed to be at 10,000 polishing cycles. Figure 4-14 through Figure 4-16 show the initial friction values based on different blends and textures. Although it might seem from Figure 4-14 that all the blends have comparable initial friction values, Figure 4-15 gives a closer analysis of the initial performance of various blends. Figure 4-15 shows the variation of DFT60 values at 10k polishing cycles with percent of siliceous aggregates in the aggregate blend for all the textures. A straight line was used to fit the data to get the idea of general trend. It can be seen from Figure 4-15 that typically the DFT60 values increase with the increase in siliceous content in the aggregate blend, albeit only slightly in some cases.

From Figure 4-16 it can be observed that all the textures did not produce comparable friction values. Plain, unground surfaces had lowest values of friction followed by Next Generation Concrete Surface. The other four conventional grinding and grooving textures (52 blades/ft, 52 blades/ft with grooves, 60 blades/ft and 60 blades/ft with grooves) produce comparable initial friction values.

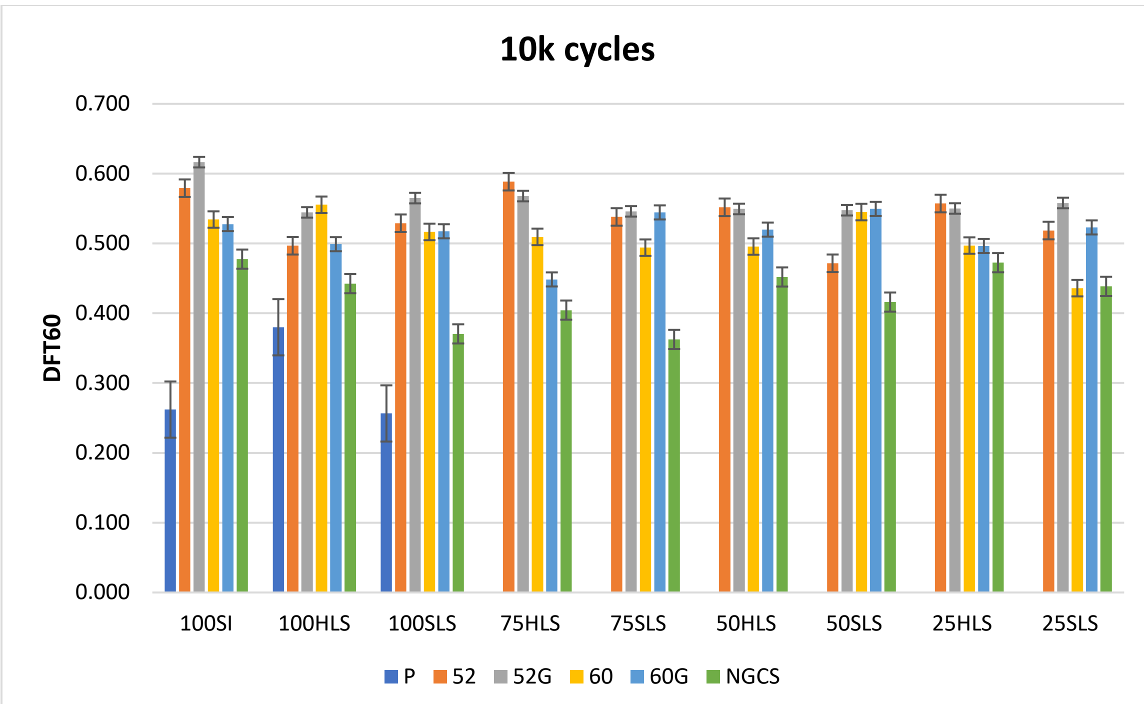


Figure 4-14. Initial friction values after 10k cycles in TWPD for all aggregate blends tested. (P) represents as-cast broom texture only used for the baseline specimens.

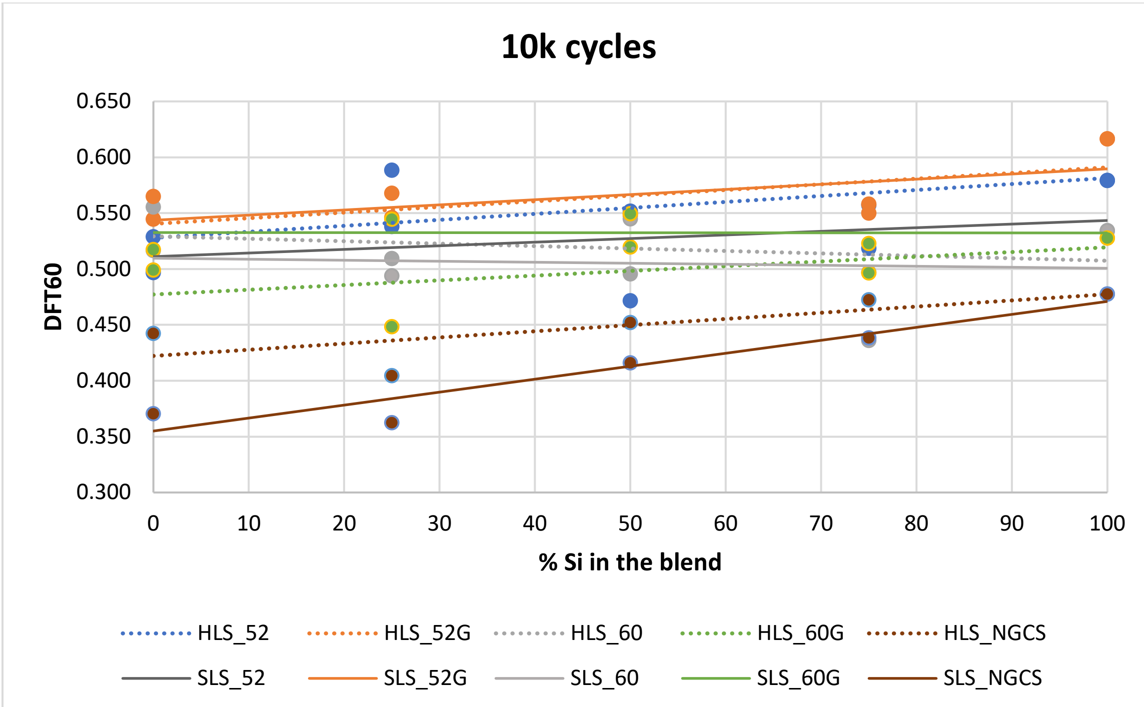


Figure 4-15. Variation of DFT60 values at 10k cycles as a function of the percentage of granite, SI, in the aggregate blend.

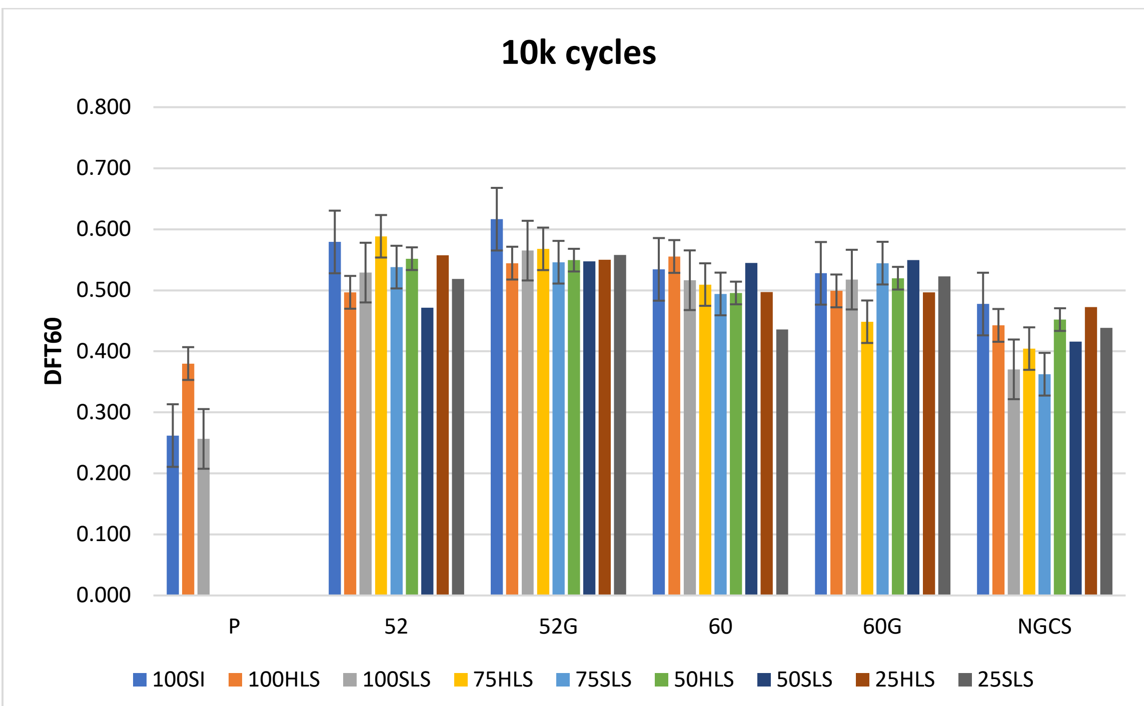


Figure 4-16. Initial friction values after 10k cycles in TWPD for all textures tested. (P) represents as-cast broom texture only used for the baseline specimens.

Final Friction

The final friction values in this study is assumed to be at 160,000 polishing cycles. Figure 4-17 through Figure 4-19 show the final friction values based on different aggregate blends and textures. Figure 4-17 gives a general performance of the aggregate blends and Figure 4-18 gives a closer examination of the performance of the aggregate blends. Figure 4-18 represents the variation of DFT60 values for different textures with the percent of siliceous aggregate in the aggregate blend. Linear regression was performed to fit the data to get an idea of the general trend. As can be seen from Figure 4-18, as the siliceous content increases in the mix, typically the friction values increase.

Figure 4-19 shows the variation of final friction values with different texture. It can be seen that the plane surfaces and NGCS have low friction values as opposed to other surface textures. Additionally, compared to other textures 52 blades/ft produces highest friction values.

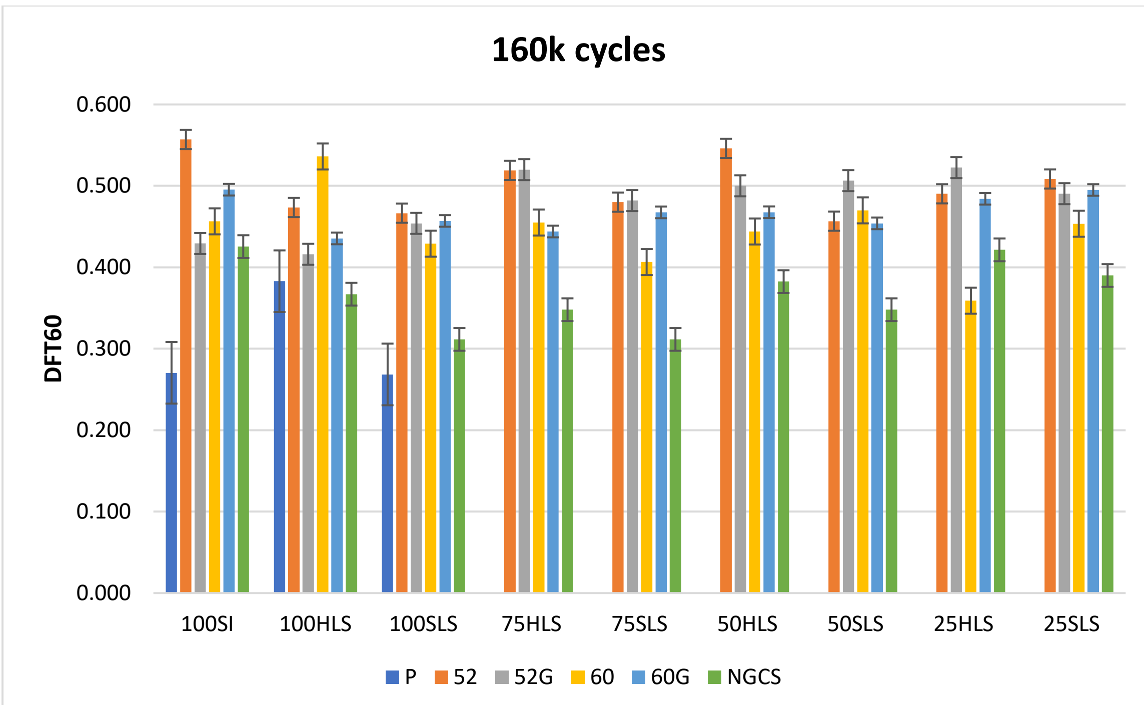


Figure 4-17. Final friction values after 160k cycles in TWPD for all aggregate blends tested. (P) represents as-cast broom texture only used for baseline specimens.

