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16. Abstract

The main objective of this study was to develop a Louisiana pavement surface friction guideline that considers polished stone value (PSV) and mixture type alike in terms of both micro- and macro- surface textures. The polishing and texture properties of aggregates were characterized using the British Pendulum, Micro-Deval and Aggregate Imaging System (AIMS). Asphalt mixture slabs were fabricated with different combinations of two aggregate sources (sandstone and limestone) and four mixture types and polished by a three-wheel accelerated polishing device developed by the National Center for Asphalt Technology (NCAT). The surface frictional characteristics of each slab were measured by Dynamic Fiction tester (DFT) and Circular Texture Meter (CTM) at various pre-determined polishing cycles. In addition, an inventory dataset of field friction number (FN) measurements was obtained from the LADOTD's Materials Laboratory and analyzed in this study to determine the effects of traffic loading, aggregate and mixture types on the measured FN values.

The laboratory results indicated that the accelerated polishing device used in this study performed just as the expectation; i.e., as the polishing cycle increases, the measured frictional property of testing slab surface decreases. It was found that the DFT measurements were fairly sensitive to the coarse aggregate types (related to micro-texture) used in mix design, but were not very sensitive to different mix types or aggregate gradations (related to macro-texture). The analysis of CTM measured Mean Profile Depth (MPD) results confirmed a strong relationship between MPD and mixture type, indicating MPD does reflect well of surface macro-texture. Because friction resistance of an asphalt mixture should account for both micro- and macro-texture, the International Friction Index (IFI) friction numbers, the F(60), were determined based on an IFI model using measured DF_{20} (the DFT measurement at a friction speed of 20 mi/hr) and MPD values for each slab tested. Further analysis of F(60) results generally indicated that an open-graded friction coarse (OGFC) mix type considered in this study had the highest friction resistance due to its largest surface macro-texture (or MPD values), followed by the stone matrix asphalt (SMA) mix type, and then by the two Superpave mix types considered (a 19-mm Superpave Level-II mix, a 12.5-mm Superpave Level-II mix). The F(60) results also indicated that a selected sandstone type (AB13) with a high polishing resistance (PSV>37) performed significantly better in terms of mixture friction resistance than a selected limestone (AA50) with an PSV of 31. Mixtures using an aggregate blend of 30 percent of selected sandstone and 70 percent of the limestone tended to have a better surface friction resistance than those with 100 percent of the limestone. This observation demonstrates that blending of low and high friction aggregates together can possibly produce an asphalt mixture with an adequate field friction resistance.

The analysis has led to the development of a set of prediction models of mixture frictional properties, and a laboratory mix design procedure that addresses the surface friction resistance of an asphalt mixture in terms of both micro- and macro- surface textures. The developed frictional mix design procedure allows estimating a friction-demand based, design SN value for an asphalt mixture during the mix design stage.

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Development of Surface Friction Guidelines for LADOTD

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ABSTRACT

The main objective of this study was to develop a Louisiana pavement surface friction guideline that considers polished stone value (PSV) and mixture type alike in terms of both micro- and macro- surface textures. The polishing and texture properties of aggregates were characterized using the British Pendulum, Micro-Deval, and Aggregate Imaging System (AIMS). Asphalt mixture slabs were fabricated with different combinations of two aggregate sources (sandstone and limestone) and four mixture types and polished by a three-wheel accelerated polishing device available at the National Center for Asphalt Technology (NCAT). The surface frictional characteristics of each slab were measured by the dynamic fiction tester (DFT) and circular texture meter (CTM) at various pre-determined polishing cycles. In addition, an inventory dataset of field friction-number (FN) measurements was obtained from the Louisiana Department of Transportation and Development's (LADOTD) Materials Laboratory and analyzed in this study to determine the effects of traffic loading and aggregate and mixture types on the measured FN values.

The laboratory results indicated that the accelerated polishing device used in this study performed just as expected, i.e., as the polishing cycle increases, the measured frictional property of testing slab surface decreases. It was found that the DFT measurements were fairly sensitive to the coarse aggregate types (related to micro-texture) used in mix design, but DFT was not very sensitive to different mix types or aggregate gradations (related to macro-texture). The analysis of CTM measured mean profile depth (MPD) results confirmed a strong relationship between MPD and mixture type, indicating MPD does reflect well of surface macro-texture. Because friction resistance of an asphalt mixture should account for both micro- and macro-textures, the International Friction Index (IFI) friction numbers, the F(60), were determined based on an IFI model using measured DF_{20} (the DFT measurement at a friction speed of 20 mi/hr) and MPD values for each slab tested. Further analysis of F(60) results generally indicated that an open-graded friction coarse (OGFC) mix type considered in this study had the highest friction resistance due to its largest surface macro-texture (or MPD values), followed by the stone matrix asphalt (SMA) mix type, and then by the two Superpave mix types considered (a 19-mm Superpave Level-II mix and a 12.5-mm Superpave Level-II mix). The F(60) results also indicated that a selected sandstone type (AB13) with a high polishing resistance (PSV>37) performed significantly better in terms of mixture friction resistance than a selected limestone (AA50) with an PSV of 31. Mixtures using an aggregate blend of 30 percent of AB13 sandstone and 70 percent of AA50 limestone tended to have a better surface friction resistance than those with 100 percent of the limestone. This observation demonstrates that blending of low- and high-friction aggregates

together can possibly produce an asphalt mixture with an adequate field friction resistance.

The analysis has led to the development of a set of prediction models of mixture frictional properties and a laboratory mix design procedure that addresses the surface friction resistance of an asphalt mixture in terms of both micro- and macro-surface textures. The developed frictional mix design procedure allows estimating a friction-demand based, design SN value for an asphalt mixture during the mix design stage.

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IMPLEMENTATION STATEMENT

The developed frictional mixture design procedure based on both micro- and macro-textures should be considered for implementation in the wearing course mix design of LADOTD. The lab and field validation should be performed before the implementation.

LADOTD should also consider implementing the results of the NCHRP 1-43, *Guide for Pavement Friction*, for the management of pavement friction on existing highways in which three to five site categories based on friction demand levels may be established and the corresponding intervention and investigatory levels of friction number values for each category may be determined to guide the frictional mix design.

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INTRODUCTION

Pavement surface friction is a current critical issue to highway safety. Historical data indicate that traffic accidents cause nearly 2.5 million injuries and over 41,000 fatalities annually in the United States (US) (Larson, 2005; Larson et al., 2008). According to the National Transportation Safety Board (NTSB), approximately 13.5 percent of fatal crashes and 25 percent of all crashes occur under wet pavement conditions (Kuemmel et al., 2000).

Factors associated with those crashes may be summarized into three main categories: driver related, vehicle related, and highway condition related (Noyce et al., 2005). Out of the three categories only the highway condition factors may be controlled by highway agencies. This has led to the strong interests at both the federal and state level in advancing crash reduction programs with specific attention focusing on better understanding the relationship between measurable surface characteristics (e.g., friction and texture) and the occurrence of wet-pavement crashes (Larson et al., 2008). On the other hand, the National Co-Operative Highway Research Program (NCHRP) Project 1-43: *Guide for Pavement Friction* recommends developing laboratory mix design procedures to address friction and texture together in order to provide better friction resistant surface mixtures (Hall et al., 2009).

The current Louisiana friction guidelines for a wearing course mixture design are based on the PSV of a coarse aggregate (which is a relative British Pendulum friction number measured on polished stones) (Road and Bridge Specification LADOTD, 2002). The basic assumption is that aggregates with a high polished stone value will automatically provide high friction resistance for a wearing course mixture. However, the field measurement on friction resistance sometimes does not necessarily support such an assumption. In fact, there are many parameters that may affect the friction resistance of a wearing course mixture and the polished stone value is just one of these parameters. The NCHRP 1-43 examined several friction-influential parameters related to a mixture design. Among them include mixture type, surface textures (micro and macro textures), polished stone value, and other aggregate and binder properties (Hall et al., 2009). Obviously, the use of only PSV of coarse aggregates would have somewhat clouded the fundamental issues related to friction resistance of a pavement surface.

In addition, since very limited highly friction-resistant aggregates are locally produced in Louisiana, such friction guidelines will tend to screen out locally available materials by requiring imported high friction-resistant aggregates in a wearing course construction, which is usually not cost-effective. Therefore, there is a need to re-examine the current friction guidelines and develop new guidelines in which more frictional characteristics can be considered in a wearing course mixture design. Ideally, the new guidelines will allow more locally available aggregates to be used in a wearing course mixture.

Background and Summary of Literature Review

Pavement Friction

The pavement friction is defined as the resisting force developed between vehicle tire and pavement surface which always acts in the opposite direction of vehicle motion. Pavement surface friction is a significant driving safety factor and plays a critical role in reducing wet-pavement crashes (FHWA, 1980; Li et al., 2005).

Friction resistance is the friction force developed at the contact area of tire and pavement (Noyce et al., 2005). Friction resistance is the pavement friction that resists sliding of vehicle tires on pavement surfaces. One of the common friction resistance measuring devices is the locked wheel skid tester (LWST), which gives the friction resistance or FN value of the pavement. According to ASTM committee E17, friction resistance is defined as the retarding force generated by the interaction between a pavement and a tire under a locked non-rotating condition (Henry et al., 2000). LADOTD uses the LWST machine to measure the in-situ friction of the pavements in Louisiana.

Several factors contribute to developing friction at the tire pavement interface and can be grouped into four major types: pavement surface characteristics, vehicle operating parameters, tire properties, and environmental factors. The friction influencing factors are given in Table 1(Wallman et al., 2001; Sandberg et al., 1997; Kummer et al., 1966). Of the four major types listed in Table 1, it may be important to note that this research focuses on the first type of factors only since the others types (factors) are beyond our control.

Pavement Friction Mechanism

Friction forces in rubber (tire) consists mainly of two components called adhesive and hysteresis (Moore, 1972). Those two components are shown in Figure 1 (Hall et al., 2009).

Adhesion. Adhesion is the friction force developed by shearing between tire and pavement at the contact area (Zimmer et al., 2003; Choubane et al., 2004). This friction force is mainly contributed by the micro-texture (surface roughness) of the road pavement because adhesion force is developed at tire-pavement interface. The small scale bonding and interlocking between rubber and pavement aggregate gives rise to this adhesion. At typical driving speed adhesion accounts for two-thirds of friction resistance developed at the tire-pavement interface (Hogervorst, 1974).

Table 1
Factors affecting the pavement friction

Pavement Surface Characteristics	Vehicle Operating Parameters	Tire Properties	Environment
• Micro-Texture • Macro-Texture • Mega-Texture • Unevenness • Material Properties • Temperature • Thermal conductivity	• Slip Speed • Vehicle Speed • Braking Action • Driving Maneuver • Turning • Overtaking	• Foot Print • Tread Design and Condition • Rubber composition and hardness • Inflation Pressure • Sliding velocity • Load • Temperature • Thermal conductivity • Specific Heat	• Climate • Wind • Temperature • Water (rainfall, condensation) • Snow and Ice • Contamination (Fluid) • Anti-skid material (salt, sand) • Dirt, mud , debris • Viscosity • Density • Film thickness • Temperature • Thermal Conductivity • Specific Heat

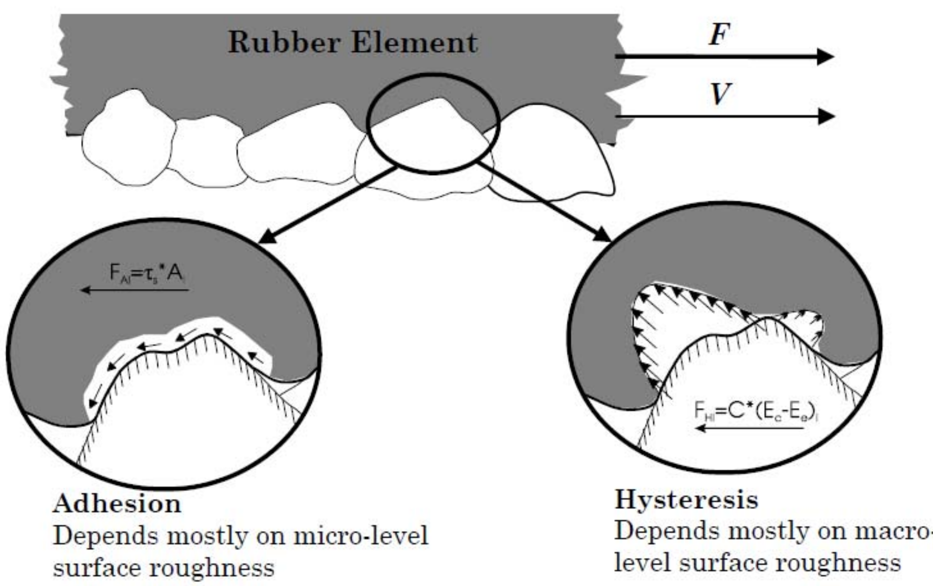

Figure 1
Adhesion and hysteresis mechanism of tire-pavement friction (Hall et al., 2009)

Hysteresis. Tire rubber stores deformation energy when the tire compresses against the pavement. When the tire comes to the state of relaxation, part of the energy stored is recovered, while part of the energy is lost as the form of energy. This loss of energy induces the friction force, which is called hysteresis (Linder et al., 2004). The hysteresis is mainly dependent on the macro-texture (surface roughness) of the pavement, since the tire makes an envelope surface at the tire-pavement interface (Hall et al., 2009).

Other components also contribute to the total friction force such as tire rubber shear, but they are insignificant in comparison with adhesion and hysteresis. The sum of these two components account for the total friction developed in the interface of tire-pavement interface.

The friction force acts in both longitudinal and lateral directions to the tire. Depending upon the direction of force, pavement friction force can be divided into a longitudinal and lateral frictional force. Longitudinal force acts in the longitudinal direction of the pavement surface while the vehicle tire is in free rolling or constant brake mode. The relative speed between the circumference of tires and the pavement is termed as slip speed. In the free rolling condition, the slip speed is zero while in the constant braked or locked mode; the slip speed reaches to the maximum. The following relationship describes slip speed (Meyer, 1982):

$$S = V - V_P = V - (0.68 \times \ \omega \ \times r) \tag{1}$$

where,

S = Slip speed mi/hr.;

V = Vehicle speed mi/hr.;

V_P = Average peripheral Speed of the tire, mi/hr.;

ω = Angular velocity of tire, radians /sec.; and

r = Average radius of the tire, ft.

Slip ratio is defined as the ratio of slip speed to the vehicle speed. The slip ratio is zero when the tire is in free rolling condition since V_P is equal to V. The slip ratio is 100% when the tire is locked, since V_P is zero, illustrated by equation (2) (Meyer, 1982).

$$SR = \frac{V-V_P}{V} \times 100 = \frac{S}{V} \times 100 \tag{2}$$

where,

SR = Slip ratio, percent.;

V = Vehicle speed mi/hr.;

V_P = Average peripheral Speed of the tire, mi/hr.; and

S = Slip speed mi/hr.

When the tire is in motion, the weight of the vehicle lies at the center, but the ground force is offset by the amount α. This offset gives rise to a moment that is encountered by a force to rotate the tire and is called rolling resistance force (F_R). The rolling resistance force (F_R) increases with increasing speed, because α increases with the speed (Henry, 2000).

An additional force called breaking slip force (F_B) is required to counter the added moment (M_B) created by breaking. This force is proportional to the degree of braking and the resulting slip ratio. The free rolling resistance force (F_R) combined with the braking slip force (F_B) gives the total frictional force developed (Henry, 2000).