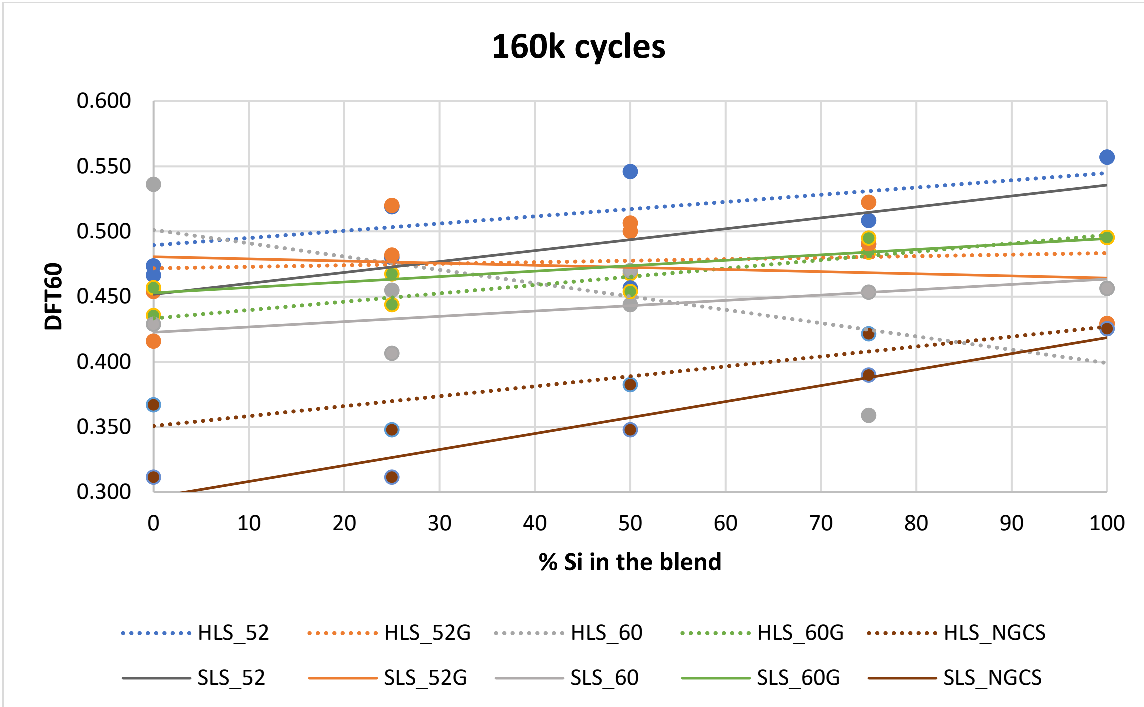


Figure 4-18. Variation of DFT60 values at 160k cycles in TWPD as a function of granite, SI, percentage for all textures tested.

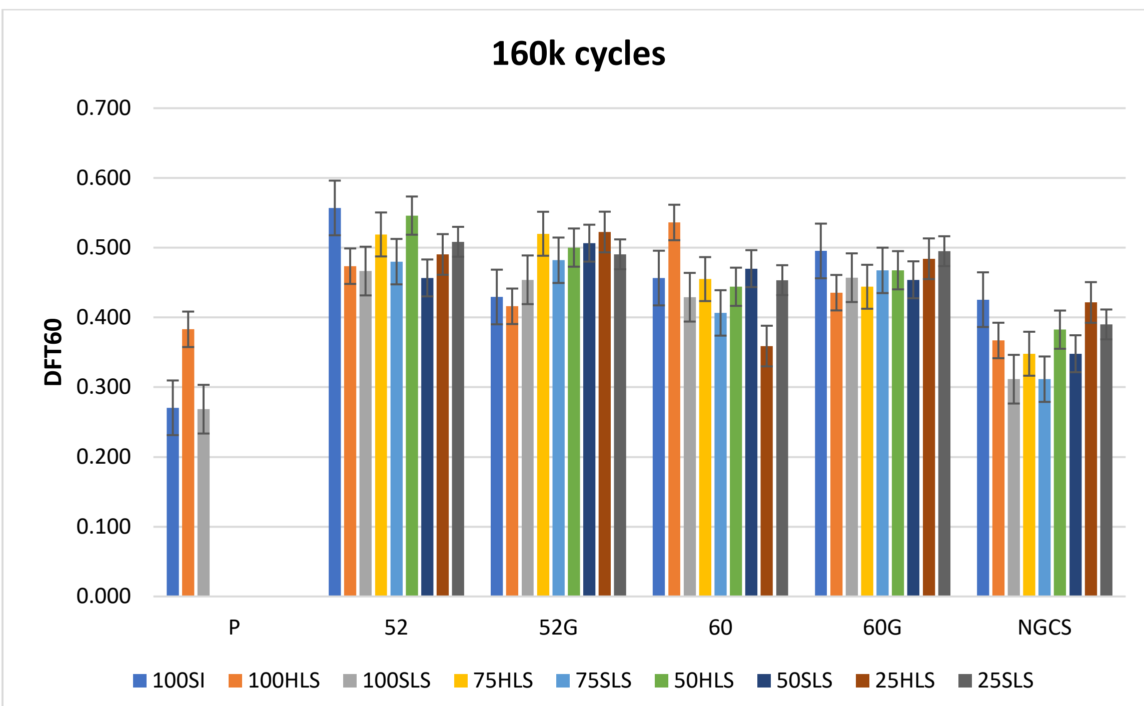


Figure 4-19. Final friction values after 160k cycles in TWPD for all textures tested. (P) represents the as-cast broom texture only used for baseline specimens.

Loss of Friction

Figure 4-20 through Figure 4-22 represent the variation of percent loss in friction with various aggregate blends and textures. While Figure 4-20 gives a broader look of the % loss in friction for different aggregate blends, Figure 4-21 shows a closer examination of the performance of aggregate blends. It can be seen from Figure 4-20 that there is a gain in friction for 4 blends and textures (100Si-P, 100HLS-P, 100SLS-P and 25SLS-60). Hence, it can be said that the control slabs gain friction. This likely due to the exposure of fine aggregates with more polishing causing an increase in friction. **This result does not, and should not, be taken as a recommendation to provide no texture to the surface of concrete pavements as the process is highly variable and inconsistent.**

Figure 4-21 represents the variation of percent loss in friction for different textures with the percent of siliceous aggregate in the blend. Linear regression is used to fit the data to observe the trend. The typical trend is the decrease in loss of friction with the increase in siliceous aggregates in the blend with the exception of 3 aggregate-texture combination (HLS-52G, SLS-52G and HLS-60).

Figure 4-22 shows the variation of percent loss of friction for different textures. It can be seen from the chart that the loss of friction is high for all the aggregate blends for NGCS texture. Within other textures, there is a variation in the percent loss depending on the aggregate blend.

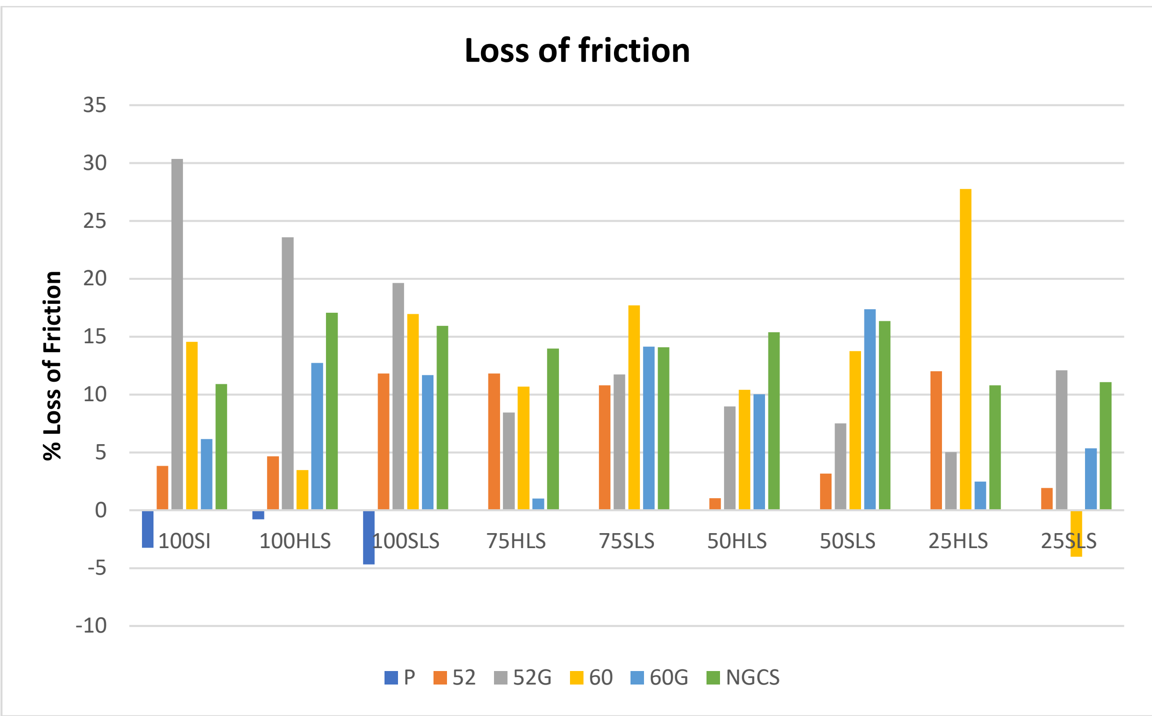


Figure 4-20. Percentage of friction loss for all aggregate blends tested from initial and final friction values. (P) represents the as-cast broom texture only used for baseline specimens.

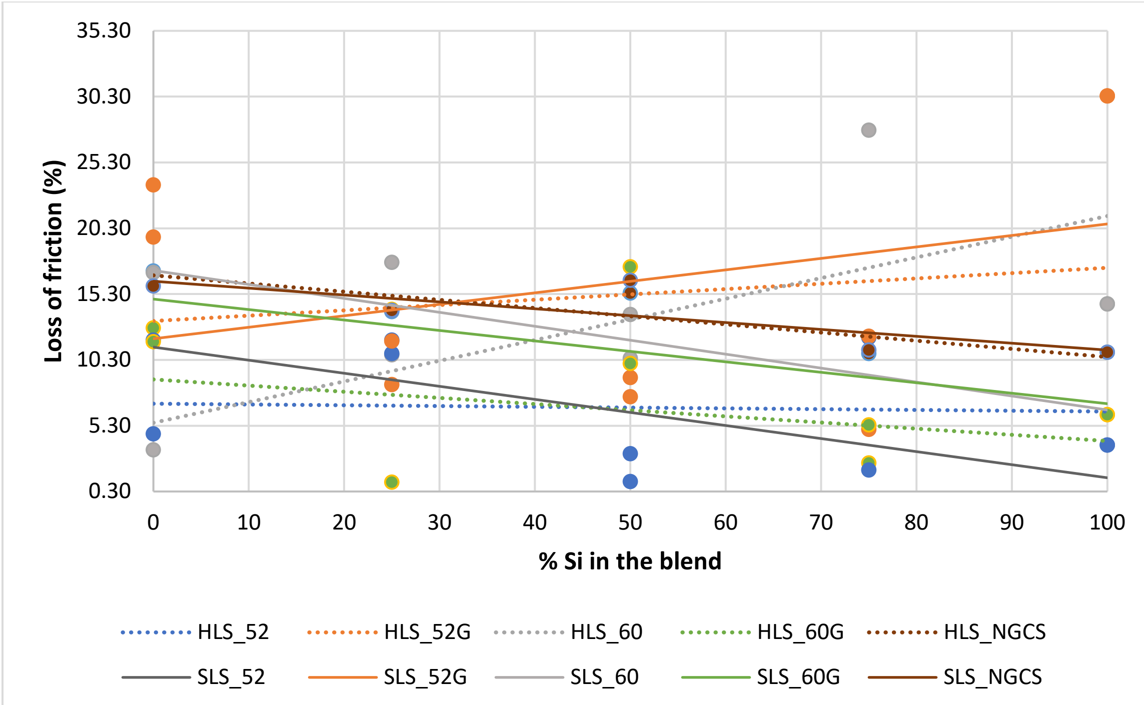


Figure 4-21. Variation of friction loss percentage as a function of granite, SI, content for all tested textures.