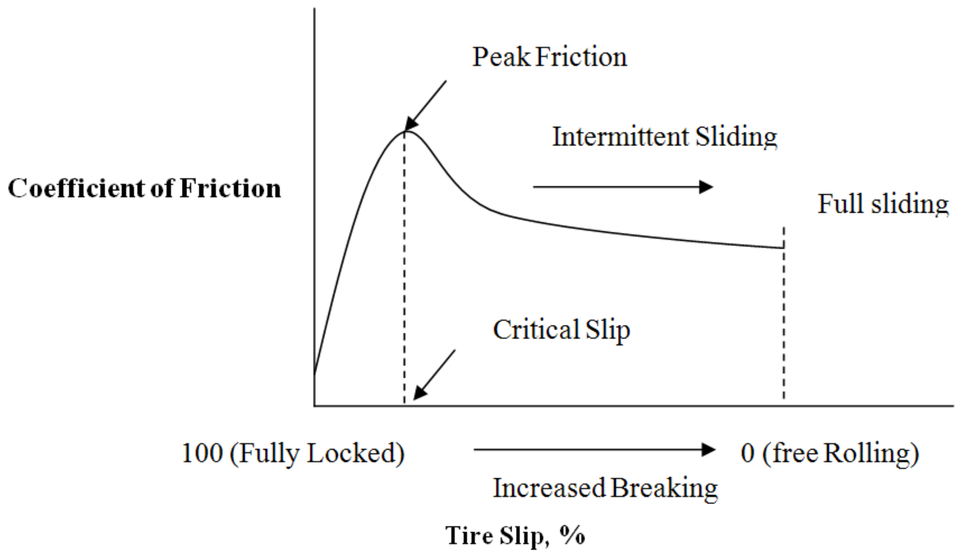

Figure 2
Pavement friction versus tire slip (Henry, 2000)

As shown in Figure 2, the coefficient of friction between tire and the road surface varies with the increasing tire slip. The coefficient of friction first rises to a peak level with increasing slip then decreases. Increased slip ratio means increased breaking. The maximum value of friction occurs just after applying the brake. The difference between peak friction and sliding value may be up to 50 percent of sliding value (Henry, 2000).

Pavement Surface Texture

Various researchers have attempted to establish relationships between pavement friction and texture of pavement surface. Yandell and Sawyer illustrated the effect of texture shape on the hysteresis friction (Yandell and Sawyer, 1994). Forster showed that pavement friction can be explained by micro-texture with the help of linear regression analysis (Forster, 1989).

Roberts showed that material properties and the separation velocity are the causes of friction force and energy dissipation between tire and pavement surface (Roberts, 1988).

Pavement surface texture is defined as the asperities present in the pavement surface (Kummer et al., 1963). The asperities are measured as the deviation of the surface from true planar surface (Noyce et al., 2005). Those deviations can be further defined by wavelength (λ) and peak to peak amplitude (A) of aggregate asperities. The pavement surface can be characterized by three levels of textures: mega-texture, macro-texture, and micro-texture (Dewey et al., 2001). The pavement texture having amplitude more than 2 in. (50 mm) is called unevenness or roughness. The wavelength (λ) and amplitude for different types of textures are listed below (Hall et al., 2009):

- Roughness/Unevenness: > Mega-Texture
- Mega-Texture: 20> λ > 2 in. (500 > λ > 50 mm) Amplitude: 0.005 to 2 in (0.1 to 50 mm) texture as the wavelength same as
- Macro-Texture: 2> λ > 0.02 in. (50 > λ > 0.5 mm) Amplitude: 0.005 to 0.8 in (0.1 to 20 mm)
- Micro-Texture: λ < 0.02 in. (λ < 0.5 mm) Amplitude: (1 to 500 µm): It is the degree of roughness given by individual aggregate particle.

Out of these three types of textures, the macro- and micro-textures are the predominant features shown in Figure 3 for the road pavement friction (ASTM E 867). Micro-texture is associated with the microscopic feature of aggregates. The micro-texture is significant at the slow speed of vehicles as it is believed to cause adhesion between tire and the pavement surface; whereas, macro-texture is responsible for the hysteresis friction and for the hydroplaning (Noyce et al., 2005). Hydroplaning is the obstruction in passage of water at the pavement-tire interface through the tread of the tire (Moore, 1975). The different characteristics of textures by the wavelength are illustrated in Figure 4 (Hall et al., 2009).

Peak brake coefficients of a standard test tire are related to the micro- and macro-texture of the pavement surface (Bond et al., 1976). Further Leu and Henry (1978) showed that friction resistance of different pavements are different based on their micro- and macro-texture. Davis et al. (2002) illustrated the significance of mixture property on the friction resistance measurement and laser profile mean texture depth measurements and stated that frictional properties of surface course can be predicted by hot mix asphalt (HMA) mix design. However, Horne and Buhlmann (1983) showed that the surface friction measurements are not represented well by pavement texture.

Micro- and macro-textures both influence the change in friction resistance with vehicle speeds (Hogervorst, 1974). A high speed of vehicle macro-texture influences the friction resistance by reducing the friction-speed gradient and facilitating the drainage of water; whereas, micro-texture influences the friction resistance at low speeds (Rose and Gallaway, 1970; Hall et al., 2006). An average texture depth of about 0.5 mm is a required minimum texture depth to ascertain the drainage of water from beneath the tire (Bloem, 1971). The hydroplaning on the pavement surface is also affected by micro-textures (Pelloli, 1972; Moore, 1975, Bond et al., 1976; Horne, 1977; Ong et al., 2005).

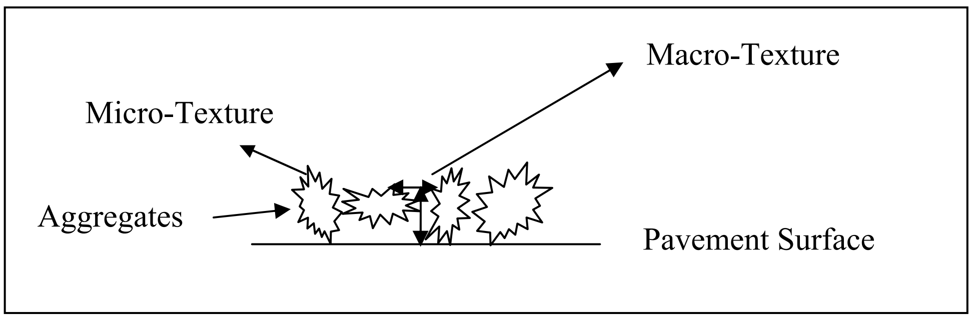

Figure 3
Microscopic view of pavement surface showing micro- and macro-texture

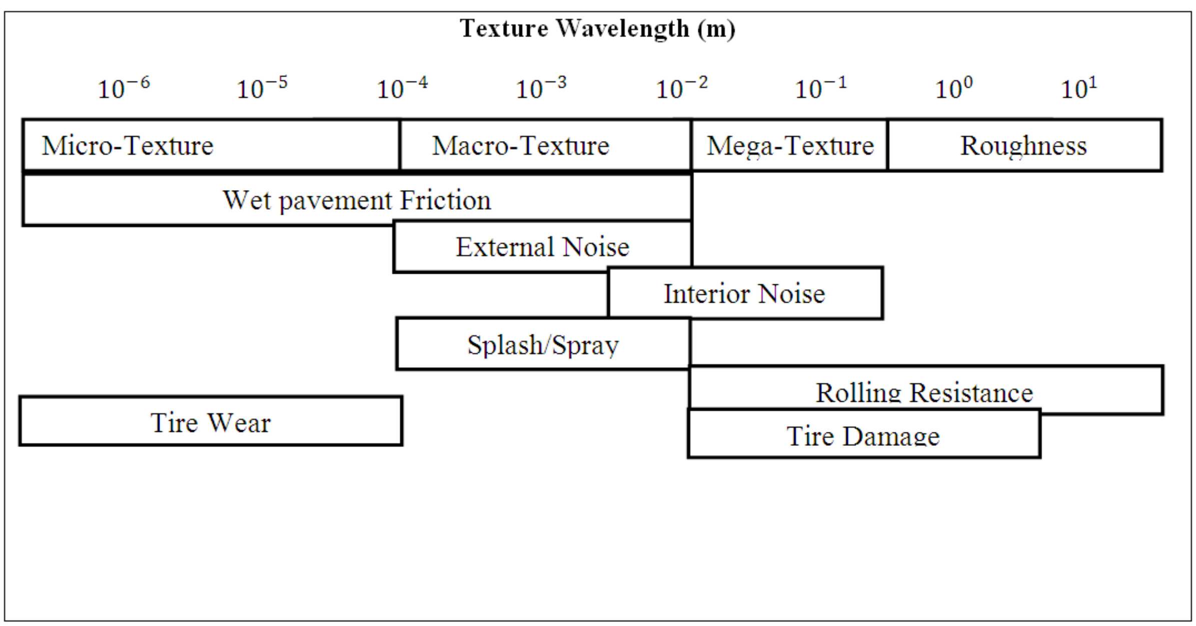

Figure 4
Texture wavelength effect on surface characteristics (Hall et al., 2009)

Measurement of pavement surface texture has been a common practice in recent years (Abe et al., 2000; Henry, 2000). Henry and Liu (1978) stated that British pendulum test (BPT) numbers can be used to represent micro-texture. BPT provides only the measure of frictional property of aggregates and pavement surfaces at low speeds (Saito et al., 1996). However some researchers showed that BPT performance was unreliable when tested on coarse textured pavement surfaces (Forde et al., 1976; Salt, 1977; Purushothaman et al., 1988). The circular texture meter is a relatively new macro-texture measuring device based on laser profiling and measures the MPD of the pavement surface (Henry et al., 2000; Abe et al., 2000; Noyce et al., 2005). Masad et al. (2005) introduced aggregate imaging system (AIMS), which is a direct texture measuring system by use of a microscope and digital image processing.

The resistance to polishing under the traffic loading is a highly desirable property of aggregates used in wearing course mix design (Whitchurst and Goodwin, 1955; Nichols et al., 1957; Gray and Renninger, 1965; Balmer and Colley, 1966; Csathy et al., 1968; Moore, 1969; Bloem, 1971; Hall et al., 2009). Different aggregates have different abilities to maintain their micro-texture against polishing (Kowalski, 2007). Coarse aggregate angularity and abrasion resistance have a significant effect on the friction resistance in pavements (Masad et al., 2005). Also pavement temperature has a significant effect on pavement frictional properties (Flintsch et al., 2005).

IFI (International Friction Index)

To harmonize the friction measurements by different devices, the World Road Association - Permanent International Association of Road Congresses (PIARC) performed an experiment in Belgium and Spain in 1992 and came up with a new friction index, IFI (Wambold et al., 1995). The IFI consists of two numbers that describe the friction resistance of pavement: speed constant (S_P) and friction number F(60).The general notation for IFI is IFI [F(60), S_P]. The number 60 in friction number F(60) denotes the test vehicle speed of 60 km/hr, though IFI can represent friction at different test speeds. The speed constant (S_P) is correlated with the result of a macro-texture measurement (Wambold et al., 1995);

$$S_P = a + b \times TX \tag{3}$$

where,

S_P = IFI speed number;

a,b = Calibration constants dependent on the method used to measure macro-texture;

For Mean Profile Depth (MPD) (ASTM E 1845), a = 14.2 and b = 89.7

For Mean Texture Depth (MTD) (ASTM E 965), a = -11.6 and b = 113.6; and

TX = Macro-texture (MPD or MTD) measurement, mm.

$$FR(60) = FR(S) \times e^{\frac{S-60}{S_P}} \tag{4}$$

where,
$FR(60)$= Adjusted value of $FR(S)$ at a slip speed of S to a slip speed of 60km/hr;
$FR(S)$ = Friction value at selected slip speed S; and
S = Selected slip speed km/hr.

$$F(60) = A + B \times FR(60) \times C \times TX \tag{5}$$

where,
$F(60)$ = IFI friction number obtained from equation (5); and
A, B, C = Calibration constant depends upon friction measuring device.

The DFT and CTM results are combined to calculate IFI for the mix slabs to evaluate their frictional resistance in terms of both micro- and macro-texture. A number of studies have already been done on the evaluation of IFI and its relationship with other friction test values. This very approach is adopted in this study to evaluate the frictional property of different mixes and to establish the relationship between DFT, CTM, and IFI values. This relationship can serve as a guide to the friction design for different mix types and aggregate blends. Hall et al. (2009) evaluates the status of micro- and macro-textures for the desired friction demand for pavement sections. Figure 5 suggested by Hall et al. indicates that it might be economically possible to achieve a same level of pavement friction by blending different aggregate types (micro-texture) with mixture types (macro-texture). This approach is applied to design the slab mixes. The viewpoint in this type of factorial design is to evaluate the effect of blending of low-friction aggregate with high-friction aggregate.

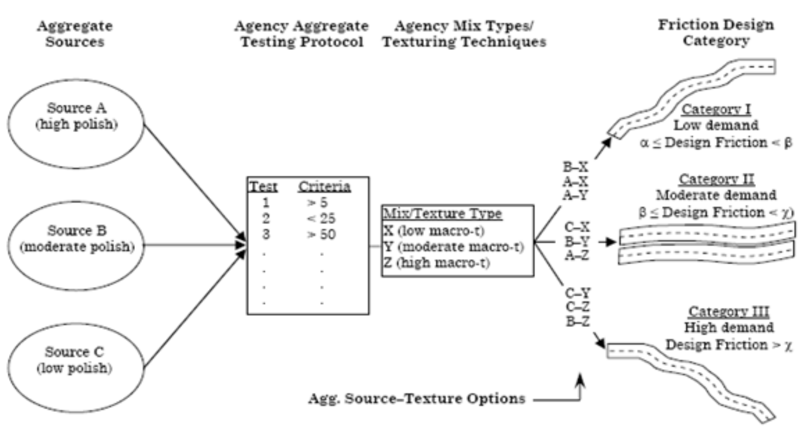

Figure 5
Example illustrations of matching aggregate sources and mix types/texturing techniques to meet friction demand

Sullivan et al. (2005) showed that the design vehicle stopping distance was expressed as a function of both micro- and macro-textures of a design surface mix. Figure 6 clearly explains that, with combination of both micro- and macro-texture, a less friction resistant aggregate may be used in a wearing coarse mixture in which a higher friction demand may be achieved through choosing a more friction resistance mixture type (e.g., OGFC or SMA) (Stephens et al., 1960; Kamel and Musgrove, 1981; Sullivan, 2005).

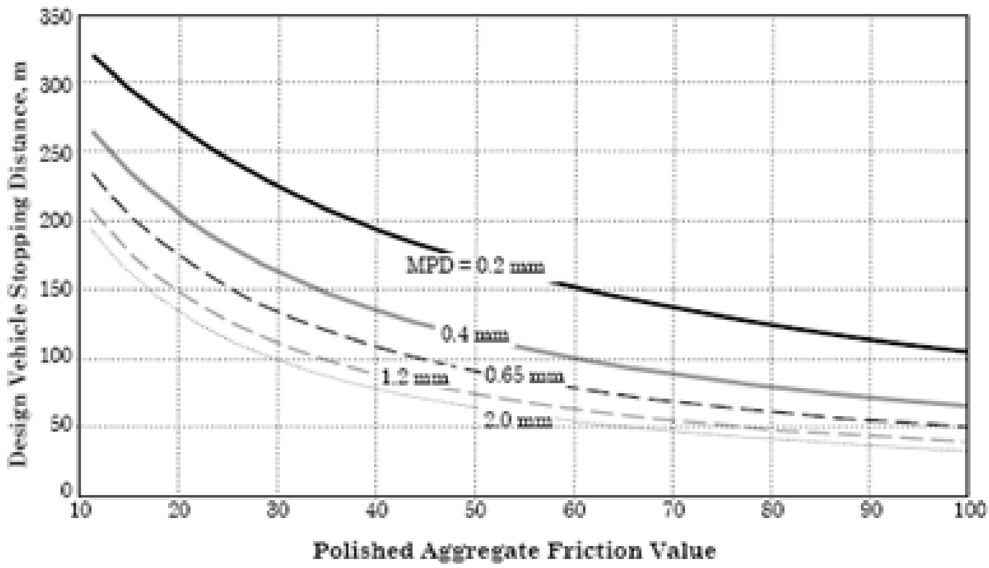

Figure 6
Illustration of vehicle response as function of PSV and MPD

The NCHRP 1-43 provides another illustration in which the requirement of DFT (20 km/hr) (micro-texture) for corresponding MPD (Macro-texture) and vice-versa can be evaluated for a desired friction level as shown in Figure 7 (Noyce et al., 2005; Khasawneh et al., 2008; Hall et al., 2009). Figure 7 is an example correlation between DFT(20) and F(60) for the specified MPD values to evaluate the choice of mix design for a specified friction level corresponding to specific micro- and macro-textures.

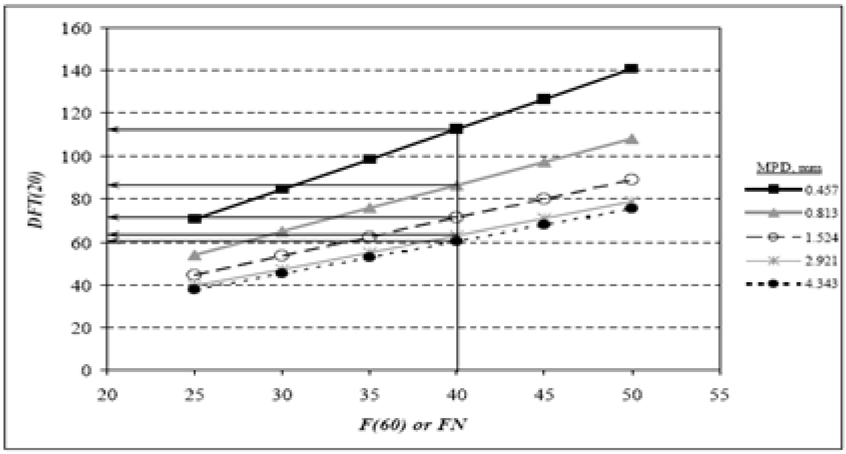

Figure 7
Example of determining DF_{20} and MPD needed to achieve a design friction level

LADOTD Current Friction Specification

The current friction specification of LADOTD is based on aggregate friction ratings. As shown in Table 2A, aggregates with high friction ratings (I or II) can be used for all wearing course mixtures, while low friction rating aggregates are used with certain restrictions.

Table 2A
LADOTD aggregate friction rating (LADOTD, 2008)

Friction Rating	Allowable Usage
I	All mixtures
II	All mixtures
III	All mixtures, except travel lane wearing courses with plan ADT greater than 7000[1]
IV	All mixtures, except travel lane wearing courses[2]

[1] When plan current average daily traffic (ADT) is greater than 7000, blending of Friction Rating III aggregates and Friction Rating I and/or II aggregates will be allowed for travel lane wearing courses at the following percentages. At least 30 percent by weight (mass) of the total aggregates shall have a Friction Rating of I, or at least 50 percent by weight (mass) of the total aggregate shall have a Friction Rating of II. The frictional aggregates used to obtain the required percentages shall not have more than 10 percent passing the No. 8 (2.36-mm) sieve.
[2] When the average daily traffic (ADT) is less than 2500, blending of Friction Rating IV aggregates with Friction Rating I and/or II aggregates will be allowed for travel lane wearing courses at the following percentages. At least 50 percent by weight (mass) of the total aggregate in the mixture shall have a Friction Rating of I or II. The frictional aggregates used to obtain the required percentages shall not have more than 10 percent passing the No. 8 (2.36-mm) sieve.

The aggregate friction ratings are based on the PSV values and empirical knowledge as shown in Table 2B.

Table 2B
Definition of friction rating (LADOTD 2008)

Friction Rating	Description
I	Aggregates that have a polish value of greater than 37 or demonstrate the ability to retain acceptable friction numbers for the life of the pavement.
II	Aggregates that have a polish value of 35 to 37 or demonstrate the ability to retain acceptable friction numbers for the life of the pavement.
III	Aggregates that have a polish value of 30 to 34 or demonstrate the ability to retain acceptable friction numbers for the life of the pavement.
IV	Aggregates with a polish value of 20 to 29.

It should be noted here that the above LADOTD friction specification is purely PSV, or micro-texture based. Literature review also indicated that some agencies have only specified the macro-texture in their friction design guidelines, such as the one developed by the French National Highway Administration (Dupont and Bauduin, 2005) as shown in Table 3 below. To get a balanced friction resistance design, LADOTD should evaluate a macro-texture based specification as a supplement to its current micro-texture based friction specification.

Table 3
French specification texture demand values (Dupont and Bauduin)

Site	Speed Limit (km/hr)	Type of Pavement	Plane Alignment	Length Profile	MTD_{spe}, mm	MTD_{min}, mm
Urban and suburban	$V \leq 50$	Two-directional	Urban arterial	–	≥ 0.40	0.30
	$50 < V < 90$				≥ 0.60	0.40
	$V \geq 90$	2 x 2 lanes	Urban expressway	Slope $\leq 5\%$	≥ 0.60	0.40
		2 x 3 lanes min.			≥ 0.70	0.50
Open country	$V = 90$	Two-directional	All cases	Slope $\leq 5\%$	≥ 0.60	0.40
				Slope $> 5\%$	≥ 0.80	0.60
	$V = 110$	2 x 2 lanes	All cases	Slope $\leq 5\%$	≥ 0.60	0.40
				Slope $> 5\%$	≥ 0.80	0.60
	$V = 130$	2 x 2 lanes	Radius > 600 m	Slope $< 5\%$	≥ 0.60	0.40
		2 x 3 lanes min.			≥ 0.70	0.50

Texas Mixture Friction Design Studies

Masad et al. (2009) reported that the friction outcome of an asphalt mix can be controlled and predicted with aggregate and mix properties. In their study, they suggested a regression equation to predict IFI for asphalt pavements based on aggregate gradation and resistance to polishing. The polishing effect on aggregate was analyzed with Micro-Deval and AIMS test results. The study includes a comprehensive analysis of DF_{20}, initial and terminal F(60) and their correlation with BPT and Micro-Deval test results. They showed that F(60) increases with the increase in BPT and Micro-Deval texture values. Equation (6) was proposed as a relationship to predict F(60) with mix, aggregate, and traffic properties (Masad et al., 2009).

$$F(60) = (a_{mix} + b_{mix}) \times \exp(-c_{mix} \times N) - a_{mix} \times \exp(-c_{mix} \times N) + a_{mix} \tag{6}$$

where,

F(60) = International Friction Number at speed 60 km/hr;

N = Number of increments of 1,000 polishing cycles (No. of polishing cycle /1000); and

$$a_{mix} = \text{Terminal } F(60) = (18.422 + \lambda) / (118.936 - 0.0013 + AMD^2) \tag{7}$$

$$(a_{mix} + b_{mix}) = \text{Initial } F(60)$$
$$= 0.4984 \times \ln(5.656 \times 10^{-2} \times (a_{agg} + b_{agg}) + 5.846 \times 10^{-2} \times \lambda - 4.985 \times 10^{-2} \times k) + 0.8 \tag{7-1}$$

$$c_{mix} = \text{Rate of change of } F(60) = 0.765 \times \exp(-7.297 \times 10^{-2} / c_{agg}) \tag{8}$$

in which,
λ, k = Weibull distribution scale factors for aggregate gradation;
AMD = Aggregate texture with AIMS after Micro-Deval test; and
a_{agg}, b_{agg}, and c_{agg} = Regression constants of equation (9) (Mahmoud, 2005; Luce, 2006).

$$\text{AIMS-Texture} = a_{agg} + b_{agg} \times \exp(-c_{agg} \times t) \tag{9}$$

where,
AIMS-Texture = Texture value obtained by AIMS;
a_{agg}, b_{agg} and c_{agg} = Regression constant; and
t = Time in Micro-Deval test.

In the second phase of the study, Masad et al. (2010) performed a field study to evaluate LWST friction number with DFT and CTM test results. This study was performed in relationship with their first phase laboratory study (Masad et al., 2009). The study showed that the friction number is affected by macro-texture for dense graded mixes; whereas, porous friction coarse mixes are affected by micro-texture property of the mix. They also suggested that the initial pavement micro-texture is dependent upon aggregate type, and DF_{20} results can be correlated with friction number for different mixes; whereas, DFT at a high speed (80 km/hr) can be correlated with friction number for only dense graded mixes. Further the study proposed a relationship [equation (10)] to predict field friction number (FN_{50}) by LWST with DFT and CTM results (Masad et al., 2010).

$$FN_{50} = 5.135 + 128.486\times(IFI - 0.045)\times\exp(-20/S_p) \tag{10}$$

where,
FN_{50} = Friction number from LWST at speed 50 mph;
IFI = International Friction Index; and
S_p = Speed constant.

Further IFI and S_p in equation (10) can be calculated with relationships given next (Wambold et al., 1995).

$$IFI = 0.081 + 0.732\times DF_{20}\times \exp(-40/S_p) \tag{11}$$

$$S_p = 14.2 + 89.7\times MPD \tag{12}$$

where,
DF_{20} = DFT result at speed 20 km/hr; and

MPD = Mean profile depth from CTM.

The same study by Masad et al. (2010) also proposed a relationship to calculate MPD from aggregate gradation.

$$MPD = 1.8 - (3.041/\lambda) - (0.382/k^2) \quad (13)$$

where,

λ and k = Weibull distribution scale factors for aggregate gradation.

OBJECTIVE

The main objective of this study was to develop a Louisiana pavement surface friction guideline that can consider the polished stone value and mixture type alike in terms of both micro- and macro-surface textures.

SCOPE

Frictional characteristics of typical Louisiana asphalt wearing course mixtures were evaluated in this study through a suite of laboratory accelerated polishing and friction tests. Laboratory aggregates tests included the Micro-Deval, British Pendulum, and aggregate imaging tests. Three-wheel accelerated polishing, DFT, and CTM tests were performed on selected asphalt mixtures. Results of laboratory tests were then analyzed through statistical comparison and correlation procedures and used to develop a frictional mix design procedure for wearing course mixtures in Louisiana considering both micro- and macro textures.

METHODOLOGY

A comprehensive laboratory testing program was designed in this study to evaluate the effects of different aggregates and asphalt mix types on pavement friction characteristics. Two aggregate sources and four typical Louisiana wearing course mix types were selected for the purpose of the research, which have resulted in a total of 12 different asphalt mixtures. Laboratory tests were conducted to determine the polishing and frictional properties for both aggregates and asphalt mixtures. Description of the laboratory experimental design, laboratory testing, and analysis procedures are presented below.

Laboratory Testing Program

Materials and Mix Design

Four typical Louisiana wearing course HMA mix types were considered in this study, namely, a 19-mm Superpave Level-II mix, a 12.5-mm Superpave Level-II mix, a SMA mix and an OGFC mix. Each mix type was further designed for three HMA mixtures based on one gradation, one asphalt binder, and three aggregate blends (i.e., 100 percent sandstone, 100 percent limestone and a combination blend of 70 percent limestone and 30 percent sandstone), resulting in 12 total HMA mixtures as outlined in Table 4. Note that the proportions used in the combination aggregate blend were for coarse aggregate portions of the HMA mixes only.

Table 4
Wearing course mixtures

No.	Mix Type	Mixture Designation
1	19-mm Superpave with 100% sandstone	SP19-SS
2	19-mm Superpave with 100% limestone	SP19-LS
3	19-mm Superpave with 70% limestone +30% sandstone	SP19-LS+SS
4	12.5-mm Superpave with 100% sandstone	SP12.5-SS
5	12.5-mm Superpave with 100% limestone	SP12.5-LS
6	12.5-mm Superpave with 70% limestone +30% sandstone	SP12.5-LS+SS
7	SMA with 100% sandstone	SMA-SS
8	SMA with 100% limestone	SMA-LS
9	SMA with 70% limestone +30% sandstone	SMA-LS+SS
10	OGFC with 100% sandstone	OGFC-SS
11	OGFC with 100% limestone	OGFC-LS
12	OGFC with 70% limestone +30% sandstone	OGFC-LS+SS

Aggregates. The crushed sandstone aggregate used in this study was supplied by Pine Bluff Sand & Gravel Co.; whereas, the crushed limestone aggregate selected was the silicious limestone obtained from the Vulcan Materials Co. According the Qualified Product List (QPL) of LADOTD, the sandstone source is designated as AB13 with a friction rating of I (the highest friction in QPL with a source PSV value of 38). The limestone is designated as AA50 in QPL with a friction rating of III (source PSV of 30). The selection of these two aggregates were based on two considerations: (1) both aggregates are the common aggregate types used in Louisiana wearing course mixtures; (2) it is possible to produce a mixture having a sufficient surface friction resistance by using a coarse aggregate blend mixed with high- and low-friction resistant aggregates (Ashby, 1980). As previously mentioned, a coarse aggregate blend of 70 percent lime stone and 30 percent sandstone was considered in the mix design of this study.

Asphalt Binder. The asphalt binder used in the mix design is classified as PG 76-22M (polymer modified), which was supplied by the Marathon, Inc., Baton Rouge, LA. The typical binder specification and lab test results are presented in Table 5.

Table 5
Lab test values and specification for the binder PG76-22 M

Tests	Spec	PG 76-22M
Test on Original Binder		
Dynamic Shear, G*/Sin(δ), (kPa)	1.00^{+}@ 76°C	1.82
Dynamic Shear, G*/Sin(δ), (kPa)	1.00^{+}@ 82°C	1.29
Force Ductility Ratio (F2/F1, 4°C, 5 cm/min, F2 @ 30 cm elongation		0.49
Rotational Viscosity @ 135°C (Pa·s)	3.0^{+}	1.7
Tests on RTFO		
Dynamic Shear, G*/Sin(δ), (kPa)	2.20^{+}@ 76°C	2.48
Dynamic Shear, G*/Sin(δ), (kPa)	2.20^{+}@ 82°C	1.67
Elastic Recovery, 25°C, 10 cm elongation, %		70
Tests on (RTFO+ PAV)		
Dynamic Shear, @ 25°C, G*Sin(δ), (kPa)	5000^{-}	2297
Bending Beam Creep Stiffness @ -12°C, (MPa)	300^{-}	152
Bending Beam m-value@ -12°C	0.300^{+}	0.327
Actual PG Grading		**PG76-22M**

Mix Design. Since this study mainly dealt with the frictional characteristics of different wearing course HMA mixtures, a complete mix design was not performed. Instead, a typical job mix formula (JMF) was obtained from LADOTD engineers for each mix type considered. Primarily due to the difference in aggregate absorption, mixtures with different aggregate blends (as shown in Table 4) may require slightly different asphalt contents in order to meet the design air voids specified in the selected JMFs. A Superpave gyratory compactor (SGC) was used in the laboratory to compact different lab-mixed mixtures and to determine the required asphalt contents for the 12 HMA mixtures evaluated in this study. The final JMFs of the 12 HMA mixtures are presented in Table 6-9.