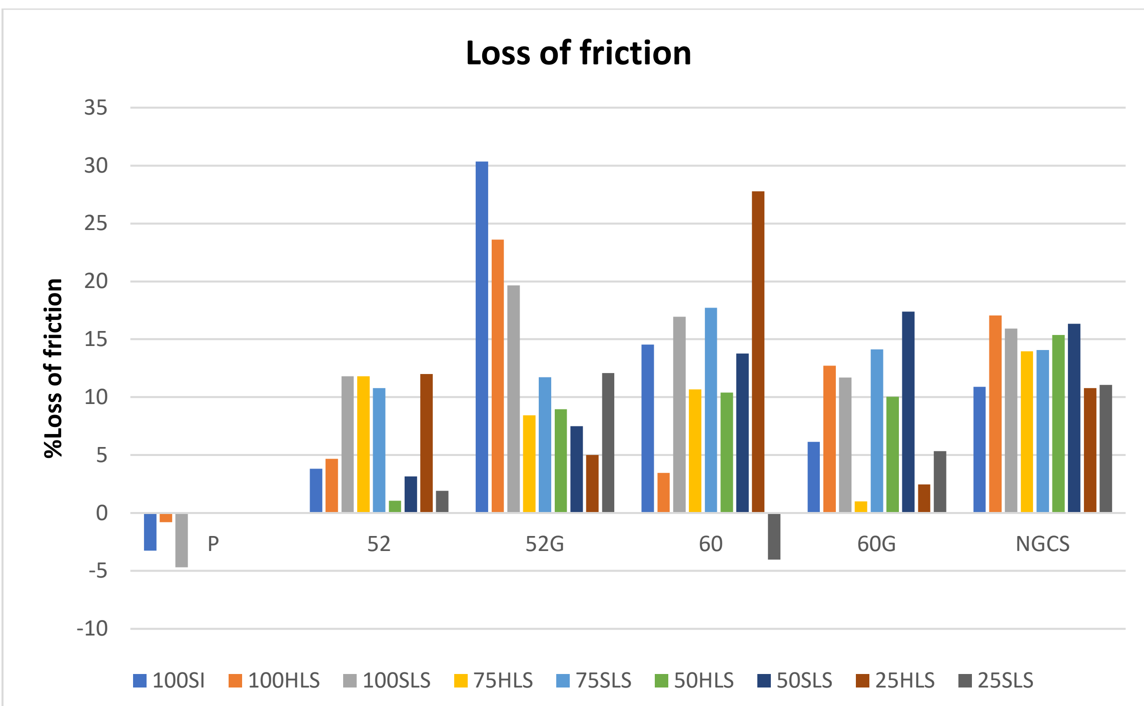


Figure 4-22. Percentage of friction loss for all tested textures from initial and final friction values. (P) represents the as-cast broom texture only used for baseline specimens.

Friction Retention

For the purpose of this study, the ratio of DFT60 at 160k cycles and 10k cycles was defined as the friction retention. Figure 4-23 shows the friction retention for different textures. It can be observed that baseline slabs had friction retention greater than 1. As mentioned earlier, this might be because of the exposure of the fine aggregate on the surface with polishing, therefore, causing an increase in friction. **This result does not, and should not, be taken as a recommendation to provide no texture to the surface of concrete pavements as the process is highly variable and inconsistent.** It can also be noted that friction retention of 52 blades/ft is highest amongst all the textures.

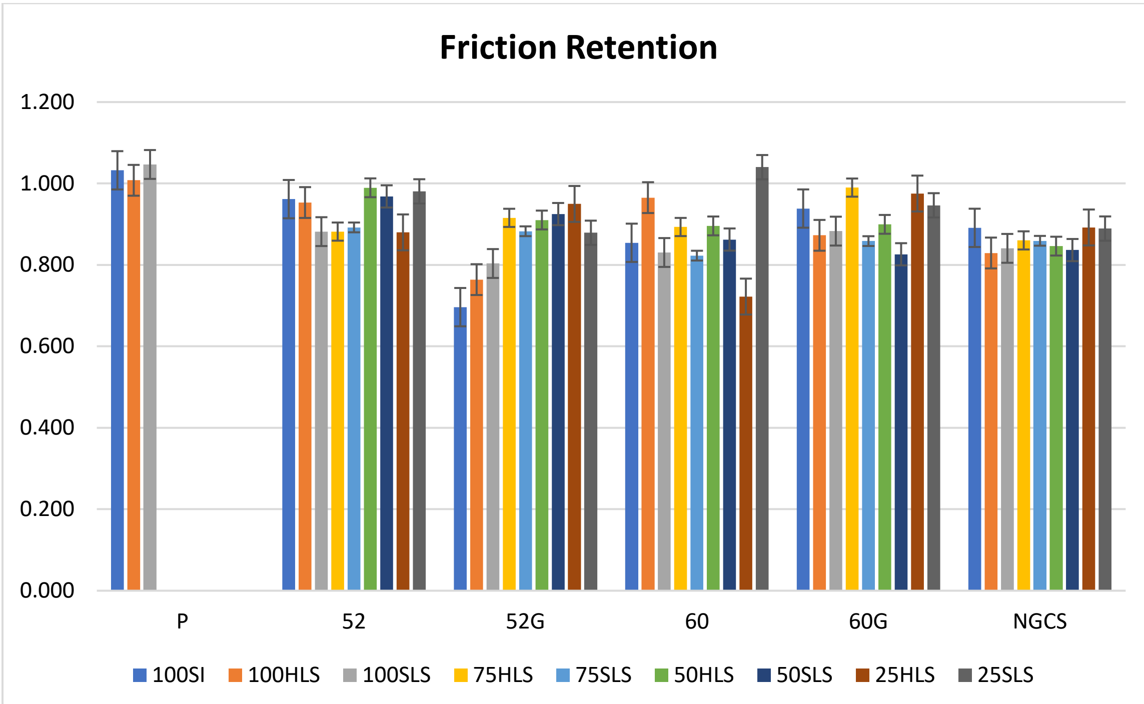


Figure 4-23. Friction retention values for all textures tested. (P) represents the as-cast broom texture only used for baseline specimens.

Predictive Capabilities

Figure 4-24 and Figure 4-25 show the variation of DFT 60 with the aggregate properties. It can be seen that DFT60 values vary for the same value of aggregate properties (%acid insoluble in acid insolubility test or %retained in sodium sulfate soundness test or %loss in LA abrasion test or %loss in Micro-Deval test) depending upon different textures provided on the surface. Therefore, it can be said that these aggregate tests cannot solely predict the performance of pavement. The performance of pavement for friction is governed by the texture provided on the pavement surface and cannot be directly predicted by any of the aforementioned test methods.

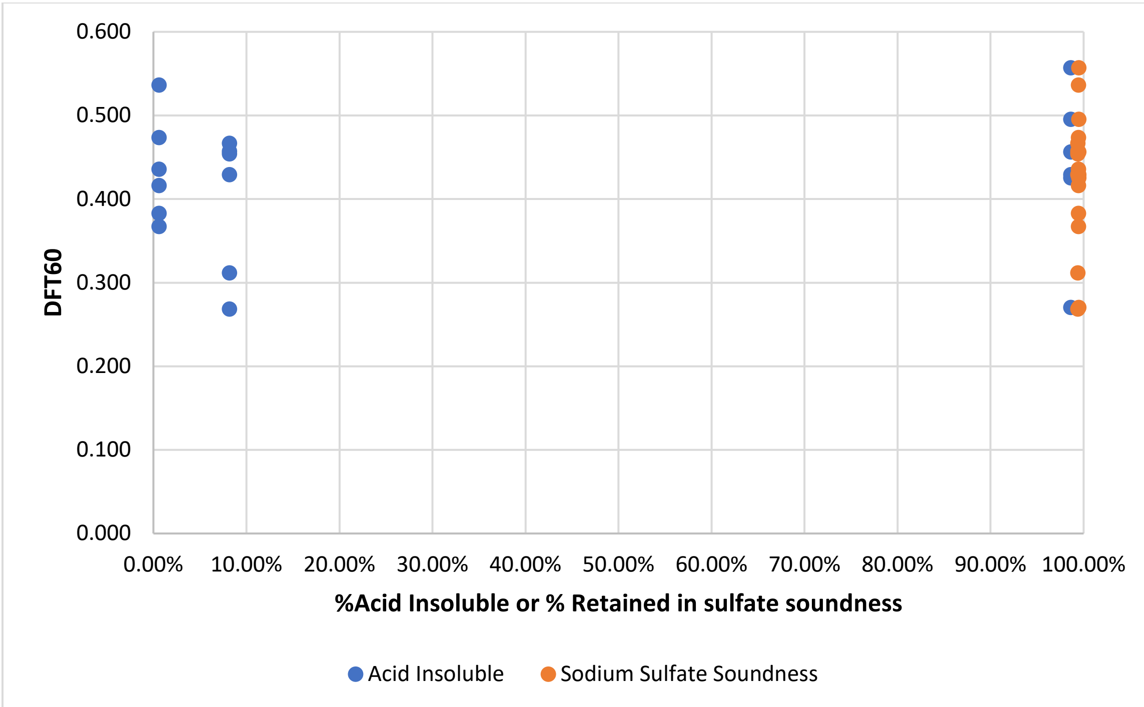


Figure 4-24. Variation of DFT60 with acid residue and sulfate soundness testing for all aggregates tested.

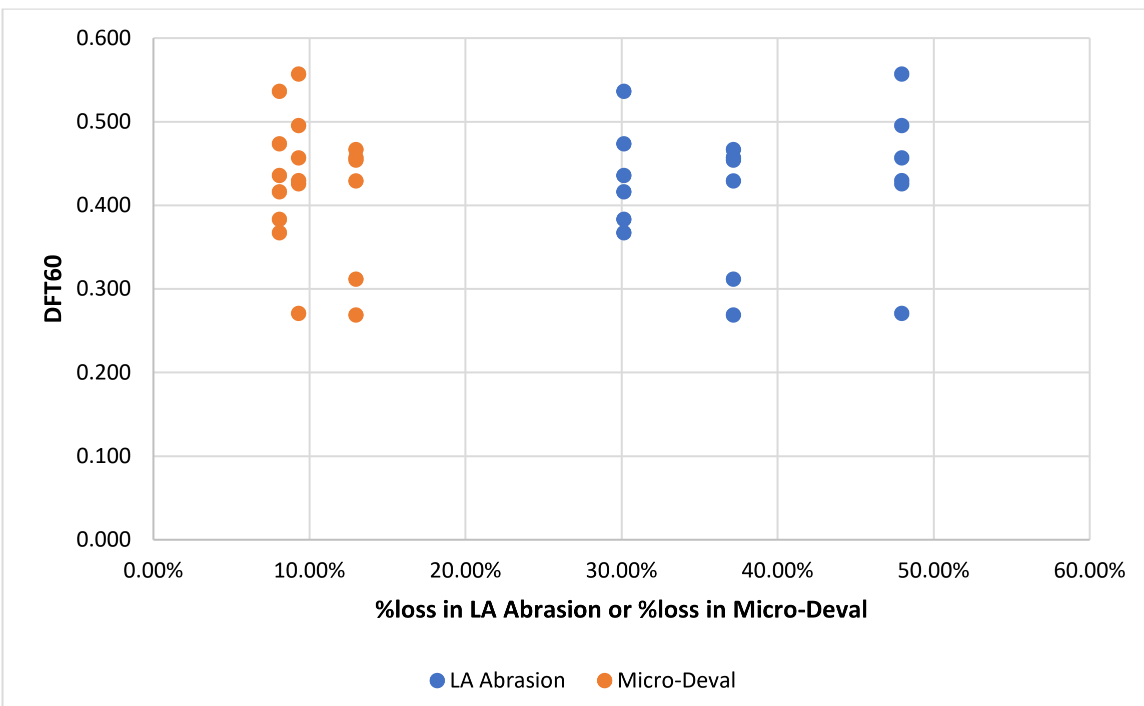


Figure 4-25. Variation of DFT60 with LA abrasion and Micro-deval testing for all aggregates tested.

Circular Track Meter Testing

A total of 75 slabs were tested using Circular Track Meter (CTM) after 160k polishing cycles to obtain Mean Profile Depth values in mm. These values are tabulated in Appendix C. MPD values obtained from the replicate slabs were averaged for the data analysis. Figure 4-26 through Figure 4-28 represent the variations of MPD values with different textures and aggregate blends.

Figure 4-26 shows a chart with MPD values for different aggregate blends and Figure 4-27 shows the variation of MPD values with increase in siliceous aggregate in the blend. It can be seen from Figure 4-27 that there is no clear trend for MPD values with the % siliceous content in the aggregate blend and therefore, can be said that the all the aggregate blends perform comparable to each other. However, Figure 4-28 shows that NGCS texture has highest MPD values followed by textures with grooving. It can be noted that although the NGCS textures have high MPD values they perform poorly on friction values with DFT.