Table 6
Job mix formula for Superpave II (19 mm) mix design

Superpave II (19 mm)						
Mixture Designation	**100% Limestone 19 mm**		**100% Sandstone 19 mm**		**(70+30) Limestone + Sandstone 19 mm**	
Mix Type	19.0 mm (3/4 in.) Superpave II					
Aggregate	#67 LS	39%	#67 SS	36%	#67 LS	16%
	#78 LS	26%	#78 SS	24%	#67 SS	10%
	#11 LS	27%	#11 SS	34%	#78 LS	30%
	CS	8%	CS	6%	#78 SS	10%
					#11 LS	27%
					CS	8%
Binder type	PG 76-22 M		PG76-22M		PG76-22M	
Binder Content, %	4.1		4.2		4.1	
G_{mm}	2.498		2.448		2.482	
G_{mb} at N_{max}	2.404		2.354		2.416	
% G_{mm} at $N_{ini\ \mathbf{08}}$	85.53		86.8		85.8	
% G_{mm} at $N_{max\ \mathbf{160}}$	96.23		96.2		97.3	
Design air void, %	5.3		5.2		4.2	
VMA, %	15.5		17		14.4	
VFA, %	65.7		69.7		70.6	
Metric (U. S.) Sieve	Composite Gradation Blend					
37. 5 mm (1½ in.)	100.0		100.0		100.0	
25.0 mm (1 in.)	100.0		100.0		100.0	
19.0 mm (3/4 in.)	97.0		100.0		98.8	
12. 5 mm (1/2 in.)	79.9		90.5		88.7	
9. 5 mm (3/8 in.)	58.5		71.6		67.4	
4. 75 mm (No. 4)	36.5		34.1		38.5	
2. 36 mm (No. 8)	26.0		23.6		26.9	
1. 18 mm (No. 16)	18.8		19.3		19.4	
0.600 mm (No. 30)	14.2		16.8		14.8	
0.300 mm (No. 50)	8.7		12.3		9.3	
0.150 mm (No. 100)	6.2		8.0		6.7	
0.075 mm (No. 200)	4.2		4.2		4.4	
Blend G_{sb}	2.682		2.561		2.663	
Blend G_{sa}	2.707		2.656		2.703	

Table 7
Job mix formula for Superpave II (12.5 mm) mix design

12.5 mm Superpave II						
Mixture Designation	**100% Limestone 12.5 mm**		**100% Sandstone 12.5 mm**		**(70+30) Limestone + Sandstone 12.5 mm**	
Mix Type	12.5 mm Superpave II					
Aggregate	#67 LS	12.0%	#67 SS	20.0%	#67 LS	8.4%
	#78 LS	44.0%	#78 SS	53.0%	#67 SS	3.6%
	#11 LS	35.0%	#11 SS	19.0%	#78 LS	35.7%
	CS	9.0%	CS	8.0%	#78 SS	15.3%
					#11 LS	29.0%
					CS	8.0%
Binder type	PG 76-22 M		PG76-22M		PG76-22M	
Binder Content, %	4		4.4		4.1	
G_{mm}	2.503		2.423		2.491	
G_{mb} at N_{max}	2.448		2.354		2.459	
% G_{mm} at $N_{ini\ \mathbf{08}}$	86.3		87.4		87.3	
% G_{mm} at $N_{max\ \mathbf{160}}$	97.8		97.2		98.7	
Design air void, %	3.5		4.2		2.6	
VMA, %	13.8		13.2		12.7	
VFA, %	74.7		68.6		79.2	
Metric (U. S.) Sieve	Composite Gradation Blend					
37. 5 mm (1½ in.)	100.0		100.0		100.0	
25.0 mm (1 in.)	100.0		100.0		100.0	
19.0 mm (3/4 in.)	99.1		100.0		99.4	
12. 5 mm (1/2 in.)	92.3		91.6		92.3	
9. 5 mm (3/8 in.)	71.3		73.1		70.8	
4. 75 mm (No. 4)	44.9		37.1		41.0	
2. 36 mm (No. 8)	31.7		25.7		28.4	
1. 18 mm (No. 16)	22.5		21.0		20.4	
0.600 mm (No. 30)	16.8		18.3		15.5	
0.300 mm (No. 50)	10.4		13.2		9.8	
0.150 mm (No. 100)	7.4		8.6		7.1	
0.075 mm (No. 200)	5.0		4.5		4.7	
Blend G_{sb}	2.689		2.559		2.665	
Blend G_{sa}	2.718		2.655		2.707	

Table 8
Job mix formula for SMA mix design

SMA						
Mixture Designation	**100% Limestone 12.5 mm**		**100% Sandstone 12.5 mm**		**(70+30) Limestone + Sandstone 12.5 mm**	
Mix Type	12.5 mm (1/2 in.) SMA					
Aggregate	#78 LS	75.0%	#78 SS	78.9%	#78 LS	53.9%
	#11 LS	13.0%	#11 SS	10.0%	#78 SS	23.1%
	Donna Fill	12.0%	Donna Fill	11.0%	#11 LS	12.0%
	Fibre	0.1%	Fibre	0.1%	Donna Fill	11.0%
					Fiber	0.1%
Binder type	PG 76-22 M		PG76-22M		PG76-22M	
Binder Content, %	6		5.9		5.9	
G_{mm}	2.418		2.380		2.405	
G_{mb} at N_{max} (160)	2.360		2.350		2.365	
% G_{mm} at $N_{ini\ \mathbf{09}}$	86.7		87.1		86.8	
% G_{mm} at $N_{max\ \mathbf{160}}$	97.6		98.8		98.3	
Design air void, % (75 rev.)	4.8		3.9		4.2	
VMA, %	19.9		16.7		18.6	
VFA, %	75.8		76.4		77.4	
Metric (U. S.) Sieve	Composite Gradation Blend					
37. 5 mm (1½ in.)	100.0		100.0		100.0	
25.0 mm (1 in.)	100.0		100.0		100.0	
19.0 mm (3/4 in.)	100.0		100.0		100.0	
12. 5 mm (1/2 in.)	96.8		96.6		95.4	
9. 5 mm (3/8 in.)	66.8		65.0		67.9	
4. 75 mm (No. 4)	30.6		27.4		31.2	
2. 36 mm (No. 8)	23.2		20.7		22.2	
1. 18 mm (No. 16)	19.3		17.4		17.5	
0.600 mm (No. 30)	17.2		15.8		14.3	
0.300 mm (No. 50)	12.7		11.7		8.4	
0.150 mm (No. 100)	9.4		8.7		5.5	
0.075 mm (No. 200)	6.0		5.6		3.4	
Blend G_{sb}	2.700		2.582		2.664	
Blend G_{sa}	2.726		2.665		2.708	

Table 9
Job mix formula for OGFC mix design

OGFC						
Mixture Designation	**100% Limestone 12.5 mm**		**100% Sandstone 12.5 mm**		**(70+30) Limestone + Sandstone 12.5 mm**	
Mix Type	12.5 mm (1/2 in.) OGFC					
Aggregate	#78 LS	99.7%	#78 SS	99.7%	#78 LS	69.7%
	Fiber	0.3%	Fiber	0.3%	#78 SS	29.9%
	Antistrip	0.8%	Antistrip	0.8%	Fibre	0.4%
					Antistrip	0.8%
Binder type	PG 76-22 M		PG76-22M		PG76-22M	
Binder Content, %	6.5		6.5		6.5	
G_{mb} at N_{des} (50)Corelok	2.015		1.935		1.908	
G_{mm}	2.456		2.372		2.444	
Design air void, % (50 rev.)	18.00		18.4		21.9	
VMA, %	30.9		29.2		33.6	
VFA, %	41.8		36.8		34.7	
Metric (U. S.) Sieve	Composite Gradation Blend					
37. 5 mm (1½ in.)	100.0		100.0		100.0	
25.0 mm (1 in.)	100.0		100.0		100.0	
19.0 mm (3/4 in.)	100.0		100.0		100.0	
12. 5 mm (1/2 in.)	95.7		95.7		95.7	
9. 5 mm (3/8 in.)	55.8		55.8		55.9	
4. 75 mm (No. 4)	9.6		9.6		9.7	
2. 36 mm (No. 8)	5.1		5.1		5.2	
1. 18 mm (No. 16)	3.9		3.9		4.0	
0.600 mm (No. 30)	3.7		3.7		3.8	
0.300 mm (No. 50)	3.5		3.5		3.6	
0.150 mm (No. 100)	3.5		3.5		3.6	
0.075 mm (No. 200)	2.4		2.4		2.5	
Blend G_{sb}	2.725		2.568		2.687	
Blend G_{sa}	2.744		2.661		2.729	

Laboratory Experimental Design

As discussed in the literature review, the friction resistance offered by an asphalt surface is directly related to its micro- and macro-texture. Micro-texture is largely influenced by the micro-asperities of coarse aggregates used and the aggregate's polishing resistance under traffic loading. Macro-texture is a function of aggregate size and mixture gradation and varies mainly by the mix type.

In this study, three test methods including the British pendulum and AIMS and Micro-Deval tests were chosen to evaluate the texture and degradation resistance for the selected aggregates. Since current HMA specifications do not provide any standard friction test procedures during mix design, a NCAT polishing/friction testing procedure for rapidly evaluating the frictional performance of HMA mixtures was selected. The NCAT procedure requires the preparation of 20-in. (500 mm) by 20-in. (500 mm) kneading-compacted testing slabs; therefore, in this study three replicate slabs were prepared for each of the 12 mixtures considered. Note that the AIMS test was performed at the FHWA's mobile asphalt testing laboratory and the polishing/friction slab tests were conducted at NCAT. Details of the preparation of friction testing slabs as well as laboratory test procedures are presented below.

Laboratory Preparation of Friction Testing Slabs. Loose HMA mixtures sufficient for the preparation of 36 testing slabs (12 mixtures x 3 replicates) were produced in the LTRC asphalt laboratory and later shipped to NCAT for testing slab fabrication. The following mixing and fabrication procedures were used:

- Loose mix preparation at LTRC. The graded aggregates, dried in a 140^{o}F oven for approximately 12 hours, were mixed together first without asphalt binder, and then mixed with the binder at a temperature of 350º F using a dough hook in a metal bucket. A total of 35,000 grams loose mix was prepared for one slab and packed in a 5-gallon bucket. A total of 36 buckets of loose mix were prepared and shipped to NCAT.

- Reheating and quartering. The slab preparation at NCAT began by reheating the metal buckets and quartering the mixes in a mold, as shown in Figure 8, to minimize segregation and preserve uniformity in slabs. After quartering, the mixtures were spread evenly to four quarters of the mold and covered by a separation paper (Figure 8).

Figure 8
Mix quartering and molding

- Slab compaction. As shown in Figure 9, steel plates, each 3/8 in. (10 mm) thick, 4 in. (100 mm) high and 20 in. (500 mm) long, were installed in the vertical position on top of the molded mixture until the plates covered the mix tightly. Then, a modified Hamburg rolling wheel compactor was used to compact the mixture to a testing slab with a 93 percent of G_{mm}. The resulted slabs, each approximately 2.5 in. (64 mm) thick with roughly air voids of 7 percent (Figure 9), were ready for the NCAT polishing/friction testing procedure.

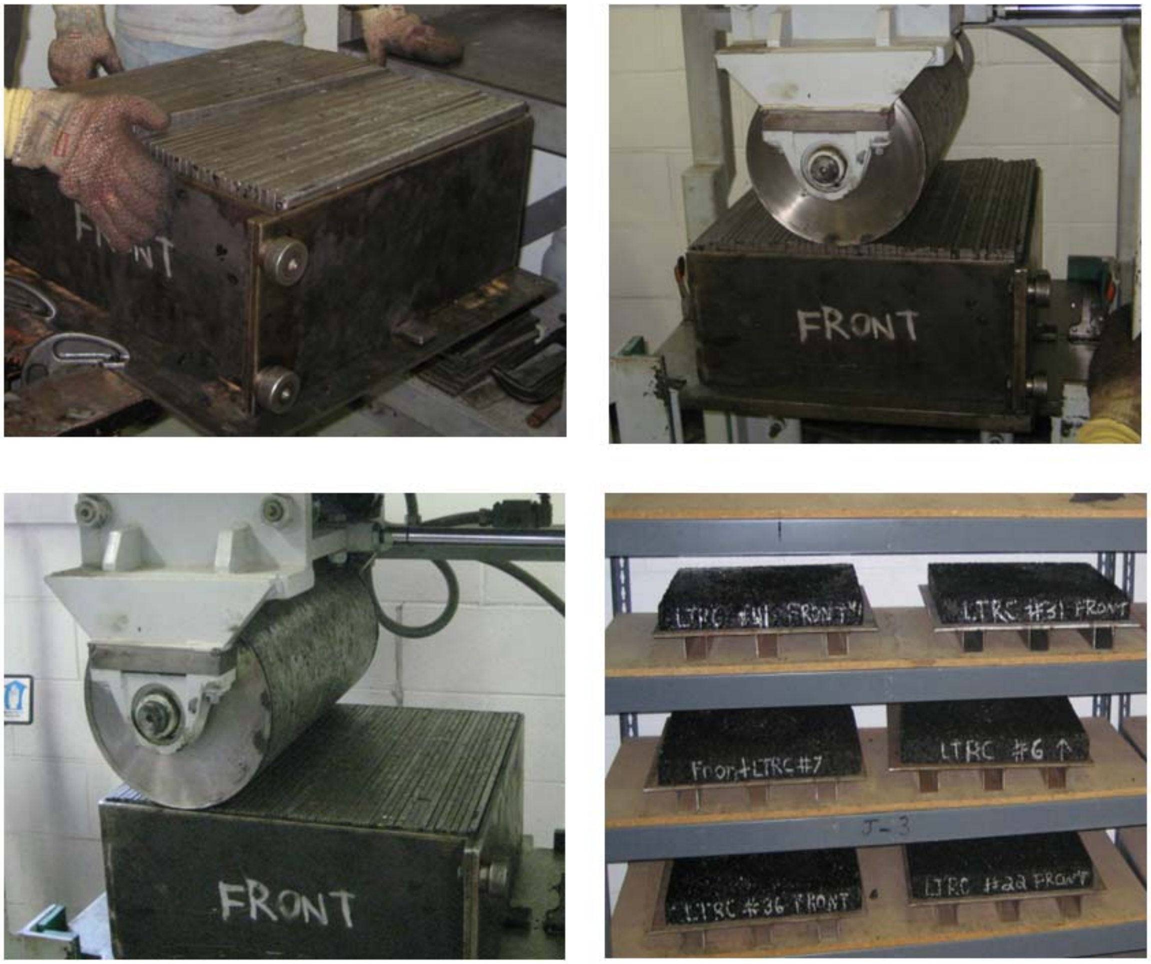

Figure 9
Slabs compaction using modified Hamburg compactor

NCAT Polishing/Friction Testing Procedure. In a recent study conducted at NCAT, a testing procedure with a laboratory accelerated polishing device was developed by Vollor and Hanson (2006). As shown in Figure 10, the accelerated polishing device is called the Three Wheel Polishing Device (TWPD) designed to simulate the traffic-polishing effects on surface friction characteristics of asphalt mixtures by using a three-abrasion-wheel assembly. The normal load during the test is 105 lb. (47.6 kg) with tire pressure of pneumatic tires maintained at 50 psi. (344 kPa) During the slab polishing, water is continuously sprayed to simulate a wet polishing in the field. It was found that such a polishing device together with a set of friction/texture measurements could be used to evaluate the frictional resistance of HMA mixtures in the laboratory that represents field measured results (Vollor and Hanson, 2006).

Figure 10
NCAT three wheel polishing device

In this study, each slab was polished under the TWPD device for the cycle periods of 2, 5, 10, 30, 50, and 100 thousand cycles, respectively. At the end of each cycle period, the polishing device was stopped and the slab was removed and dried for the evaluation of its surface texture and friction using the ASTM E 2157 CTM for slab surface texture and ASTM E 1911 DFT for slab surface friction. In addition, the post-construction friction and surface texture properties of the slabs (before TWPD polishing) were also measured. Specifically, three replicate measurements were made for each DFT test and five replicates for each CTM test during each measurement period of the slab polishing. More details regarding the testing procedure and the TWPD device may be referred to elsewhere (Vollor and Hanson, 2006).

Dynamic Friction Test. As shown in Figure 11, the DFT has three rubber sliders spring-mounted on a disk at a diameter of 350 mm. The disk is initially suspended above the pavement surface and is driven by a motor until the tangential speed of the sliders is 90 km/h. Then the motor is disengaged and the disk is lowered while applying water to the surface. The three rubber sliders contact the surface and the friction force is measured by a transducer as the disk spins down. The friction force and the speed during the spin down are saved into a file. The DFT system can be used to measure the friction at a speed over the range of 0 to 90 km/h and friction characteristics of laboratory slab samples that are at least 450 by 450 mm.

Figure 11
Dynamic friction tester

Circular Texture Meter. The CTM is a laser based profiler that measures the profile of a circle of 284 mm diameter and provides the MPD for the surface under consideration (Figure 12). The detailed test procedure is given in ASTM E2157. The profile of the circular surface is divided into eight segments of 111.5 mm. The average MPD for each segment is determined and again averaged as the MPD of the whole circular area. The CTM can measure a flat surface area, which has the area of at least 450 by 450 mm for the lab produced sample. The MPD data correlates well with the mean texture depth (MTD) and the test is regarded as repeatable, reproducible, and independent of operators.

Figure 12
Circular texture meter

Micro-Deval Test. The Micro-Deval test characterizes the aggregates' capability to resist abrasion and is standardized as AASHTO T 327-05. This test is believed to be a better indicator of abrasion than the LA Abrasion test as it evaluates the abrasion resistance in a wet condition (Rogers, 1991). In this test 1500 g of aggregate sample in the range of 4.75 mm to 16 mm is rotated in a steel container with 5000 g of steel balls in the presence of water. The aggregate is rotated 9600 to 12000 revolutions and the sample aggregate (passing #16 sieve) weight loss is obtained. The weight loss is reported as the test value. The less value of weight loss is preferred. This test method is more repeatable and reproducible than other aggregate degradation tests (Jayawickrama et al., 2006).

British Pendulum Test. This test is one of the oldest friction resistant tests for the aggregate and asphalt mix surface. The BPT was invented by Percy Sigler in the 1940s, which was later modified by UK Transport Laboratory (British pendulum, 2008). It measures the friction property of both aggregate and asphalt mix surface as specified in AASHTO T 278 and T 279 or ASTM E 303 and D3319. The test result is reported as British pendulum number (BPN) or polish stone value (PSV).

To evaluate the aggregate's PSV, coupons of aggregates are first made with resin exposing the aggregate's flat surface. These coupons are then tested with the swinging pendulum with a specific normal load and standard rubber pad. The PSV result is a strong indicator of the micro-texture of aggregate surface. In this study the BPT was performed for coarse aggregates (#67 and #78) of the selected limestone, sandstone, and limestone/sandstone blend of in 50/50 proportions.

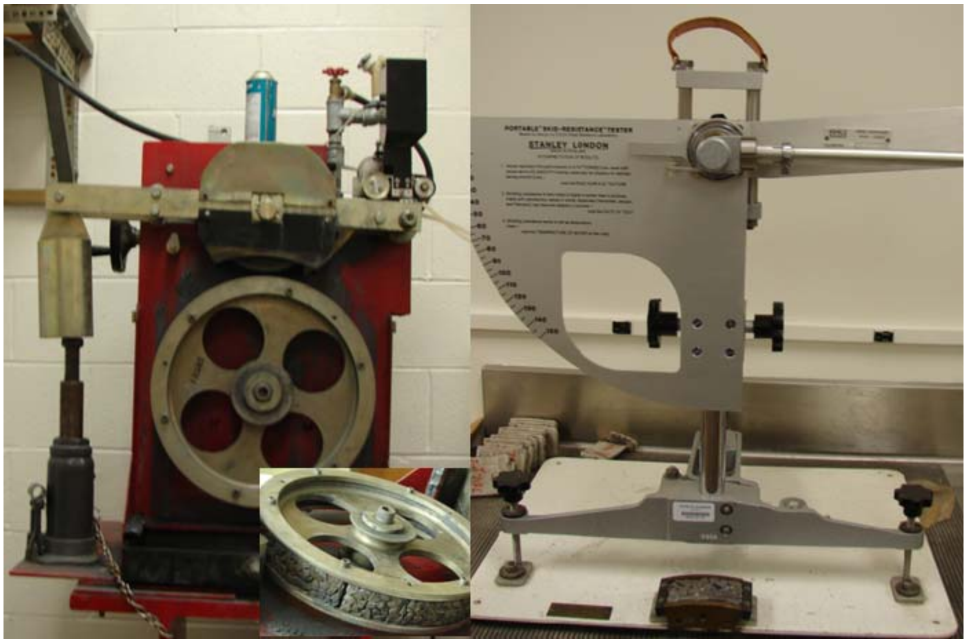

Figure 13
British pendulum

Aggregate Imaging System. The AIMS is an automated image processing system that directly evaluates the aggregate texture and shape properties (Masad et al., 2005). It can characterize the angularity, shape, and the texture of coarse aggregate as well as shape and singularity of fine aggregates. As shown in Figure 14, the AIMS includes a scanning camera, light system, a computing processor, and a tray to place coarse aggregates at 7×8 grid points and fine aggregates at 20×20 grid points. In this system, three measures of aggregate shape properties are evaluated by processing 2-D images taken by the scanning camera at a high intensity of light. The texture of the aggregate is measured as a texture index based on the wavelet theory; whereas, the angularity is measured as an angularity index calculated by gradient method and measured as the deviation of the aggregate shape from a perfect circle (Masad et al., 2005; Al-Rousan, 2004). This system also evaluates the sphericity index that measures aggregates’ closeness to a perfect sphere. In this study the AIMS was used to evaluate aggregate surface properties for the selected limestone and sandstone before and after the Micro-Deval testing at the FHWA Mobile Asphalt Laboratory.

Figure 14
Aggregate imaging system (courtesy: Al-Rousan, 2004)

Historical Friction Data Analysis

A set of LWST-measured FN (friction number) data was obtained from LADOTD's Material Laboratory and analyzed in this study. The inventory FN data were primarily measured from 1984 to 2000 including 294 different project sites. Statistical analyses were performed to determine the effects of traffic loading and aggregate and mixture types on the measured FN values. Also, critical FN values of investigatory and intervention friction levels of Louisiana asphalt pavements were determined based on the method recommended by the NCHRP Project 1-43: *Guide for Pavement Friction*. Furthermore, a set of regression models for prediction of FN based on mixture gradation and traffic loading index was developed. Because the inventory data's generally lack of control sections and high variability in terms of the aggregate type, mixture type, pavement function type, measurement interval, and data accuracy, the analysis results on the historical friction data of this study are considered as preliminary and further validation is largely needed. Therefore, all analysis results are presented in the appendix of this report.

DISCUSSION OF RESULTS

This section contains the results of the different measurements performed on the aggregates and mixtures considered in this study. It discusses the results of aggregate testing on AB13 sandstone and AA50 limestone using British Pendulum, Micro-Deval and AIMS devices. The results of the DFT and CTM measurements performed on the 12 asphalt mixtures are analyzed and further used to determine an IFI friction number, F(60). These analyses will help to develop a laboratory frictional mix design procedure that can address the effects of both micro- and macro-textures on mixture friction resistance during the mix design stage.

Aggregate Characteristics

As mentioned in the literature review, the available surface friction of an asphalt pavement comes from the right combination of pavement surface micro-texture and micro-texture for a given pavement condition. The micro-texture is defined by the surface aggregate material properties. The important aggregate properties that affect the pavement friction resistance may include mineralogy, petrography, angularity and texture, abrasion and polish resistance, and durability (Hall et al., 2009). Because this study is focused on the friction resistance of different mixture types, a complete set of measurements for evaluating different aggregate characteristics is beyond the scope. The aggregate test results obtained in this study are tabulated in Table 10. Note that several test results such as silica content, LA abrasion, Mg soundness, and absorption were obtained from the aggregate source data, not being tested under this research.

Table 10
Aggregate test results

Aggregate Type	Silica,%	LA% Wt. Loss	Mg Sound-ness, %Loss	Absorp-tion %	Polish Stone Value, PSV	MD % Wt. Loss	Texture			Angularity			Friction Rating
							Before MD	After MD	% change	Before MD	After MD	% change	
SS (AB13)	92.5	22.0	3.4	1.0	38	13.9	364	313	14.0	2821	2022	28.3	I
LS (AA50)	13.7	17.0	0.5	0.7	31	9.8	544	351	35.4	2840	2132	24.9	III
50%SS+ 50%LS	n/a	n/a	n/a	n/a	35	n/a	n/a	n/a	n/a	n/a	n/a	n/a	II

Note: SS – sandstone; LS – limestone; LA – Los Angles Abrasion; MD – Micro-Deval; Mg – Magnesium; Wt. – Weight; n/a – not available.

The British Pendulum test results indicate that the PSV value for the selected sandstone and limestone is 38 and 31, respectively. According to the LADOTD friction rating criteria, the two aggregates fall into the friction ratings of I and III, respectively. This confirms the source friction ratings for the two aggregates. The BP test results also indicate that, when testing a mixed aggregate blend of these two aggregates on a 50/50 proportion basis, the resultant PSV for the blend is 35. This value is believed to be the average of two PSVs for the sandstone and limestone. Similar results were also reported by other studies (Masad et al., 2009 and Ashby, 1980). The BP test results generally confirmed that the selected sandstone aggregate has a better polishing resistance (or better micro-texture) than the selected limestone. When mixing the two aggregates into a mixture design, an intermediate aggregate micro-texture can be expected to obtain and will improve the mixture friction resistance as compared to the limestone-only mixtures. Since this study chose to use 30 percent sandstone and 70 percent limestone in mix design, a PSV of 33 was determined for such aggregate blends by the linear interpolation of the test results.

The NCHRP 1-43 study recommends a set of typical range of aggregate test values for good friction performance (Hall et al., 2009). The related typical range values include: Micro-Deval, % loss ≤ 17 to 20; LA Abrasion, % loss ≤ 35 to 45; Magnesium Soundness, % loss ≤ 10 to 20. The Micro-Deval test for coarse aggregates has been reported to be a good indicator of the potential for aggregate breakdown and wear resistance (Kandhal and Parker, 1998). As shown in Table 10, the selected limestone has a lower weight loss in the Micro-Deval than the sandstone, but both aggregates can meet the criteria for good friction performance as recommended in the NCHRP 1-43 study. Similarly, both the LA Abrasion and Magnesium Soundness test results also indicate that the limestone has a slightly lower weight loss than the sandstone, and both properties meet the good friction performance criteria.

In addition, the AIMS test results show that the limestone (AA50) had higher texture values before and after Micro-Deval than the sandstone (AB13). However, the limestone experienced a much shaper drop in the texture (the percentage change after the Micro-Deval) than the sandstone evaluated. Furthermore, the AIMS test determined similar angularity values for the two aggregates as shown in Table 10. A recent study conducted by Masad et. al (2009) found that the change in texture before and after Micro-Deval and the texture after Micro-Deval both are significant factors for mixture friction resistance. The AIMS test results of this study somewhat only support one of the significant friction factors: the change in texture before and after Micro-Deval. A higher texture value after Micro-Deval for the limestone aggregate evaluated seems to be opposite of its PSV results.

In summary, test values of the PSV and AIMS's change in texture correctly suggested that the sandstone (AB13) have better polishing resistance than the limestone (AA50), while other aggregate tests only showed that the limestone may have a slightly better or similar abrasion and wear resistance as compared to the AB13 sandstone. The better polishing resistance of AB13 sandstone will be discussed further in the following sections.

Results of Polishing/Friction Slab Tests of HMA Mixtures

Dynamic Friction Tester Measurements

As previously stated, surface frictional properties of each lab-fabricated HMA testing slab were measured by DFT and CTM at different polishing cycles. DFT measures surface friction resistance properties of polished slabs under four friction speeds (i.e., 20, 40, 60, and 80 km/h). Figures 15 through 18 present the average results of DF20, DF40, DF60, and DF80, respectively, measured at specified polishing cycles for all HMA mixtures considered in this study.

As can be seen in those figures, DFT results generally indicate that all HMA mixtures with 100 percent sandstone (AB13) performed significantly better in friction resistance than the corresponding mixtures with either 100 percent limestone (AA50) or the combination aggregate blends of limestone and sandstone under all polishing cycles. Such results are expected since the AB13 sandstone showed a much higher PSV value in the BPT test (Table 10) than the AA50 limestone, implying a better frictional resistance of AB13 due to its rougher micro-asperity surface. It is also evident from Figures 15-17 that the friction resistance of mixtures decreases as the polishing cycle increases. However, Figure 18 indicates that the DF80 results (DFT measured friction at 80 km/h) could not tell the difference in friction among mixtures with different aggregate blends. Also, the DF80 results did not show a decreasing trend with increasing polishing cycles. This implies that the micro-texture difference of various HMA mixtures cannot be captured by the DFT measurements at high speed of 80 km/h, possibly due to smaller mean values with large testing variability.