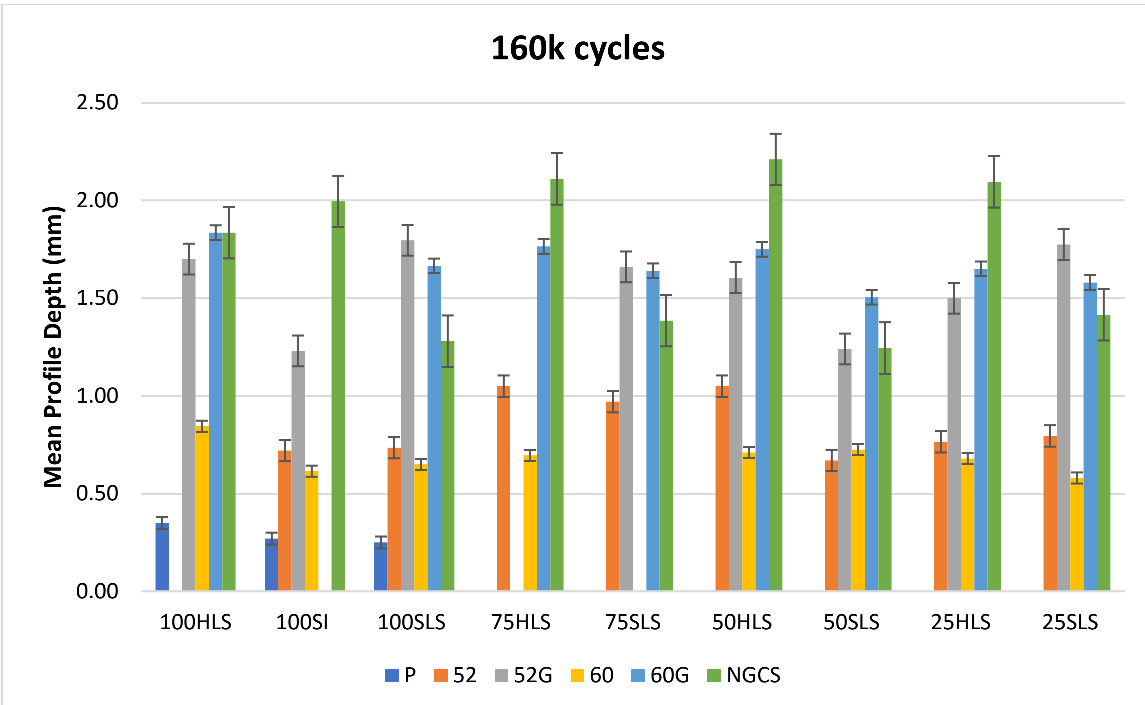


Figure 4-26. Final MPD values after 160k cycles for all aggregates tested. (P) represents the as-cast broom texture only used for baseline specimens.

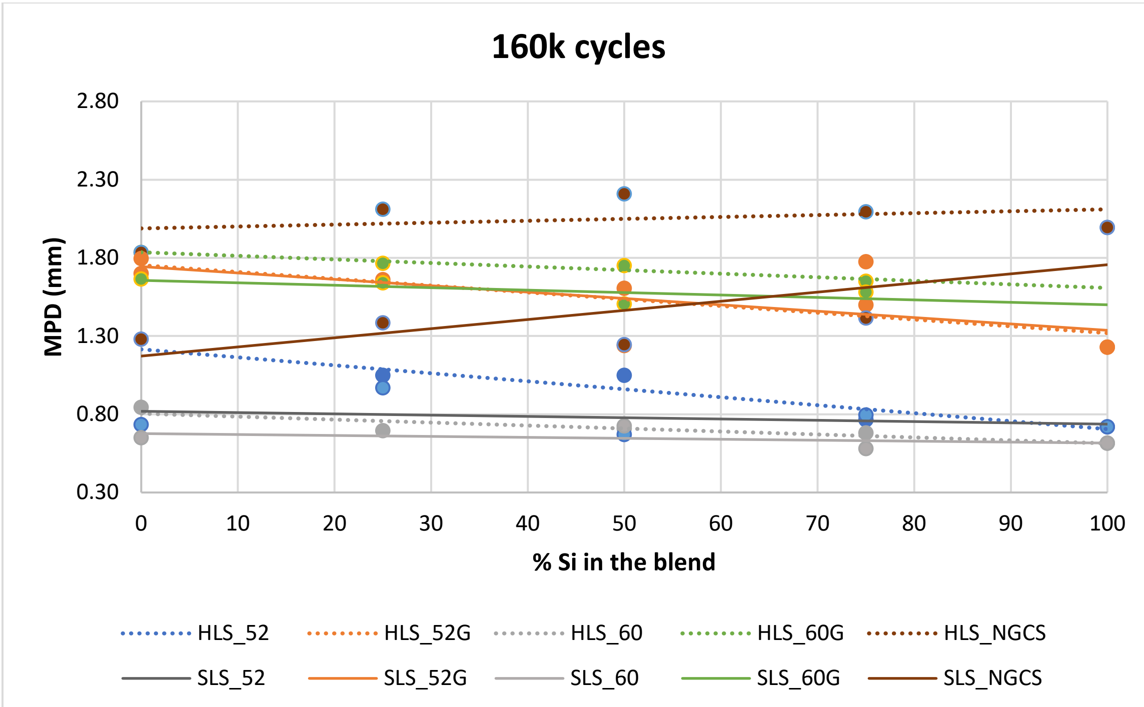


Figure 4-27. Variation of MPD values as a function of granite, SI, for all tested textures.

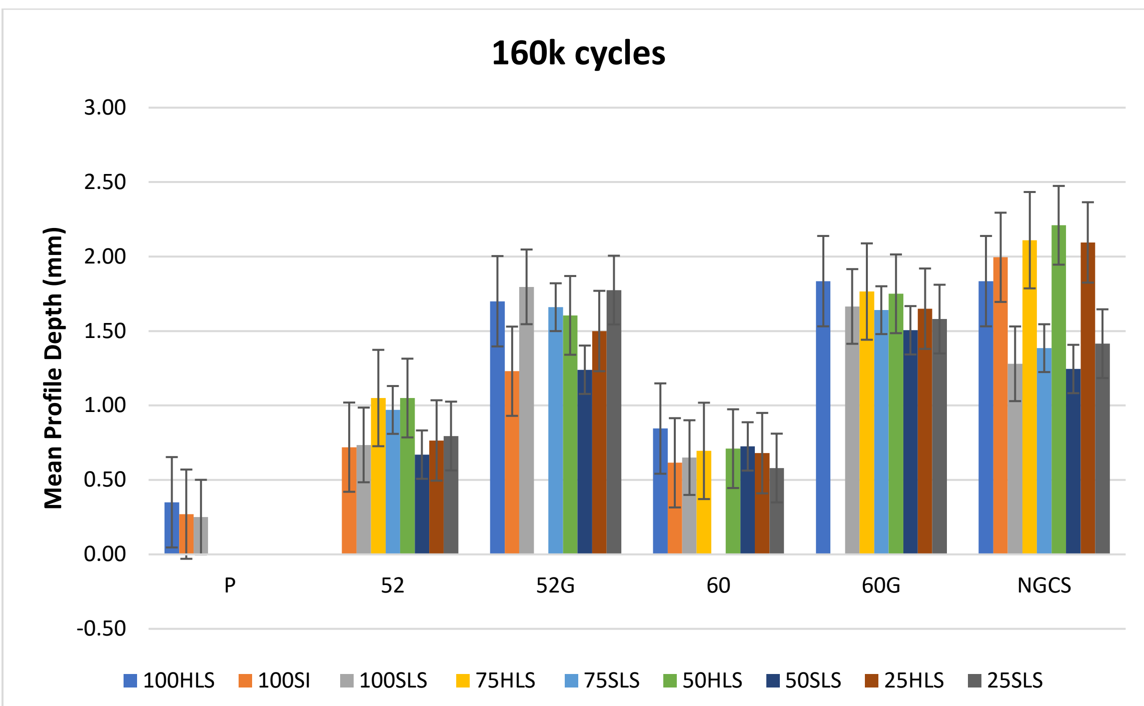


Figure 4-28. Final MPD values after 160k cycles for all textures tested.

Circular Track Meter and DFT

Figure 4-29 shows the correlation of DFT60 values obtained from DFT with MPD values obtained from CTM. Linear regression was used to see the trends and the fit of the data. It can be seen that DFT60 and MPD are best correlated from plain slabs followed by NGCS textures. Overall, the correlation is poor for all the grinding and grooving textures with very low R^2 values. It can also be noted that for the texture with grooving (52 blades/ft with grooving and 60 blades/ft with grooving), the DFT60 and MPD values are inversely related. It can therefore be said that a high MPD value does not imply a high friction value. It is highly likely that the DFT and CTM measurements, being circular in nature, are not fully characterizing the nature of the applied, longitudinal texture.

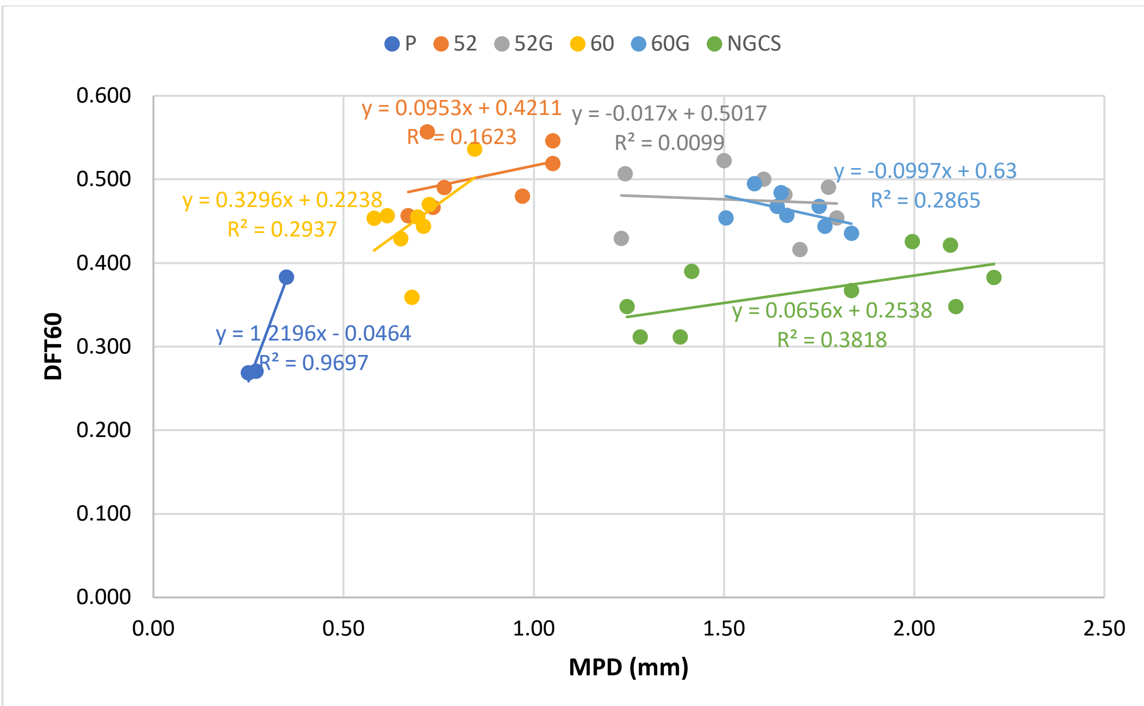


Figure 4-29. Correlation of DFT60 and MPD data for all textures tested.

Circular Track Meter and Aggregate Properties

Figure 4-30 and Figure 4-31 show the variation of MPD values with the aggregate properties. Similar to the DFT60 values, MPD values also vary for the same value of aggregate properties (%acid insoluble in acid insolubility test or %retained in sodium sulfate soundness test or %loss in LA abrasion test or %loss in Micro-Deval test) depending upon different textures provided on the surface. Therefore, it can be said that these aggregate tests cannot solely predict the performance of pavement and performance of pavement for friction is governed by the texture provided on the pavement surface.

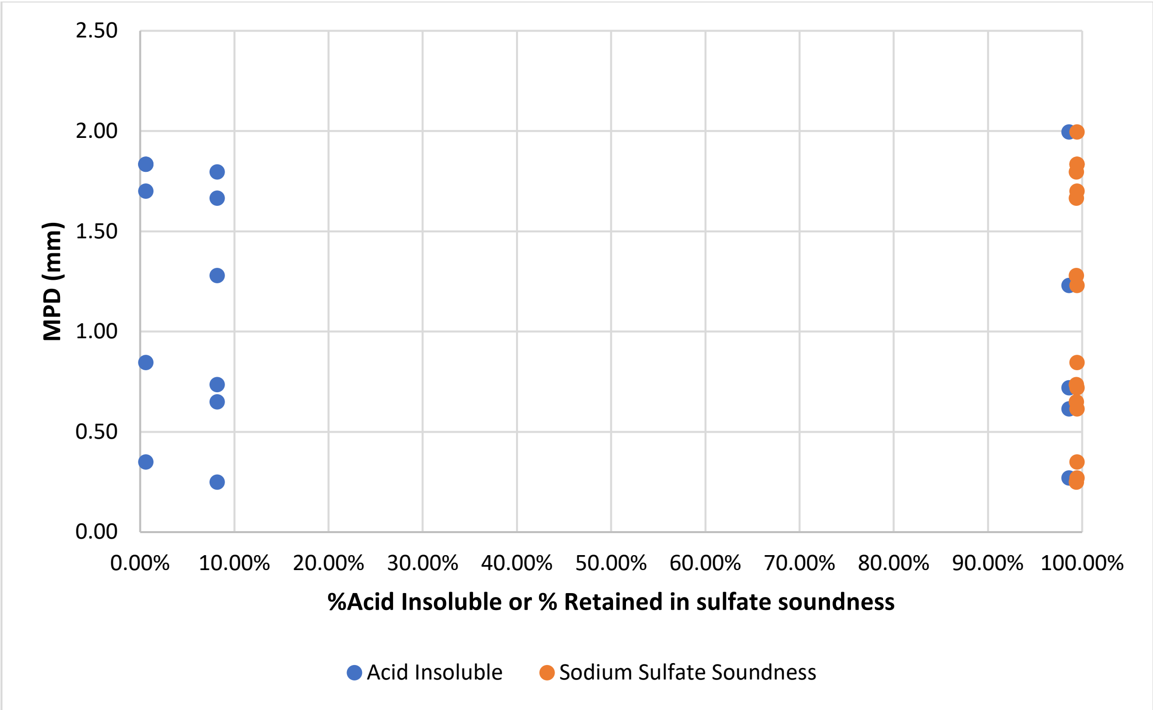


Figure 4-30. Variation of MPD with acid residue and sulfate soundness testing for all aggregates tested.

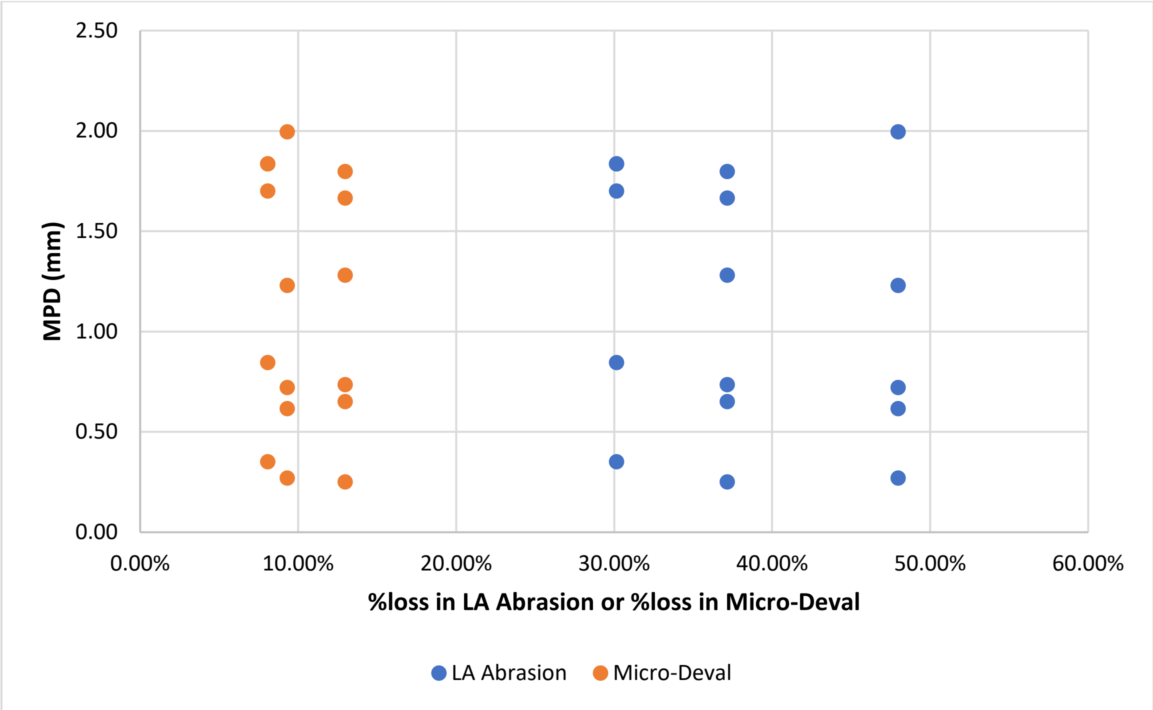


Figure 4-31. Variation of MPD with LA abrasion and Micro-deval testing for all aggregates tested.

5.0 Conclusions and Recommendations

The conclusions will be limited to the scope of the experimental study while the recommendations will propose possible solutions and/or changes to existing procedures.

Study Conclusions

The major conclusions from the experimental study can be summarized as follows:

1. The British Pendulum Tester is not a valid assessment tool for **laboratory scale** testing. As tested, the reduced foot size and contact area is a very poor and unreliable measure of surface characteristics of laboratory cast concrete pavement slab specimens.
2. The DFT is a sufficient and reliable tool to evaluate laboratory pavement specimens for surface friction properties. The values from the DFT represent an average, and almost isotropic friction value due to the circular nature of the test relative to the applied texture of the laboratory concrete pavement slabs.
3. Across the board, the highest performing texture was that with no grooves and 52 blades/ft. See the recommendations for caveats to this finding.
4. Very generally, the initial, final, and retention values for friction increased with increasing granite content. However, some of the trends were extremely minor and in a few cases granite caused higher friction loss.
5. Friction values of the baseline slabs with an as-cast broom texture were observed to have an increase in friction values upon polishing. This was likely due to the exposure of the fine aggregate during the polishing process and is not necessarily indicative of a good quality pavement surface.
6. CTM testing does not indicate any benefit in increasing granite content. Additionally, there is no good correlation between DFT and CTM.
7. Aggregate durability tests (i.e. acid residue, sulfate soundness, LA abrasion, and Micro-deval) are not good indicators of polishing behavior and no correlation between DFT/CTM and aggregate property values was observed.

Study Recommendations

- Do not use BPT for laboratory polished pavement specimens unless size of specimens is sufficiently large that no modification to the BPT procedure is needed.
- The dolomitic content of limestone sources should be listed as part of List I-1 that is maintained by ALDOT. This would allow ALDOT to monitor the performance over time based on mineralogy.
- Acid residue and sulfate soundness do not provide predictive capabilities regarding aggregate polishing but should still be employed to assure aggregate durability.

- The currently used BPN9 test (ALDOT-382) should be used, in conjunction with LA abrasion and Micro-deval, to characterize aggregate sources with respect to polishing.
- While the ground-only texture with a 52 blade/ft spacing was found to be the best overall texture, it is known from experience that grooving a pavement can offer advantages in water removal that may not be sufficiently captured in a DFT. It is recommended that ALDOT use the ACPA/IGGA recommendations for aggregate hardness for the initial blade spacing and then groove regardless of aggregate type. The data shows that both 52 blades/ft and 60 blades/ft with grooving can provide some of the highest friction retention based on the aggregate hardness. Since grooving, in a historical context, is relatively new, it is likely the lack of grooving in the late 1980s led to the polishing and poor performance on the critical ALDOT maintained roadways.
- While NGCS has been proven on numerous occasions to reduce road noise and in some cases increase friction, it is **not recommended** that ALDOT begin implementing NGCS without further study. It is recommended that ALDOT support the construction of a small test section in an actual concrete pavement so that the performance can be evaluated in the field.
- Along with recommendations from NCAT, it is not recommended to attempt a correlation between the DFT and CTM values.

Recommended Specification Changes

The research team has recommended the following specification changes. Limestone coarse aggregate used in wearing surfaces for mainline concrete paving applications shall meet the following requirements:

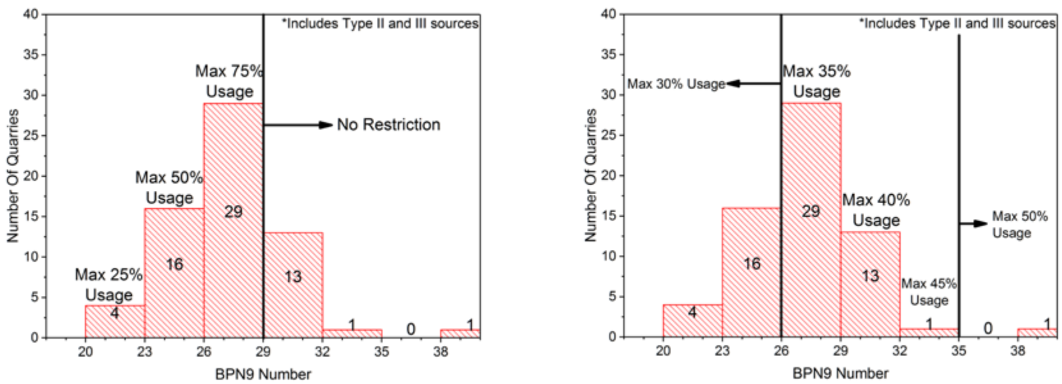
- Be an approved aggregate source on List I-1
- Surface grinding and grooving shall follow IGGA recommendations
- Maximum LA abrasion mass loss of 40%
- Maximum Micro-deval mass loss of 18%
- BPN9 values shall limit usage of limestone as follows:
    - <20 – source shall not be used in any amount
    - 20-22 – maximum of 25% of total coarse aggregate mass
    - 23-25 – maximum of 50% of total coarse aggregate mass
    - 26-28 – maximum of 75% of total coarse aggregate mass
    - >28 – no restriction

ALDOT has recommended the following specification changes, based on the current HMA carbonate restrictions. It should be noted that asphalt and PCC pavement polish via significantly different mechanisms:

- BPN9 values shall limit usage of limestone as follows:

- ≤ 20 – shall not be used in any amount
- 21-25 – maximum of 30% of total coarse aggregate mass
- 26-28 – maximum of 35% of total coarse aggregate mass
- 29-31 – maximum of 40% of total coarse aggregate mass
- 32-34 – maximum of 45% of total coarse aggregate mass
- ≥ 35 – maximum of 50% of total coarse aggregate mass

The histogram below shows the number of limestone sources, approved on the May 6, 2019 I-1 list, that would qualify into each category for both researcher and ALDOT recommended ranges (Figure 5-1).



(a) (b)

Figure 5-1: Final specification recommendation comparison. The authors (a) recommendation and ALDOT recommendation (b) are both presented. The data from the quarries comes from the I-1 materials list approved on May 6, 2019.

6.0 References

Alabama Department of Transportation Standard Specifications for Highway Construction. (2018). Alabama Department of Transportation.

Asi, I. M. (2007). "Evaluating skid resistance of different asphalt concrete mixes." *Building and Environment*, 42(1), 325–329.

Balmer, G. (n.d.). "Pavement Texture: Its Significance and Development." *Federal Highway Administration, U.S. Department of Transportation*.

Bazlamit, S. M., and Reza, F. (2005). "Changes in Asphalt Pavement Friction Components and Adjustment of Skid Number for Temperature." *Journal of Transportation Engineering*, 131(6), 470–476.

Buddhavarapu, P., Smit, A. D. F., Prozzi, J. A., Asce, A. M., and Trevino, M. (2017). "Overall Changes in Surface Characteristics after Diamond Grinding Continuously Reinforced Concrete Pavement." 2012(1), 1–9.

Caltrans Division of Maintenance. (2007). "Chapter 5 Diamond Grinding and Grooving." *MTAG Volume II - Rigid Pavement Preservation 2nd Edition*, II, 1–23.

Chen, D., and Hong, F. (2015). "Long-Term Performance of Diamond Grinding." *ASCE*, 29(1), 1–7.

Clarkson, J. R., Price, T. J., and Adams, C. J. (1992). "Role of metastable phases in the spontaneous precipitation of calcium carbonate." *Journal of the Chemical Society, Faraday Transactions*, 88(2), 243.

Degens, E. T. (1965). *Geochemistry of Sediments: A Brief Survey*. Prentice-Hall, Inc.

Descornet, G. (2000). "Low-Noise Road Surface Techniques and Materials." (August), 1–5.

Ergun, M., Iyinam, S., and Iyinam, A. F. (2005). "Prediction of Road Surface Friction Coefficient Using Only Macro- and Microtexture Measurements." *Journal of Transportation Engineering*, 131(April), 311–319.

"FDOT Standard Specifications For Road And Bridge Construction." (2017). (January), 1–1215.

Folk, R. (1959). "Practical Petrographic Classification of Limestones." *AAPG Bulletin*, 43(1), 1–38.

Fowler, D. W., Allen, J. J., Lange, A., and Range, P. (2006). "The prediction of coarse aggregate performance by Micro-Deval and other aggregate tests." … *Center for Aggregate* …, 616.

Fwa, T. F., Asce, M., and Ong, G. P. (2006). "Transverse Pavement Grooving against

Hydroplaning . II : Design." 132(June), 449–457.

Hall, J. W., Smith, K. L., and Littleton, P. (2009). *Texturing of Concrete Pavements*.

Heitzman, M., Turner, P., and Greer, M. (2015). *High Friction Surface Treatment Alternative Aggregates Study*.

Hiti, M., and Ducman, V. (2014). "Analysis of the slider force calibration procedure for the British Pendulum Skid Resistance Tester." *Measurement Science and Technology*.

Holt, F. B., and Musgrove, G. R. (1982). "Surface Texture Classification : A Guide to Pavement Skid Resistance." *American Society for Testing and Materials*, 31–44.

Horne, B. W. B. (1969). "Results from Studies of Highway Grooving and Texturing at NASA Wallops Station."