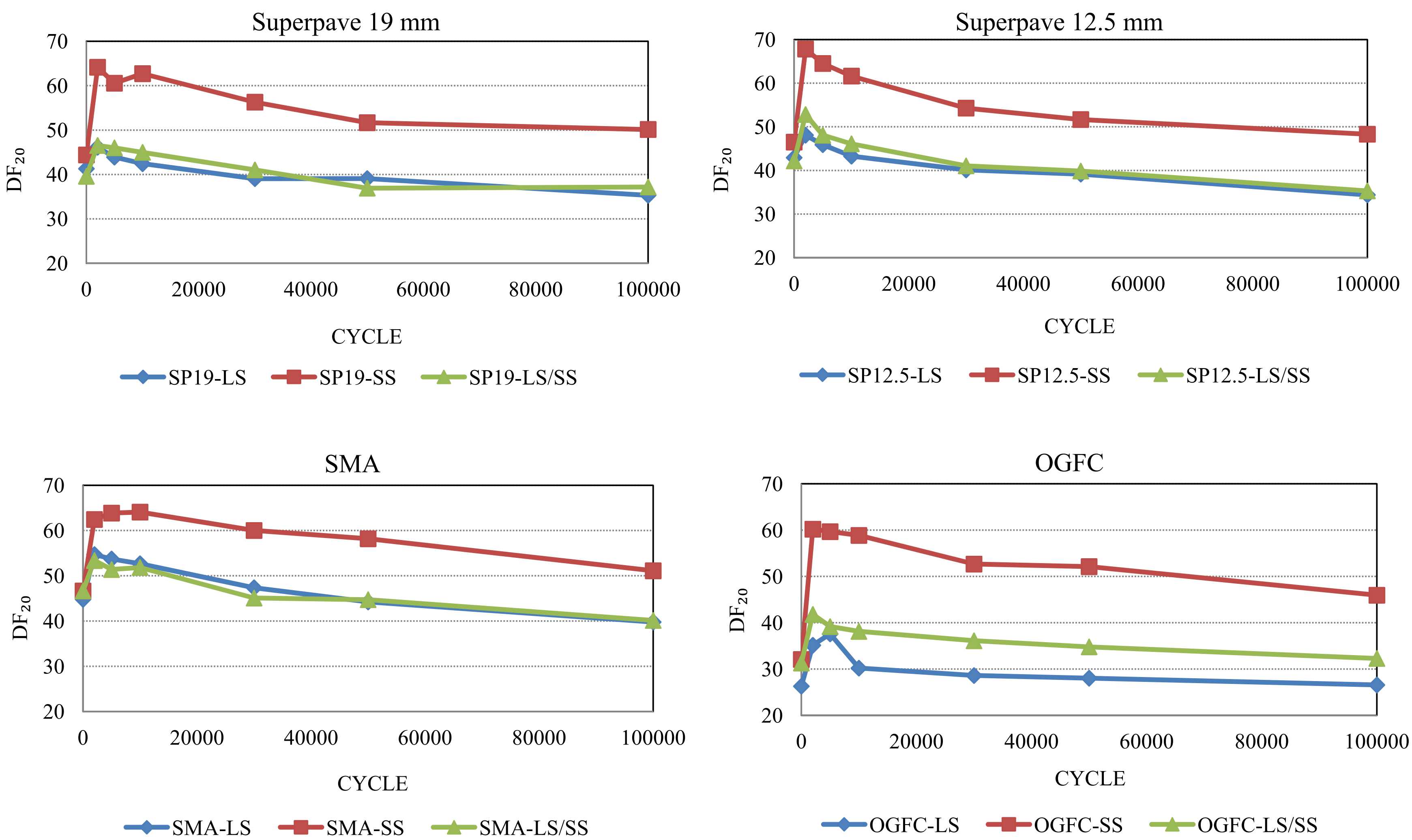

Figure 15
DF_{20} values by polishing cycles for different mix and aggregate types

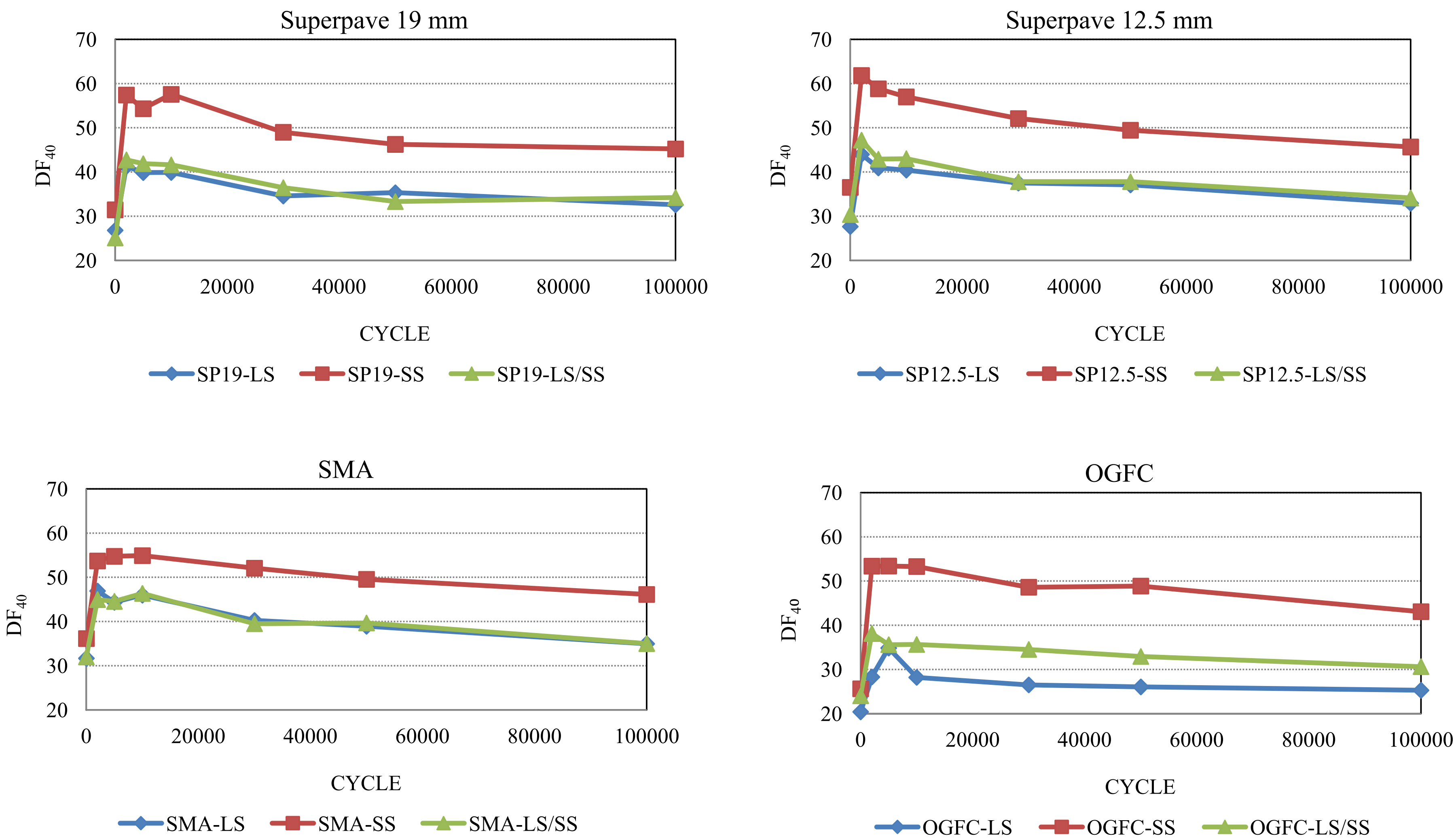

Figure 16
DF_{40} values by polishing cycles for different mix and aggregate types

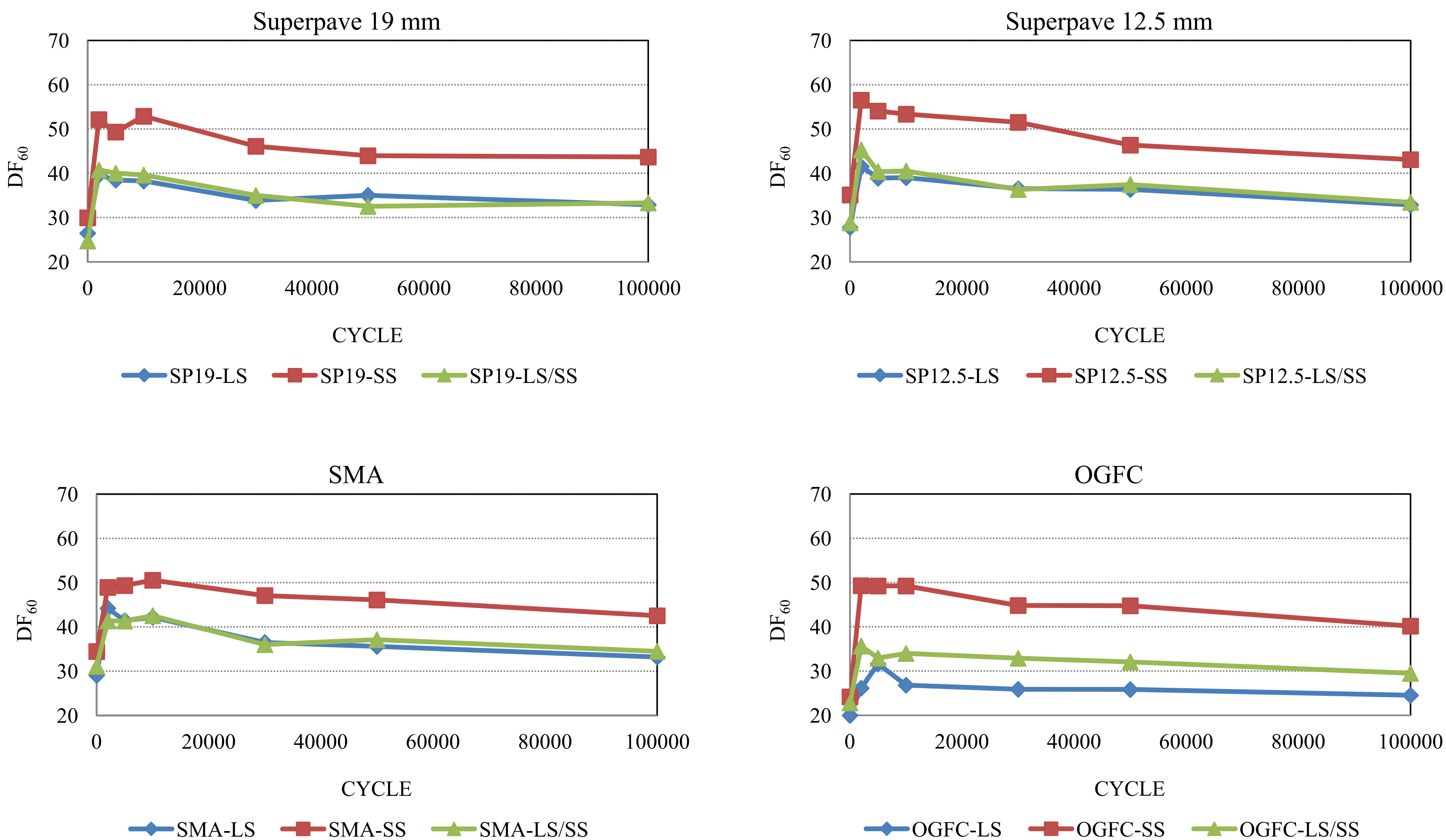

Figure 17
DF_{60} values by polishing cycles for different mix and aggregate types

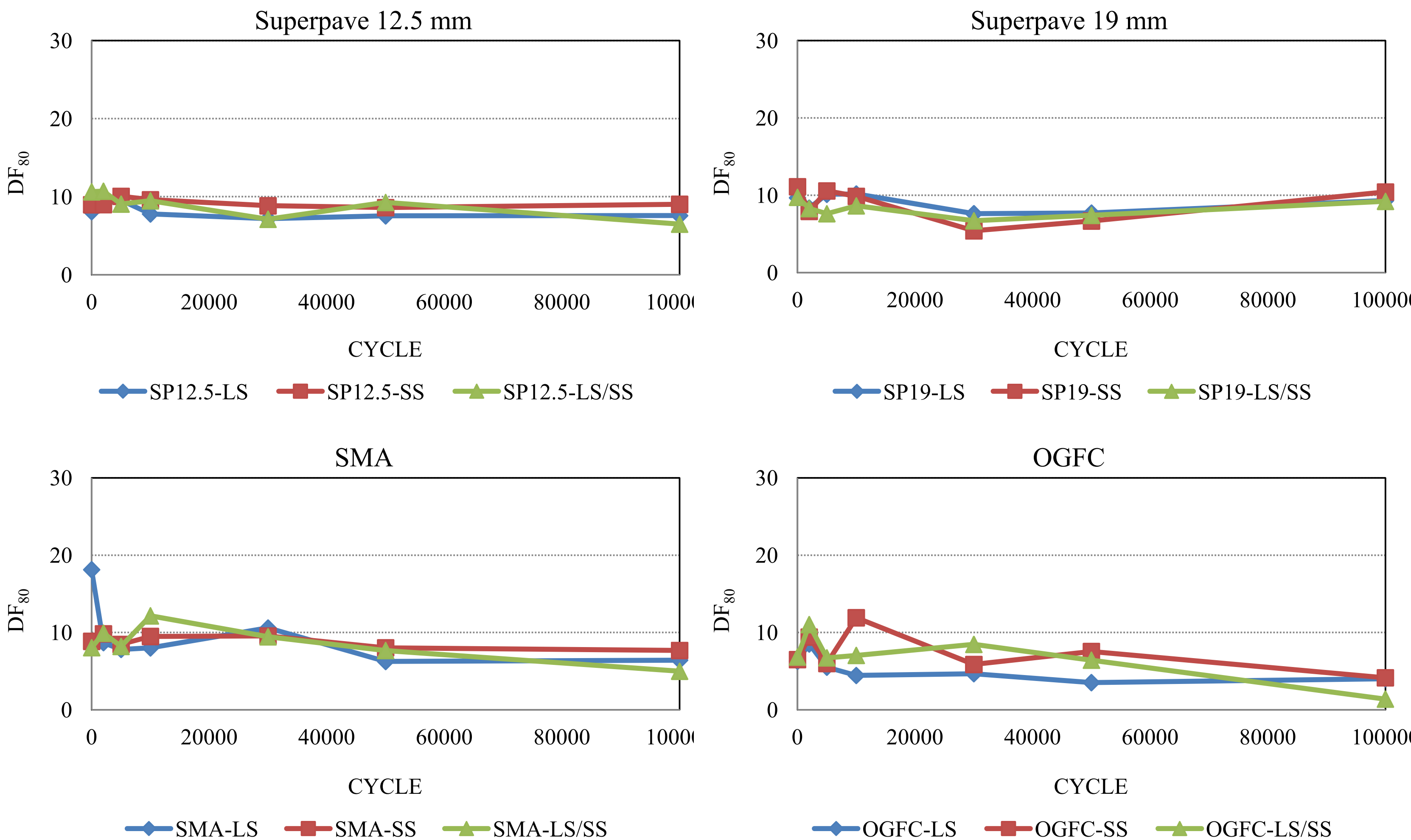

Figure 18
DF_{80} values by polishing cycles for different mix and aggregate types

Furthermore, the DFT results showed an initial increase in friction resistance measurements and the maximum friction values occurred at approximately 2,000-5,000 polishing cycles. After reaching this peak DFT point, the friction resistance of a slab surface started to decrease as the polishing cycle increased. This is due to the development of an early surface roughness or textures of the coated aggregate particles (e.g., remove the excess binder from the surface and expose the aggregate). It was found that the average ratio between of the maximum DFT values and the initial DFT values (without polishing) for all mixture slab tested in this study is 1.45.

It can be also found in Figure 15-17 that the OGFC mix type showed generally higher DFT measured friction numbers for mixtures with the combination aggregate blends of limestone and sandstone than those with limestone only. This result is promising since it can be used to prove a hypothesis commonly used in mix design that blending of low and high friction aggregates together could produce an asphalt mixture with a satisfactory field friction resistance. Although other mix types considered seemed not able to differentiate the friction difference between mixtures with only limestone and with the combination aggregate blends, it is believed that could be related to the low percentage of sandstone used in the combination blends. Only 30 percent of coarse sandstone aggregates used in the combination aggregate blends appears to be too low to improve the surface friction resistance of those HMA mixtures (expect the OGFC mix type) with high percentage of low friction resistant limestone coarse aggregates. In current state of practice, LADOTD typically requires 50/50 of low/high friction resistant coarse aggregate ratio for a friction-resistant mix design.