Horne, W. B., and Buhlmann, F. (1983). "A Method for Rating the Skid Resistance and Micro/Macrotexture Characteristics of Wet Pavements." *ASTM STP 793*, 191–218.

Kandhal, K., and Bishara, E. A. (1992). "Evaluation of Limestone Aggregates in Asphalt Wearing Courses Phase.II." *National Center for Asphalt Technology*.

Kane, M., Artamendi, I., and Scarpas, T. (2013). "Long-term skid resistance of asphalt surfacings: Correlation between Wehner-Schulze friction values and the mineralogical composition of the aggregates." *Wear*, 303(1–2), 235–243.

Klenke, N., Stone, M., Ph, D., Lindly, J. K., Ph, D., Giannini, E. R., and Ph, D. (2015). "Investigation into the Use of Carbonate Coarse Aggregates in Alabama Concrete Pavements : Final Report." *PCA*.

Kummer, H. W., and Meyer, W. E. (1966). "Skid or Slip Resistance?" *Journal of Materials*, 1(3), 667–688.

Laplante, P., Aitcin, P.-C., and Vezina, D. (1991). "ABRASION RESISTANCE OF CONCRETE." *ASCE*, 19–28.

Li, S., Harris, D., and Wells, T. (2016). "Surface texture and friction characteristics of diamond-ground concrete and asphalt pavements." *Journal of Traffic and Transportation Engineering (English Edition)*, Elsevier Ltd, 3(5), 475–482.

Lund, K., Fogler, H. S., and McCune, C. C. (1973). "Acidization—I. The dissolution of dolomite in hydrochloric acid." *Chemical Engineering Science*, 28(3), 691-IN1.

Mackey, G. (n.d.). "Road Surface Friction: Measurement, Testing and Accuracy."

Mahmoud, E., and Masad, E. (2007). "Experimental Methods for the Evaluation of Aggregate Resistance to Polishing, Abrasion, and Breakage." *ASCE*.

"Mississippi Standard Specifications for Road and Bridge Construction." (2017). 804.

Neithalath, N., Garcia, R., Weiss, J., and Olek, J. (2005). "Tire-pavement interaction noise: Recent research on concrete pavement surface type and texture." *Proceedings - 8th International Conference on Concrete Pavements: Innovations for Concrete Pavement: Technology Transfer for the Next Generation*, 2(January), 523–540.

Ong, G. P., Fwa, T. F., and Asce, M. (n.d.). "Transverse Pavement Grooving against Hydroplaning. I: Simulation Model."

Panagouli, O. K., and Kokkalis, A. G. (1998a). "Skid resistance and fractal structure of pavement surface." *Chaos, Solitons & Fractals*, 9(3), 493–505.

Panagouli, O. K., and Kokkalis, A. G. (1998b). "Skid resistance and fractal structure of pavement surface." *Chaos, Solitons & Fractals*, 9(3), 493–505.

Pin, Y., Lee, K., Sang, Y., and Associate, C. (n.d.). "EFFECT OF PAVEMENT SURFACE TEXTURE ON BRITISH PENDULUM TEST."

Pirsson, L. V. (1908). *Rocks and Rock Minerals*. (E. Pirsson and A. Knopf, eds.), John Wiley & Sons, Inc.

Rado, Z. (2009). "Evaluating Performance of Limestone Prone to Polishing." (510401).

Rado, Z., and Kane, M. (2014). "An initial attempt to develop an empirical relation between texture and pavement friction using the HHT approach." *Wear*, Elsevier, 309(1–2), 233–246.

Rao, S., Yu, H., Khazanovich, L., Darter, M., and Mack, J. (1999a). "Longevity of Diamond-Ground Concrete Pavements." *Transportation Research Record*, 1684(1), 128–136.

Rao, S., Yu, H. T., and Darter, M. I. (1999b). *The Longevity and Performance of Pavements*. Portland Cement Association.

Rezaei, A., Masad, E., and Chowdhury, A. (2011). "Development of a Model for Asphalt Pavement Skid Resistance Based on Aggregate Characteristics and Gradation." *Journal of Transportation Engineering*.

Rizenbergs, R. L. (1968). *Discussion on Skid Resistance of Pavement Surfaces*.

Roger, R., and Richard, M. (1948). "Petrographic and Mineralogic Characteristics of Aggregates." 20–48.

Scofield, L. (2017). "Development and Implementation of the Next Generation Concrete Surface." *ACPA*, 39.

Senga, Y., Dony, A., Colin, J., Hamlat, S., and Berthaud, Y. (2013). "Study of the skid resistance of blends of coarse aggregates with different polish resistances.” *Construction and Building Materials*, 48, 901–907.

Shalabi, F. I., Cording, E. J., and Al-hattamleh, O. H. (2007). “Estimation of rock engineering properties using hardness tests.” *Engineering Geology*, 90, 138–147.

Singurindy, O., and Berkowitz, B. (2003). “Flow, dissolution, and precipitation in dolomite.” *Water Resources Research*, 39(6).

“Standard Specifcations Construction of Transportation Systems.” (2013). .

TDOT. (2015). “Tennessee Department of Transportation Standard Specifications for Road and Bridge Construction.” 1–37.

TxDOT. (2004). “Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.” *TxDOT*, (December).

Vanderdeelen, J. (2012). “IUPAC-NIST Solubility Data Series. 95. Alkaline Earth Carbonates in Aqueous Systems. Part 2. Ca.” *Journal of Physical and Chemical Reference Data*, (A. De Visscher, ed.), 41(2), 023105-023105-137.

Whitney, D., Fowler, D. W., and Rached, M. (2013). “Use of manufactured sands for concrete pavement.”

Yoshitake, I., Ueno, S., Ushio, Y., Arano, H., and Fukumoto, S. (2016). “Abrasion and skid resistance of recyclable fly ash concrete pavement made with limestone aggregate.” *Construction and Building Materials*.

7.0 Appendix A: BPT Results

S. No.	Slab designation	10k cycles	40k cycles	100k cycles	160k cycles
1	*100Si P-1*	26.50	23.17	23.50	24.00
2	*100Si P-2*	22.83	24.33	22.67	25.17
3	*100Si 52-1*	29.33	28.67	35.33	35.00
4	*100Si 52-2*	29.67	31.00	32.50	38.17
5	*100Si 52G-1*	40.20	34.63	23.67	34.83
6	*100Si 52G-2*	36.00	39.33	40.33	38.67
7	*100Si 60-1*	40.50	33.88	30.12	34.00
8	*100Si 60-2*	31.10	31.50	33.67	37.83
9	*100Si 60G-1*	34.67	37.33	35.83	34.67
10	*100Si 60G-2*	27.33	25.33	28.33	31.00
11	*100Si NGCS-1*	29.67	36.50	34.17	30.50
12	*100Si NGCS-2*	29.67	32.17	32.17	28.00
13	*100HLS P-1*	32.58	28.59	30.25	26.60
14	*100HLS P-2*	32.17	29.67	20.67	28.83
15	*100HLS 52-1*	32.17	31.83	31.50	36.33
16	*100HLS 52-2*	25.33	29.17	30.33	30.00
17	*100HLS 52G-1*	30.17	32.67	37.50	30.50
18	*100HLS 52G-2*	35.33	34.00	36.50	36.00
19	*100HLS 60-1*	35.17	31.17	28.33	30.00
20	*100HLS 60-2*	33.50	30.83	29.33	38.00
21	*100HLS 60G-1*	32.67	37.50	37.17	32.33
22	*100HLS 60G-2*	35.33	35.33	34.67	33.33
23	*100HLS NGCS-1*	33.50	31.00	34.67	30.67
24	*100HLS NGCS-2*	32.83	32.50	30.67	31.67
25	*100SLS P-1*	25.17	22.67	22.67	21.00
26	*100SLS P-2*	23.33	23.17	20.67	23.00
27	*100SLS 52-1*	36.50	37.17	34.33	32.17
28	*100SLS 52-2*	35.17	34.67	37.17	35.00
29	*100SLS 52G-1*	34.50	36.17	31.83	30.33
30	*100SLS 52G-2*	35.67	35.33	31.33	33.50
31	*100SLS 52G-3*	35.50	30.00	32.17	32.50
32	*100SLS 60-1*	37.50	29.33	35.00	32.17
33	*100SLS 60-2*	37.50	37.67	33.67	30.33
34	*100SLS 60G-1*	31.50	35.33	32.83	35.33
35	*100SLS 60G-2*	34.50	33.67	35.83	33.17
36	*100SLS NGCS-1*	31.00	30.17	30.00	27.83
37	*100SLS NGCS-2*	31.17	29.33	31.50	27.17
38	*75HLS 52-1*	32.00	32.33	35.17	34.67
39	*75HLS 52-2*	35.00	32.00	35.67	31.67

40	*75HLS 52G-1*	34.83	35.33	37.17	36.33
41	*75HLS 52G-2*	31.83	30.33	30.33	33.83
42	*75HLS 60-1*	37.50	32.50	32.00	37.83
43	*75HLS 60-2*	33.17	29.33	28.67	29.67
44	*75HLS 60G-1*	33.83	34.83	35.67	37.83
45	*75HLS 60G-2*	32.50	34.33	37.50	30.33
46	*75HLS NGCS-1*	29.00	28.33	28.33	29.67
47	*75HLS NGCS-2*	30.50	30.50	33.50	31.17
48	*75SLS 52-2*	36.50	31.50	36.00	33.50
49	*75SLS 52G-1*	37.67	36.33	33.67	34.33
50	*75SLS 52G-2*	34.67	34.67	37.17	35.67
51	*75SLS 60-1*	29.67	35.00	39.67	30.33
52	*75SLS 60-2*	33.50	37.17	32.50	29.67
53	*75SLS 60G-1*	35.83	35.33	33.00	35.00
54	*75SLS 60G-2*	32.33	35.50	32.17	32.33
55	*75SLS NGCS-1*	30.00	31.83	24.67	26.17
56	*75SLS NGCS-2*	31.33	32.50	32.50	32.67
57	*50HLS 52-1*	37.83	36.33	32.67	31.67
58	*50HLS 52-2*	31.83	31.17	32.50	--
59	*50HLS 52G-1*	38.33	41.50	32.83	34.50
60	*50HLS 52G-2*	28.67	32.50	30.33	29.67
61	*50HLS 60-1*	34.00	33.67	34.50	31.00
62	*50HLS 60-2*	35.50	36.33	31.33	27.50
63	*50HLS 60G-1*	33.17	31.33	32.83	33.67
64	*50HLS 60G-2*	30.17	32.17	33.00	31.50
65	*50HLS NGCS-1*	29.67	29.67	31.00	29.33
66	*50HLS NGCS-2*	34.67	32.83	31.00	32.17
67	*50SLS 52-1*	37.00	33.83	34.83	32.17
68	*50SLS 52-2*	33.50	28.83	32.00	36.17
69	*50SLS 52G-1*	30.83	34.17	28.50	36.00
70	*50SLS 52G-2*	35.83	33.33	40.67	37.33
71	*50SLS 60-1*	32.83	32.83	29.67	34.33
72	*50SLS 60-2*	33.17	32.83	36.00	36.83
73	*50SLS 60G-1*	35.67	32.50	33.17	31.33
74	*50SLS 60G-2*	35.00	33.33	34.67	29.67
75	*50SLS NGCS-1*	34.00	33.50	31.00	31.17
76	*50SLS NGCS-2*	34.50	32.17	33.50	24.83
77	*25HLS 52-1*	29.17	33.83	33.33	33.17
78	*25HLS 52-2*	36.33	25.33	31.83	28.00
79	*25HLS 52G-1*	30.33	31.83	32.83	34.50
80	*25HLS 52G-2*	42.50	39.17	38.17	37.50

81	*25HLS 60-1*	32.80	34.38	30.83	39.17
82	*25HLS 60-2*	32.83	28.83	30.67	31.83
83	*25HLS 60G-1*	33.17	37.50	32.17	35.50
84	*25HLS 60G-2*	39.00	33.33	32.17	35.00
85	*25HLS NGCS-1*	27.50	28.83	30.00	29.67
86	*25HLS NGCS-2*	26.83	32.50	32.50	29.33
87	*25SLS 52-1*	36.50	35.50	30.67	34.50
88	*25SLS 52-2*	39.67	37.50	31.17	36.00
89	*25SLS 52G-1*	34.00	38.17	36.67	33.17
90	*25SLS 52G-2*	32.17	35.33	33.33	34.33
91	*25SLS 60-1*	33.00	34.33	36.00	35.33
92	*25SLS 60-2*	30.83	30.67	34.17	34.17
93	*25SLS 60G-1*	36.00	33.17	34.00	34.83
94	*25SLS 60G-2*	39.83	30.50	35.33	33.33
95	*25SLS NGCS-1*	31.33	32.17	31.83	29.50
96	*25SLS NGCS-2*	30.33	31.00	31.67	32.50

8.0 Appendix B: DFT Results

S. No.	Slab designation	10k cycles	40k cycles	100k cycles	160k cycles
1	*100Si P-1*	0.288	0.280	0.263	0.257
2	*100Si P-2*	0.236	0.249	0.259	0.284
3	*100Si 52-1*	0.359	0.313	0.578	0.551
4	*100Si 52-2*	0.579	0.564	0.533	0.557
5	*100Si 52G-1*	0.664	0.493	0.416	0.350
6	*100Si 52G-2*	0.569	0.533	0.534	0.509
7	*100Si 60-1*	0.554	0.508	0.508	0.523
8	*100Si 60-2*	0.515	0.479	0.466	0.390
9	*100Si 60G-1*	0.520	0.484	0.494	0.493
10	*100Si 60G-2*	0.536	0.541	0.527	0.498
11	*100Si NGCS-1*	0.457	0.426	0.436	0.421
12	*100Si NGCS-2*	0.498	0.458	0.432	0.430
13	*100HLS P-1*	0.418	0.399	0.384	0.415
14	*100HLS P-2*	0.342	0.338	0.357	0.351
15	*100HLS 52-1*	0.544	0.507	0.491	0.495
16	*100HLS 52-2*	0.449	0.376	0.461	0.452
17	*100HLS 52G-1*	0.512	0.380	0.395	0.372
18	*100HLS 52G-2*	0.577	0.528	0.495	0.460
19	*100HLS 60-1*	0.571	0.552	0.551	0.535
20	*100HLS 60-2*	0.540	0.553	0.512	0.537
21	*100HLS 60G-1*	0.508	0.476	0.447	0.441
22	*100HLS 60G-2*	0.490	0.458	0.430	0.430
23	*100HLS NGCS-1*	0.443	0.411	0.389	0.374
24	*100HLS NGCS-2*	0.442	0.409	0.377	0.360
25	*100SLS P-1*	0.280	0.274	0.270	0.269
26	*100SLS P-2*	0.233	0.272	0.247	0.268
27	*100SLS 52-1*	0.547	0.508	0.473	0.463
28	*100SLS 52-2*	0.511	0.510	0.487	0.470
29	*100SLS 52G-1*	0.561	0.508	0.480	0.450
30	*100SLS 52G-2*	0.536	0.487	0.432	0.423
31	*100SLS 52G-3*	0.598	0.526	0.505	0.489
32	*100SLS 60-1*	0.531	0.495	0.461	0.446
33	*100SLS 60-2*	0.502	0.456	0.420	0.412
34	*100SLS 60G-1*	0.505	0.474	0.455	0.434
35	*100SLS 60G-2*	0.530	0.508	0.500	0.480
36	*100SLS NGCS-1*	0.392	0.353	0.334	0.320
37	*100SLS NGCS-2*	0.349	0.332	0.317	0.303
38	*75HLS 52-1*	0.559	0.536	0.528	0.519
39	*75HLS 52-2*	0.618	0.563	0.534	0.519

40	*75HLS 52G-1*	0.556	0.490	0.491	0.494
41	*75HLS 52G-2*	0.580	0.545	0.513	0.546
42	*75HLS 60-1*	0.509	0.523	0.481	0.455
43	*75HLS 60-2*	0.290	0.462	0.451	0.429
44	*75HLS 60G-1*	0.454	0.462	0.461	0.465
45	*75HLS 60G-2*	0.443	0.436	0.429	0.423
46	*75HLS NGCS-1*	0.355	0.362	0.333	0.328
47	*75HLS NGCS-2*	0.454	0.424	0.399	0.368
48	*75SLS 52-2*	0.538	0.511	0.483	0.480
49	*75SLS 52G-1*	0.542	0.480	0.460	0.477
50	*75SLS 52G-2*	0.550	0.506	0.483	0.487
51	*75SLS 60-1*	0.499	0.455	0.421	0.384
52	*75SLS 60-2*	0.489	0.433	0.414	0.429
53	*75SLS 60G-1*	0.508	0.489	0.461	0.448
54	*75SLS 60G-2*	0.581	0.534	0.507	0.487
55	*75SLS NGCS-1*	0.335	0.326	0.311	0.289
56	*75SLS NGCS-2*	0.390	0.354	0.348	0.334
57	*50HLS 52-1*	0.593	0.606	0.573	0.546
58	*50HLS 52-2*	0.511	0.475	0.508	
59	*50HLS 52G-1*	0.577	0.507	0.505	0.489
60	*50HLS 52G-2*	0.522	0.486	0.518	0.511
61	*50HLS 60-1*	0.496	0.442	0.450	0.444
62	*50HLS 60-2*	0.382	0.322	0.248	0.499
63	*50HLS 60G-1*	0.523	0.487	0.420	0.442
64	*50HLS 60G-2*	0.517	0.501	0.485	0.493
65	*50HLS NGCS-1*	0.452	0.413	0.398	0.373
66	*50HLS NGCS-2*	0.452	0.411	0.383	0.392
67	*50SLS 52-1*	0.394	0.452	0.448	0.433
68	*50SLS 52-2*	0.549	0.514	0.513	0.480
69	*50SLS 52G-1*	0.561	0.520	0.523	0.519
70	*50SLS 52G-2*	0.534	0.509	0.515	0.494
71	*50SLS 60-1*	0.553	0.525	0.497	0.472
72	*50SLS 60-2*	0.537	0.489	0.483	0.468
73	*50SLS 60G-1*	0.528	0.443	0.455	0.441
74	*50SLS 60G-2*	0.571	0.520	0.499	0.467
75	*50SLS NGCS-1*	0.426	0.376	0.371	0.364
76	*50SLS NGCS-2*	0.406	0.364	0.331	0.332
77	*25HLS 52-1*	0.557	0.551	0.508	0.490
78	*25HLS 52-2*	0.286	0.170	0.424	0.351
79	*25HLS 52G-1*	0.556	0.507	0.512	0.533
80	*25HLS 52G-2*	0.544	0.526	0.496	0.512

81	*25HLS 60-1*	0.497	0.494	0.430	0.359
82	*25HLS 60-2*	0.308	0.222	0.515	0.488
83	*25HLS 60G-1*	0.496	0.494	0.486	0.477
84	*25HLS 60G-2*	0.497	0.455	0.424	0.491
85	*25HLS NGCS-1*	0.465	0.488	0.459	0.435
86	*25HLS NGCS-2*	0.480	0.461	0.429	0.408
87	*25SLS 52-1*	0.510	0.511	0.502	0.504
88	*25SLS 52-2*	0.527	0.524	0.516	0.513
89	*25SLS 52G-1*	0.517	0.493	0.459	0.462
90	*25SLS 52G-2*	0.599	0.562	0.533	0.519
91	*25SLS 60-1*	0.351	0.394	0.414	0.431
92	*25SLS 60-2*	0.521	0.508	0.487	0.476
93	*25SLS 60G-1*	0.508	0.479	0.473	0.476
94	*25SLS 60G-2*	0.538	0.517	0.497	0.514
95	*25SLS NGCS-1*	0.442	0.418	0.400	0.382
96	*25SLS NGCS-2*	0.435	0.419	0.406	0.398

9.0 Appendix C: CTM Results

S. No.	Slab Designation	Mean Profile Depth (mm)
1	*100Si P-1*	0.23
2	*100Si P-2*	0.31
3	*100Si 52-1*	0.72
4	*100Si 52G-1*	1.23
5	*100Si 60-1*	0.59
6	*100Si 60-2*	0.64
7	*100Si NGCS-1*	2.13
8	*100Si NGCS-2*	1.86
9	*100HLS P-2*	0.35
10	*100HLS 52G-1*	1.62
11	*100HLS 52G-2*	1.78
12	*100HLS 60-1*	0.90
13	*100HLS 60-2*	0.79
14	*100HLS 60G-1*	2.01
15	*100HLS 60G-2*	1.66
16	*100HLS NGCS-1*	1.96
17	*100HLS NGCS-2*	1.71
18	*100SLS P-2*	0.25
19	*100SLS 52-1*	0.96
20	*100SLS 52-2*	0.51
21	*100SLS 52G-1*	1.88
22	*100SLS 52G-2*	1.78
23	*100SLS 52G-3*	1.73
24	*100SLS 60-1*	0.69
25	*100SLS 60-2*	0.61
26	*100SLS 60G-1*	1.65
27	*100SLS 60G-2*	1.68
28	*100SLS NGCS-1*	1.34
29	*100SLS NGCS-2*	1.22
30	*75HLS 52-1*	1.11
31	*75HLS 52-2*	0.99
32	*75HLS 60-1*	0.80
33	*75HLS 60-2*	0.59
34	*75HLS 60G-1*	1.80
35	*75HLS 60G-2*	1.73
36	*75HLS NGCS-1*	2.15
37	*75HLS NGCS-2*	2.07
38	*75SLS 52-2*	0.97
39	*75SLS 52G-1*	1.66

40	*75SLS 60G-1*	1.64
41	*75SLS NGCS-1*	1.27
42	*75SLS NGCS-2*	1.50
43	*50HLS 52-1*	1.05
44	*50HLS 52G-1*	1.48
45	*50HLS 52G-2*	1.73
46	*50HLS 60-2*	0.71
47	*50HLS 60G-1*	1.77
48	*50HLS 60G-2*	1.73
49	*50HLS NGCS-1*	2.31
50	*50HLS NGCS-2*	2.11
51	*50SLS 52-1*	0.67
52	*50SLS 52G-2*	1.24
53	*50SLS 60-1*	0.66
54	*50SLS 60-2*	0.79
55	*50SLS 60G-1*	1.63
56	*50SLS 60G-2*	1.38
57	*50SLS NGCS-1*	1.05
58	*50SLS NGCS-2*	1.44
59	*25HLS 52-1*	0.73
60	*25HLS 52-2*	0.80
61	*25HLS 52G-1*	1.50
62	*25HLS 60-1*	0.64
63	*25HLS 60-2*	0.72
64	*25HLS 60G-1*	1.83
65	*25HLS 60G-2*	1.47
66	*25HLS NGCS-1*	2.10
67	*25HLS NGCS-2*	2.09
68	*25SLS 52-1*	0.87
69	*25SLS 52-2*	0.72
70	*25SLS 52G-1*	1.56
71	*25SLS 52G-2*	1.99
72	*25SLS 60-1*	0.58
73	*25SLS 60G-1*	1.58
74	*25SLS NGCS-1*	1.61
75	*25SLS NGCS-2*	1.22

10.0 Appendix D: May 6, 2019 I-1 List

ALABAMA DEPARTMENT OF TRANSPORTATION

LIST I-1

SOURCES OF COARSE AND FINE AGGREGATES

APPROVED: March 8, 1988
REVISED: May 6, 2019

CRUSHED LIMESTONE
TYPE I

ID NUMBER	SOURCE NAME/LOCATION	LOOSE UNIT MASS kg/m^3 (lbs/ft^3)	BULK SPECIFIC GRAVITY	BULK SPECIFIC GRAVITY (SSD)	ABSORPTION %	FLAT/ELONG. 3:1 5:1	L.A. ABRASION %	BRITISH POLISHING NUMBER (BPN)	SODIUM SULFATE SOUNDNESS % SOUND	TOTAL SILICA CONTENT %
0048	MARTIN MARIETTA MATERIALS AUBURN, AL	1474(92)	2.797	2.810	0.5	10.1 0.3	20	26	99	2.4
0077	C. A. LANGFORD COMPANY, INC. GUNTERSVILLE, AL	1410(88)	2.653	2.682	1.1	14.7 0.3	17	27	98	3.9
0134	VULCAN MATERIALS COMPANY DOLCITO QUARRY TARRANT, AL	1378(86)	2.718	2.749	1.1	16.0 1.5	11	22	100	2.6
0137	VULCAN MATERIALS COMPANY OHATCHEE, AL	1490(93)	2.826	2.839	0.4	7.8 0.5	15	25	100	2.5
0140	ROGERS GROUP, INC. MOULTON, AL SEE NOTE 5	1490(93)	2.622	2.657	1.3	4.35 0.0	21	26	98	2.7
0141	VULCAN MATERIALS COMPANY FT PAYNE, AL	1394(87)	2.676	2.694	0.7	6.6 0.0	16	24	100	1.8
0142	ROGERS GROUP, INC. TUSCUMBIA, AL	1330(83)	2.552	2.595	1.7	8.3 1.1	32	27	94	2.9
0144-L	DUNN CONSTRUCTION LONGVIEW OPERATION SAGINAW, AL	1346(84)	2.705	2.718	0.5	17.7 2.8	21	27	100	2.7
0148	VULCAN MATERIALS COMPANY CHEROKEE, AL	1346(84)	2.573	2.608	1.4	12.3 1.1	48	29	99	1.9
0151	VULCAN MATERIALS COMPANY LACON, AL	1442(90)	2.668	2.693	0.9	8.5 1.0	23	27	99	1.4
0152	VULCAN MATERIALS COMPANY TRINITY, AL	1378(86)	2.646	2.670	0.9	13.0 0.3	23	26	99	2.0