The Tukey pair-wise comparison analysis was performed to study the sensitivity of the DFT measured friction results due to the changes in mixture type, aggregate type, friction speed, and polishing cycle. The Statistical Analysis Software (SAS) software program was used. The Tukey test basically performs a pair-wise comparison of the equality of means for each variable considered in the sensitivity analysis. When a p-value (the significance level parameter) is less than 0.05, it indicates that the difference between two compared mean values is significant at a 95 percent of confidence.

Table 11 provides the p-values for the comparison of measured DFT results among different mixture types. It shows that at a 95 percent level of confidence there is no statistical difference in mean DFT measurements for the Superpave 12.5-mm and Superpave 19-mm mix types. On the other hand, the mean differences of the DFT values among other mix types are all significant at a

95 percent level of confidence. Such results indicate that the DFT test is capable of differentiating the friction difference for various mix types except the two Superpave mix types. The JMFs for the two Superpave mixes indicate that both mixes had a coarse-graded gradation, implying their macro-textures should be not much different.

Table 11
Comparison significance level (p-values) of DFT values

Mix Type	Superpave 19 mm	Superpave 12.5 mm	SMA	OGFC
Superpave 19 mm		0.89	0.00	0.00
Superpave 12.5 mm	0.89		0.00	0.00
SMA	0.00	0.00		0.00
OGFC	0.00	0.00	0.00	

Note: Non-Significant P-values are highlighted.

The sensitivity analyses of DFT measurements due to the changes in aggregate type are presented in Tables 12 and 13. The mean DF20 results at 5,000 and 100,000 polishing cycles were used in the Tukey pair-wise comparison analysis in which DF20 at 5,000 and 100,000 cycles were representative of the initial and terminal friction numbers, respectively. In general, the comparison analysis indicates that, at a 95 percent level of confidence, DFT can tell the differences of frictional properties between the sandstone and limestone mixes, and between sandstone and limestone/sandstone combination mixes. However, DFT cannot differentiate frictional differences between limestone mixes and limestone/sandstone combination mixes at a 95 percent level of confidence. Moreover, DFT did show somewhat significant differences of frictional properties between the limestone OGFC mixes and the limestone/sandstone combination OGFC mixes at a 93 percent level of confidence.

Table 12
Comparison significance level (p-values) of DFT values of different aggregate type at polish cycle 5000 and speed 20 km/hr.

Superpave 19 mm				Superpave 12.5 mm			
Aggregate Type	**Limestone**	**Sandstone**	**Limestone + Sandstone**	**Aggregate Type**	**Limestone**	**Sandstone**	**Limestone + Sandstone**
Limestone		0.00	0.99	**Limestone**		0.00	0.99
Sandstone	0.00		0.00	**Sandstone**	0.00		0.00
Limestone + Sandstone	0.99	0.00		**Limestone + Sandstone**	0.99	0.00	
SMA				**OGFC**			
Aggregate Type	Limestone	Sandstone	Limestone + Sandstone	**Aggregate Type**	Limestone	Sandstone	Limestone + Sandstone
Limestone		0.01	0.99	**Limestone**		0.00	0.99
Sandstone	0.01		0.00	**Sandstone**	0.00		0.00
Limestone + Sandstone	0.99	0.00		**Limestone + Sandstone**	0.99	0.00	

Note: Non-Significant P-values are highlighted.

Table 13
Comparison significance level (p-values) of DFT values of different aggregate types at polish cycle 100,000 and speed 20 km/hr.

Superpave 19 mm				Superpave 12.5 mm			
Aggregate Type	**Limestone**	**Sandstone**	**Limestone + Sandstone**	**Aggregate Type**	**Limestone**	**Sandstone**	**Limestone + Sandstone**
Limestone		0.00	0.99	**Limestone**		0.00	1.00
Sandstone	0.00		0.00	**Sandstone**	0.00		0.00
Limestone + Sandstone	0.99	0.00		**Limestone + Sandstone**	1.00	0.00	
SMA				**OGFC**			
Aggregate Type	Limestone	Sandstone	Limestone + Sandstone	**Aggregate Type**	Limestone	Sandstone	Limestone + Sandstone
Limestone		0.00	1.00	**Limestone**		0.00	0.07
Sandstone	0.00		0.00	**Sandstone**	0.00		0.00
Limestone + Sandstone	1.00	0.00		**Limestone + Sandstone**	0.07	0.00	

Note: Non-Significant p-values are highlighted.

When the DFT measured friction numbers are plotted at different slip friction speeds, an actual friction curve for a braking process from free rolling to a locked-wheel state would be expected to develop. However, the DFT measurements obtained in this study were not able to differentiate from each other as indicated in the following statistical analysis. Also, to develop a friction curve at different speed is beyond the scope of this study.

The sensitivity of DFT measured coefficients of friction to different slip friction speeds was analyzed based on the Tukey comparison procedure at two polishing cycles (initial and terminal) for each mixture considered. Table 14 presents the compassion p-value results.

For the limestone mixes, the difference between DF20 and DF60 is not significant for most of the mixes at polish cycle of 5000, which seems to indicate that the limestone mixes are not sensitive to test speed. On the other hand, most sandstone mixes except OGFC show difference between DF20 and DF60. The limestone/sandstone blend also show the difference between DF20 and DF60 for most of the mixes except OGFC mixes. Presumably due to having very high asphalt contents, the OGFC mixes generally tend to not very sensitive to different friction test speeds during the beginning 5000 polishing cycles. For a polish cycle of 100,000, most of the mixes except Superpave 19 mm and OGFC sandstone do not show test speed influence. At 100,000 cycles, the mix surface is highly polished, which could be the reason for the absence of influence of test speed on DFT measurements.

Table 14
Significance level of the DFT values compared for speed effect at 5000 and 100,000 cycles

Mix Type	Aggregate Type	Cycle 5000				Cycle 100,000			
SP-19	**Limestone**	**Speed (km/hr)**	**20**	**40**	**60**	**Speed (km/hr)**	**20**	**40**	**60**
		20		0.53	0.13	**20**		0.87	0.93
		40	0.53		0.99	**40**	0.87		1.0
		60	0.13	0.99		**60**	0.93	1.0	
	Sandstone	**20**		0.45	0.00	**20**		0.11	0.01
		40	0.45		0.21	**40**	0.11		0.99
		60	0.00	0.21		**60**	0.01	0.99	
	Limestone + Sandstone	**20**		0.49	0.06	**20**		0.79	0.41
		40	0.49		0.99	**40**	0.79		1.0
		60	0.06	0.99		**60**	0.41	1.0	
SP-12.5	**Limestone**	**20**		0.01	0.00	**20**		0.99	0.99
		40	0.01		0.89	**40**	0.99		1.0
		60	0.00	0.89		**60**	0.99	1.0	
	Sandstone	**20**		0.00	0.00	**20**		0.91	0.08
		40	0.00		0.02	**40**	0.91		0.91
		60	0.00	0.02		**60**	0.08	0.91	0.00
	Limestone + Sandstone	**20**		0.01	0.00	**20**		1.0	0.99
		40	0.01		0.66	**40**	1.0		1.0
		60	0.00	0.66		**60**	0.09	1.0	
SMA	**Limestone**	**20**		0.07	0.00	**20**		0.86	0.45
		40	0.07		0.99	**40**	0.86		1.0
		60	0.00	0.99		**60**	0.45	1.0	
	Sandstone	**20**		0.09	0.00	**20**		0.83	0.11
		40	0.09		0.76	**40**	0.83		0.98
		60	0.00	0.76		**60**	0.11	0.98	
	Limestone + Sandstone	**20**		0.42	0.04	**20**		0.80	0.68
		40	0.42		0.99	**40**	0.80		1.0
		60	0.04	0.99		**60**	0.68	1.0	
OGFC	**Limestone**	**20**		0.99	0.90	**20**		0.99	0.98
		40	0.99		0.99	**40**	0.99		1.0
		60	0.90	0.99		**60**	0.98	1.0	
	Sandstone	**20**		0.86	0.19	**20**		0.84	0.04
		40	0.86		0.99	**40**	0.84		0.83
		60	0.19	0.99		**60**	0.04	0.83	
	Limestone + Sandstone	**20**		0.99	0.87	**20**		0.99	0.87
		40	0.99		1.0	**40**	0.99		1.0
		60	0.87	1.0		**60**	0.87	1.0	

CTM Results

Figure 19 presents the CTM results in terms of MPD (mean profile depth) values plotted against the polishing cycle. The CTM results clearly show the distinction of MPD values according to mix type; that is, the OGFC mix has the maximum MPD followed by SMA and Superpave mixes, respectively. The OGFC mix has higher air voids and larger pores in the surface, so having high MPD value confirms to the mix design. Among the OGFC mix, the limestone/sandstone blend shows the highest MPD value; whereas, the sandstone-only blend has the highest MPD value for SMA. Such difference reflects the variation in the mix design or experiment errors. The two Superpave mixes are clustered together and do not show clear difference in MPD results. The MPD values for different mixes are about same throughout the polishing cycle after the 2000 polish cycle, which indicates that the MPD values are un-affected by the polishing. The initial change in MPD (Figure 19) could be related to aggregate abrasion during polishing or experiment errors (Masad et al., 2009).

The MPD value represents the macro-texture of the asphalt surface, which is more dependent on the mix design than the aggregate type. Figure 20 shows the mean MPD value with one standard deviation for the mix type at 5000 and 100,000 polish cycles. The OGFC mix shows the highest MPD at both cycles, followed by SMA and Superpave mixes. Figure 20 also indicates that the polishing has less impact on MPD values.

Results of the statistical comparison analysis further confirmed that, based on the p-values shown in Table 15, the mean MPD values at 5000 polishing cycles are significantly different for different mixes including Superpave, SMA, and OGFC. However, the mean MPD values for the two Superpave mixes are not significantly different from each other, implying that the macro-texture (i.e., MPD) of a surface HMA mixture is more dependent on the aggregate gradation type (which is associated with mix type) and less dependent on the aggregate size.

The effects of different aggregate types on the measured MPD values are presented in Table 16. The comparison results indicate that MPD values show no dependence on aggregate type, since all comparison p-values are significantly greater than 0.05 as shown in Table 16.

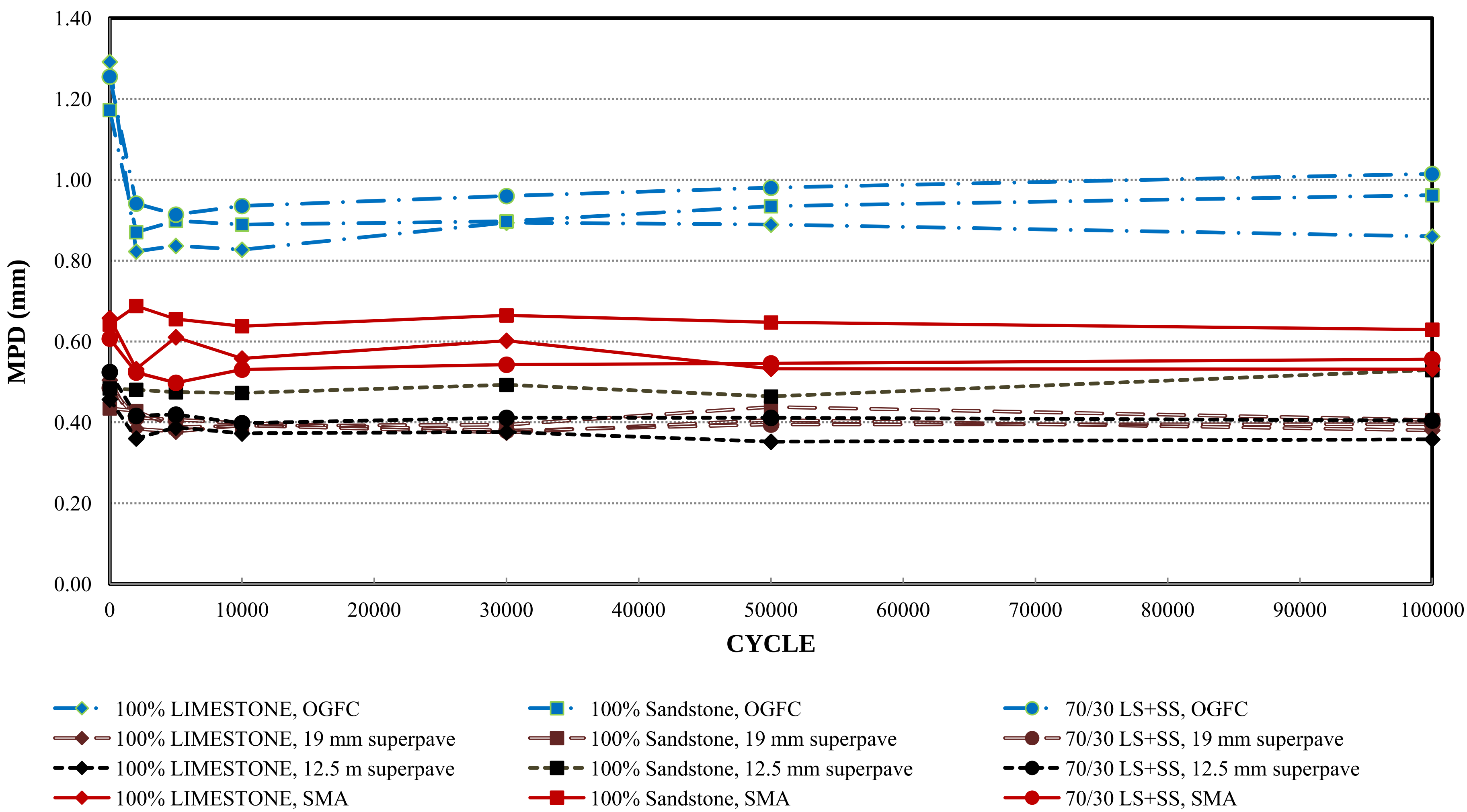

Figure 19
Average MPD by mix and aggregate type

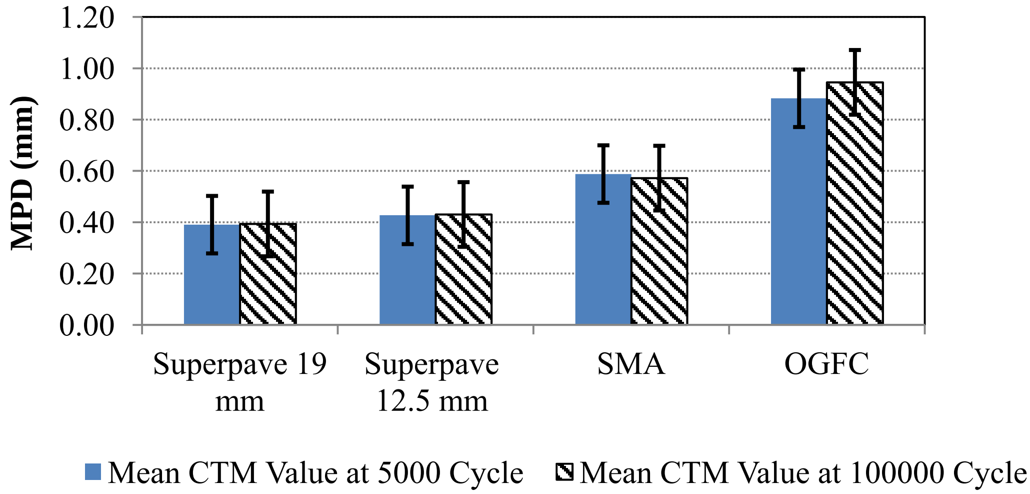

Figure 20
Mean CTM values by mix type

Table 15
Comparison significance level (p-values) of MPD values of different mixes at polish cycle 5000

Mix Type	Superpave 19 mm	Superpave 12.5 mm	SMA	OGFC
Superpave 19 mm		0.82	0.00	0.00
Superpave 12.5 mm	0.82		0.00	0.00
SMA	0.00	0.00		0.00
OGFC	0.00	0.00	0.00	

*Non-Significant p-values are highlighted.