CRUSHED LIMESTONE
TYPE I

ID NUMBER	SOURCE NAME/LOCATION	LOOSE UNIT MASS kg/m^3 (lbs/ft^3)	BULK SPECIFIC GRAVITY	BULK SPECIFIC GRAVITY (SSD)	ABSORPTION %	FLAT/ELONG. 3:1 5:1	L.A. ABRASION %	BRITISH POLISHING NUMBER (BPN)	SODIUM SULFATE SOUNDNESS % SOUND	TOTAL SILICA CONTENT %
0155	VULCAN MATERIALS COMPANY CALERA, AL	1490(93)	2.718	2.738	0.7	8.5 0.0	10	25	100	3.1
0158	VULCAN MATERIALS COMPANY GLENCOE, AL	1426(89)	2.710	2.730	0.4	6.1 0.6	20	23	100	3.7
0159	VULCAN MATERIALS COMPANY BEARDEN QUARRY HELENA, AL	1506(94)	2.654	2.679	0.9	7.1 0.0	12	22	100	1.4
0168	VULCAN MATERIALS COMPANY SCOTTSBORO, AL	1474(92)	2.690	2.705	0.5	8.7 0.6	19	26	99	4.7
0169	VULCAN MATERIALS COMPANY TUSCUMBIA, AL	1362(85)	2.607	2.632	1.0	6.0 0.0	46	26	99	2.0
0170	WADE SAND & GRAVEL EAST THOMAS QUARRY BIRMINGHAM, AL	1426(89)	2.754	2.780	0.9	5.8 0.6	15	25	99	3.1
0315	MARTIN MARIETTA MATERIALS MAYLENE, AL	1410(88)	2.780	2.810	0.7	7.4 0.0	22	20	99	2.1
0414	VULCAN MATERIALS COMPANY CHILDERSBURG, AL	1426(89)	2.705	2.719	0.5	12.5 7.5	21	25	99	3.1
0497	VULCAN MATERIALS COMPANY HUNTSVILLE, AL	1554(97)	2.632	2.662	1.1	11.7 0.0	22	26	97	2.7
0624	BLUE WATER INDUSTRIES ALLSBORO, AL	1442(90)	2.506	2.554	2.0	5.91 0.18	28	28	98	3.1
0654	MADISON MATERIALS COMPANY GUNTERSVILLE, AL	1378(86)	2.682	2.702	0.7	2.0 1.2	16	26	99	1.9
0783	MADISON MATERIALS COMPANY SUMMIT, AL	1330(83)	2.694	2.711	0.6	20.0 2.0	16	27	99	3.0

CRUSHED LIMESTONE
TYPE I

ID NUMBER	SOURCE NAME/LOCATION	LOOSE UNIT MASS kg/m^3 (lbs/ft^3)	BULK SPECIFIC GRAVITY	BULK SPECIFIC GRAVITY (SSD)	ABSORPTION %	FLAT/ELONG. 3:1 5:1	L.A. ABRASION %	BRITISH POLISHING NUMBER (BPN)	SODIUM SULFATE SOUNDNESS % SOUND	TOTAL SILICA CONTENT %
0910	HOOVER, INC. LOWER LEVEL HUNTSVILLE, AL	1474(92)	2.650	2.672	0.9	10.4 0.0	22	26	99	2.1
1405-G	MARTIN MARIETTA MATERIALS O'NEIL QUARRY SAGINAW, AL	1410(88)	2.692	2.719	1.0	12.7 1.1	19	24	99	2.1
1414	MCCARTNEY CONSTRUCTION COMPANY SPEEDWAY QUARRY EASTABOGA, AL	1474(92)	2.797	2.816	0.7	12.3 1.2	12	25	99	1.9
1504	MARTIN MARIETTA MATERIALS VANCE, AL	1506(94)	2.688	2.714	0.9	9.7 0.0	13	25	100	2.2
1549	VULCAN MATERIALS COMPANY BESSEMER, AL	1474(92)	2.726	2.764	1.3	16.2 1.9	18	24	100	4.0
1608	VULCAN MATERIALS COMPANY TUSCALOOSA QUARRY VANCE, AL	1458(91)	2.722	2.755	1.2	17.3 0.9	16	26	96	3.3
1612	BLOUNT SPRINGS SAND & GRAVEL BATTLEGROUND QUARRY FALKVILLE, AL	1410(88)	2.671	2.687	0.6	3.2 0.0	19	28	100	2.8
1616	MK MATERIALS RUSSELLVILLE, AL	1330(83)	2.532	2.580	2.0	10.1 1.8	28	31	97	1.1
1651	VULCAN MATERIALS COMPANY SOUTH RUSSELLVILLE QUARRY RUSSELLVILLE, AL	1121(70)	2.617	2.676	1.1	12.7 0.6	22	28	98	2.1

CRUSHED LIMESTONE
TYPE I

ID NUMBER	SOURCE NAME/LOCATION	LOOSE UNIT MASS kg/m^3 (lbs/ft^3)	BULK SPECIFIC GRAVITY	BULK SPECIFIC GRAVITY (SSD)	ABSORPTION %	FLAT/ELONG. 3:1 5:1	L.A. ABRASION %	BRITISH POLISHING NUMBER (BPN)	SODIUM SULFATE SOUNDNESS % SOUND	TOTAL SILICA CONTENT %
1656	VULCAN MATERIALS COMPANY PRIDE QUARRY TUSCUMBIA, AL	1346(84)	2.502	2.555	2.1	10.6 0.0	36	29	97	3.9
1664	BLOUNT SPRINGS SAND & GRAVEL CLAYTON QUARRY ATTALLA, AL	1346(84)	2.671	2.688	0.6	4.5 0.0	13	27	99	2.2
1709	BLOUNT SPRINGS SAND & GRAVEL ASHVILLE QUARRY ASHVILLE, AL	1394(87)	2.650	2.680	0.1	13.4 1.1	17	26	100	2.2
1728	VULCAN MATERIALS COMPANY LAKESHORE QUARRY BIRMINGHAM, AL	1442(90)	2.678	2.719	1.5	19.3 0.3	10	25	99	1.7
1758	ROGERS GROUP, INC. LACEY'S SPRINGS, AL	1410(88)	2.664	2.687	0.9	20.0 4.4	22	25	99	4.5
1770	VULCAN MATERIALS COMPANY NORTH BIRMINGHAM QUARRY BIRMINGHAM, AL	1490(93)	2.689	2.702	0.5	14.0 0.7	21	26	100	6.3
1802	ROGERS GROUP, INC. SOUTH LIMESTONE TANNER, AL	1362(85)	2.604	2.642	1.5	16.5 3.6	20	31	99	4.5
1821	VULCAN MATERIALS CO. GURLY, AL	1362(85)	2.643	2.667	0.9	11.0 1.0	19	27	99	1.8
1839	REED CONTRACTING SERVICES HOLLYWOOD, AL	1474(92)	2.640	2.670	1.2	14.4 0.6	15	23	99	1.8
1844	ROGERS GROUP, INC. HUNTSVILLE, AL WEST HUNTSVILLE QUARRY	1362(85)	2.636	2.672	1.4	16.5 3.6	19	29	98	3.2

CRUSHED LIMESTONE
TYPE II

ID NUMBER	SOURCE NAME/LOCATION	LOOSE UNIT MASS kg/m^3 (lbs/ft^3)	BULK SPECIFIC GRAVITY	BULK SPECIFIC GRAVITY (SSD)	ABSORPTION %	FLAT/ELONG. 3:1 5:1	L.A. ABRASION %	BRITISH POLISHING NUMBER (BPN)	SODIUM SULFATE SOUNDNESS % SOUND	TOTAL SILICA CONTENT %
0511	VULCAN MATERIALS COMPANY CHATTANOOGA, TN	1426(89)	2.739	2.762	0.8	13.6 0.6	17	25	100	1.8
1456	ROGERS GROUP, INC. PULASKI, TN	1410(88)	2.669	2.688	0.7	25.6 1.6	15	27	99	3.9
1470	VULCAN MATERIALS COMPANY HERMITAGE QUARRY HERMITAGE, TN	1410(88)	2.678	2.695	0.6	8.0 2.1	15	28	99	2.4
1643	VULCAN MATERIALS COMPANY PARSONS, TN	1426(89)	2.682	2.699	0.6	8.2 0.0	28	27	98	3.3
1692	MARTIN MARIETTA MATERIALS SIX MILE QUARRY ROME, GA	1458(91)	2.642	2.668	2.7	19.0 0.3	23	22	99	7.1
1725	VULCAN MATERIALS COMPANY FRANKLIN QUARRY FRANKLIN, TN	1490(93)	2.612	2.657	1.7	2.3 0.0	27	32	94	1.8
1726	VULCAN MATERIALS COMPANY DANELY QUARRY NASHVILLE, TN	1378(86)	2.603	2.632	1.1	20.0 2.6	46	26	97	3.2
1792	ROGERS GROUP, INC. GORDONSVILLE, TN	1426(89)	2.710	2.740	1.1	7.3 0.0	15	26	98	4.5
1834	ROGERS GROUP, INC. WHITES CREEK QUARRY NASHVILLE, TN	1426(89)	2.612	2.662	2.0	12.5 2.0	35	38	100	1.0
1843	BLUEWATER INDUSTRIES GREENBACK QUARRY LENOIR CITY, TN SEE NOTE 5	1490(93)	2.757	2.780	0.8	17.9 0.1	17	26	100	3.1

CRUSHED LIMESTONE
TYPE III

ID NUMBER	SOURCE NAME/LOCATION	LOOSE UNIT MASS kg/m^3 (lbs/ft^3)	BULK SPECIFIC GRAVITY	BULK SPECIFIC GRAVITY (SSD)	ABSORPTION %	FLAT/ELONG. 3:1 5:1	L.A. ABRASION %	BRITISH POLISHING NUMBER (BPN)	SODIUM SULFATE SOUNDNESS % SOUND	TOTAL SILICA CONTENT %
0925	VULCAN MATERIALS CO. / ICA CALICA QUARRY QUINTANA ROO, MEXICO	1201(75)	2.262	2.372	5.0	4.1 0.0	32	29	99	3.2
0927	LAFARGE NORTH AMERICA THREE RIVER QUARRY SMITHLAND, KY	1474(92)	2.622	2.660	1.4	11.3 0.0	19	30	98	1.9
0930	MARTIN MARIETTA MATERIALS BAHAMA ROCK FREEPORT, GRAND BAHAMAS	1201(75)	2.305	2.398	4.0	5.5 0.0	29	26	98	1.8
1654	APAC MIDSOUTH - BRICKEY'S STONE									
	BURLINGTON FORM.	1394(87)	2.573	2.621	1.9	14.0 / 1.0	23	25	97	4.2
	KISSMICK FORM.	1297(81)	2.550	2.604	2.1	16.3 / 2.1	37	29	95	2.2
	PLATTIN FORM. BLOOMSDALE, MO	1426(89)	2.572	2.619	1.8	9.1 / 1.8	28	24	95	3.3
1745	PINE BLUFF SAND AND GRAVEL SALEM, KY SEE NOTE 5	2179(136)	2.646	2.673	1.0	15.3 0.0	21	29	98	4.7
1782	WARREN PAVING SLATS LUCAS QUARRY SALEM, KY	1426(89)	2.652	2.672	0.8	15.2 0.4	19	29	99	5.4
1831	WINN MATERIALS-GRAND RIVERS QUARRY	1330(83)	2.596	2.632	1.4	5.0 / 0.4	25	30	97	3.0
	(WARSAW FORMATION) (ST. LOUIS FORMATION) SEE NOTE 5 GRAND RIVERS, KY	2114(132)	2.675	2.692	0.6	13.4 / 0.7	18	30	99	3.8
1850	ALANZA AGREGADOS PUERTO CORTES, HONDURAS	1394(87)	2.619	2.656	1.4	10.8 0.0	17	31	98	3.5

NOTES

NOTE 1: Material Shall not be used in Portland Cement Concrete or Bituminous Applications, due to L.A. Abrasion as per Section 801.03(b) of the ALDOT Specifications. Materials is suitable for all other uses.

NOTE 2: Material shall not be used in Portland Cement Concrete due to total silica content per Section 801.02 of the ALDOT Specifications, unless otherwise mitigated in mix per ALDOT Specifications 801.

NOTE 3: Material has limited use as per Section 423 and/or 424 of the ALDOT Specifications, due to its sand equivalent number.

NOTE 4: Material Shall not be used in bituminous plant mixes and bridge superstructure concrete (except prestressed concrete), due to its bulk specific gravity per Section 801.03(a), and/or absorption per Section 801.02(g) of the ALDOT Specifications.

NOTE 5: Material shall not be used in bituminous surface treatments or concrete as per Section 801.02 of the ALDOT Specifications due to wash test failure.

NOTE 6: Material shall not be used in Stone Matrix Asphalt (SMA) as per Section 423.02(a) of the ALDOT Specifications due to its content of 3:1 flat and elongated particles.

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