Table 16
Comparison significance level (p-values) of MPD values of different aggregate type at polish cycle 5000

Aggregate Type	Limestone	Sandstone	Limestone + Sandstone
Limestone		0.35	0.98
Sandstone	0.35		0.44
Limestone + Sandstone	0.98	0.44	

*Non-significant p-values are highlighted.

Polishing Effect on Friction Resistance

The friction resistance of the HMA mixture is a function of its polishing resistance. As shown in previous DFT measurement results, as the polishing cycle increases, the DFT value decreases. In this study, the following nonlinear equation, proposed by Mahmoud et al., was used to fit the DF20 measurement results with polishing cycles (Mahmoud et al., 2005).

$$DF_{20} = a + b \times \exp(-c \times 1000cycle) \tag{14}$$

where, a, b, and c are regression constants.

As previously discussed, the DF20 values can differentiate the difference in friction resistance of HMA mixtures designed with various aggregate and mix types. In addition, DFT value measured at 20 km/h is more representative of the friction resistance due to the effect of micro-texture of an HMA mixture. The result of this study also indicated that the mean DFT values at different speeds were not significantly different from each other for the HMA mixtures tested in this study.

Table 17
Regression parameters values of the model for DF_{20}

Mix Type	a	b	c	R^2
SP19-LS	0.35	0.12	0.04	0.97
SP19-SS	0.49	0.16	0.03	0.92
SP19-LS+SS	0.36	0.12	0.04	0.97
SP12.5-LS	0.34	0.14	0.02	0.96
SP12.5-SS	0.49	0.20	0.04	0.99
SP12.5-LS+SS	0.34	0.16	0.02	0.98
SMA-LS	0.37	0.19	0.02	0.99
SMA-SS	0.42	0.26	0.01	0.97
SMA-LS+SS	0.39	0.15	0.01	0.93
OGFC-LS	0.25	0.08	0.02	0.98
OGFC-SS	0.42	0.19	0.01	0.96
OGFC-LS+SS	0.30	0.09	0.02	0.99

The parameters a, b, and c in equation (14) denote the changes in frictional properties of an HMA mixture during polishing. The parameter “a” represents the terminal friction value and “a+b” is the initial friction value; whereas, the parameter “c” is called the polishing rate (Mahmoud et al., 2005). In general, a high “a” value and “a+b” value and a low “c” value are expected for a friction resistant surface mixture, which can not only provide a high friction number to the pavement but also maintain the high frictional property to a terminal pavement stage (e.g., before overlay) with a low polishing rate.

Figure 21 shows the measured DF_{20} values and the fitted curves for the 12 mixtures considered. Table 17 presents the corresponding magnitudes of each regression constant obtained by the curve fitting. In general, all regression analyses had high R-square values indicating equation (14) fits well with the DF_{20} measurement results. As can be seen in Table 17, the "a" values of the sandstone mixtures are generally higher that those of the limestone and combination aggregate mixtures. This is consistent with the direct analysis of DFT results, indicating all sandstone mixtures had better terminal friction resistance than the corresponding mixtures with other aggregate blends considered. However, a mixed-bag result was obtained when comparing terminal (the "a" value) and initial (the "a+b" value) friction values among different mixture type considered. The terminal and initial values of OGFC mixtures seem to be smaller than other mix types, Table 17.

On the other hand, the polishing rate (the "c" value) shows the clear differences among various mixtures. As seen in Figure 22, the polishing rates of SMA and OGFC mixtures are all significantly smaller than those of Superpave mixtures, indicating more polishing resistant properties. As explained in the literature review section, the macro-texture represents the surface roughness due to aggregate texture, shape, and orientation of the aggregate contacts, and it may not change significantly with the polishing. Since both SMA and OGFC mix types had higher macro-texture values (presented as MPD) than Superpave mixtures, the friction resistance of those mixtures should be represented by both the micro- and macro-texture effects.

For the SMA and OGFC mixes, the rate of polishing is small compared to Superpave mixes, which indicates that these mixes play a role in retaining the micro-texture property of the aggregate.

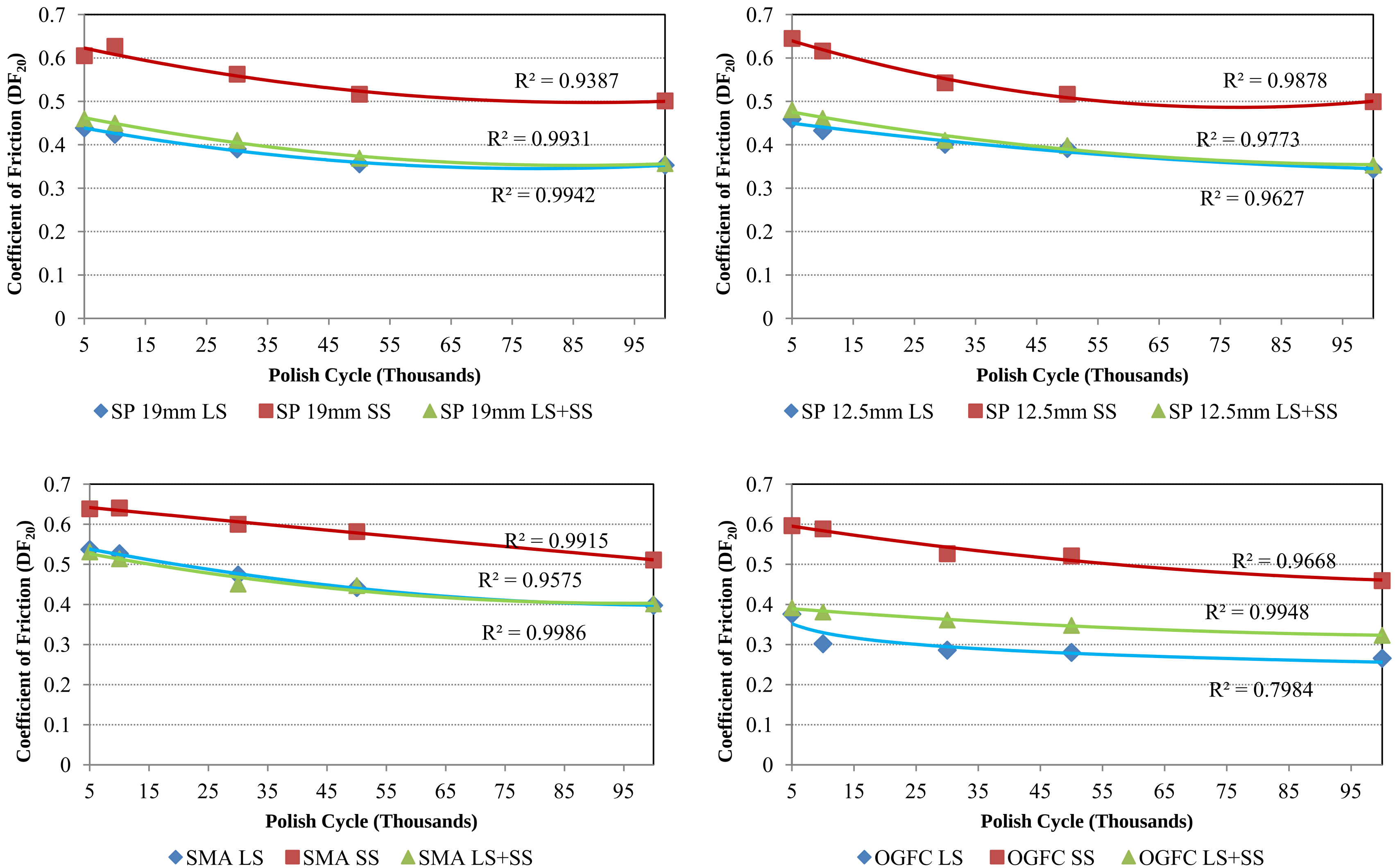

Figure 21
Fitted line for DF_{20} values by polishing cycles for different mix and aggregate types

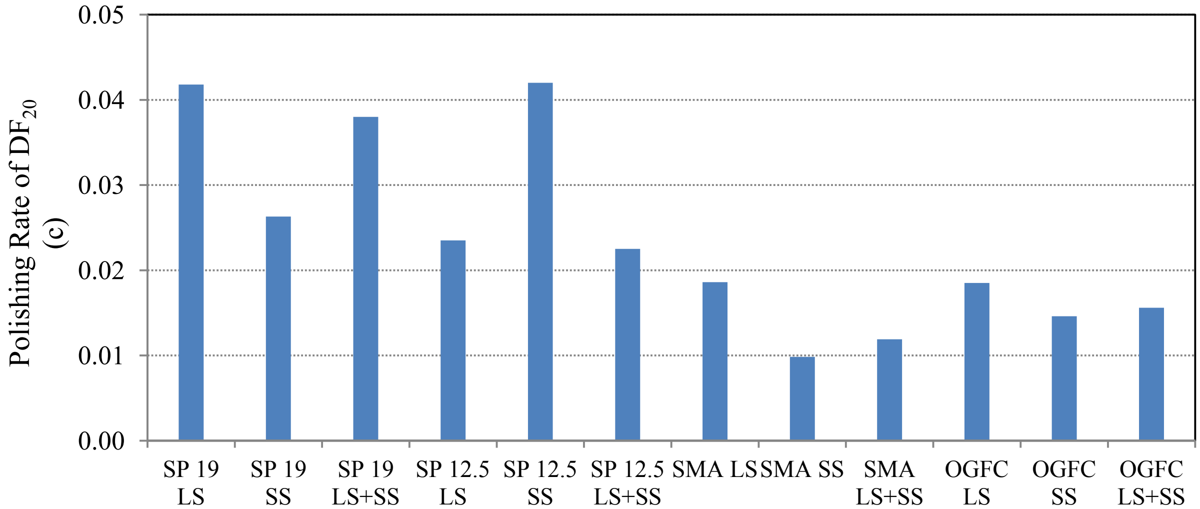

Figure 22
Polishing rate of DF_{20} for different aggregate and mix type

International Friction Index

The IFI friction index, F(60) values, were calculated using equations (11-12) for each HMA mixture considered in the laboratory polishing/friction tests of this study. As can be seen in equation (11), F(60) is a function of both DF20 and MPD, indicating F(60) can be a better representation of mixture surface friction resistance. Figure 23 shows the calculated F(60) results for different mix and aggregate types versus the polishing cycles. As shown in Figure 23, F(60) decreases with an increase in polishing cycle. The sandstone OGFC and SMA have the highest F(60) values followed by the sandstone Superpave mixes, limestone SMA, and LS/SS blend for OGFC and SMA. The limestone Superpave mixes have the lowest F(60) values. Although the sandstone Superpave mixes had high DF20 values, their F(60) values are comparatively less than the sandstone OGFC and SMA mixes, which indicates that the F(60) addresses both micro- and macro-texture and DF20 alone cannot address the surface friction resistance of a pavement.

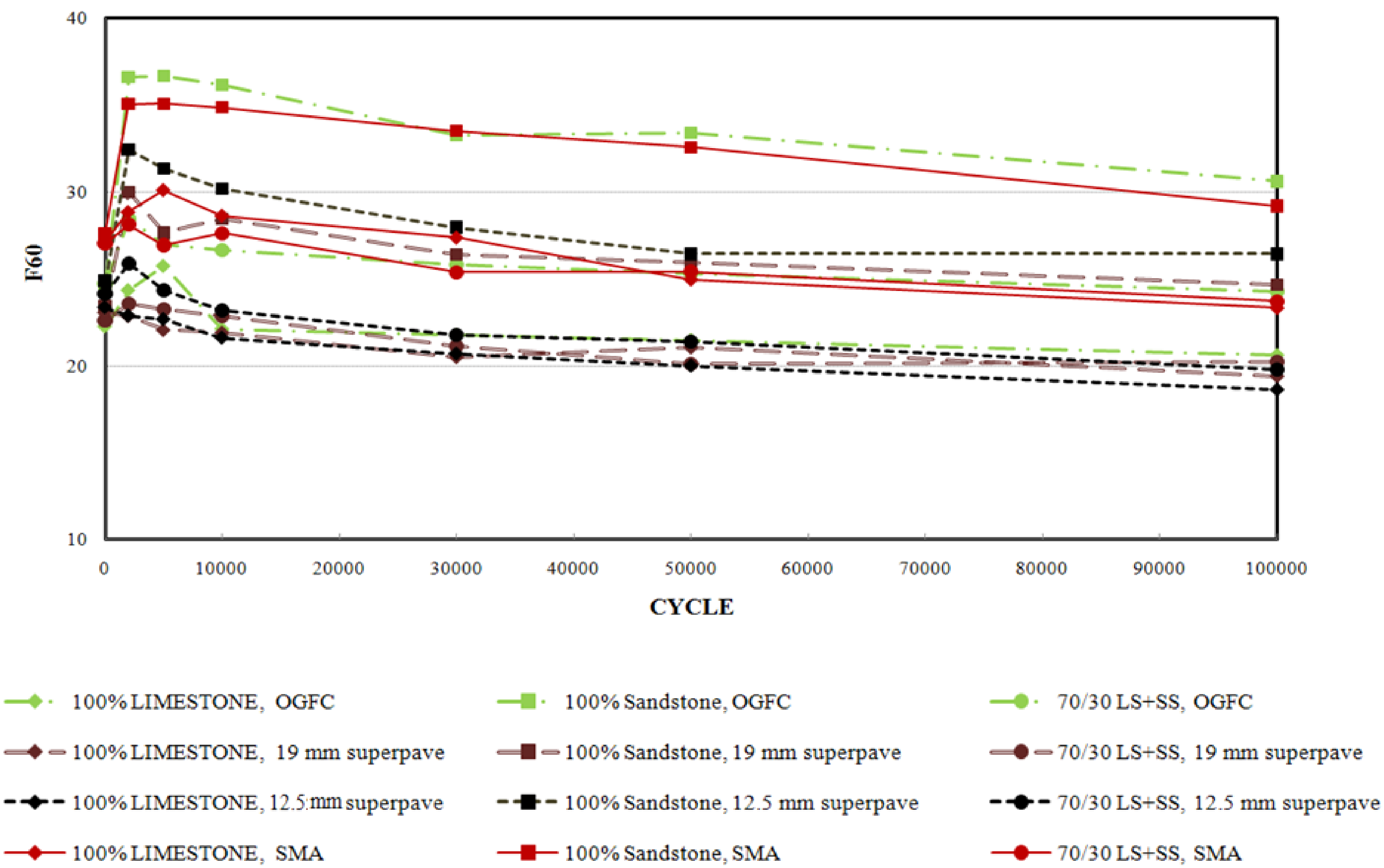

Figure 23
F(60) for different mix and aggregate type by polishing cycle

Figure 24 presents the mean F(60) values for different mixes including all aggregate types for initial and terminal (5000 and 100,000) polish cycles. Figure 24 indicates that the higher the macro-texture (MPD), the greater the coefficient of friction for a mixture, when the same aggregate blends are considered.

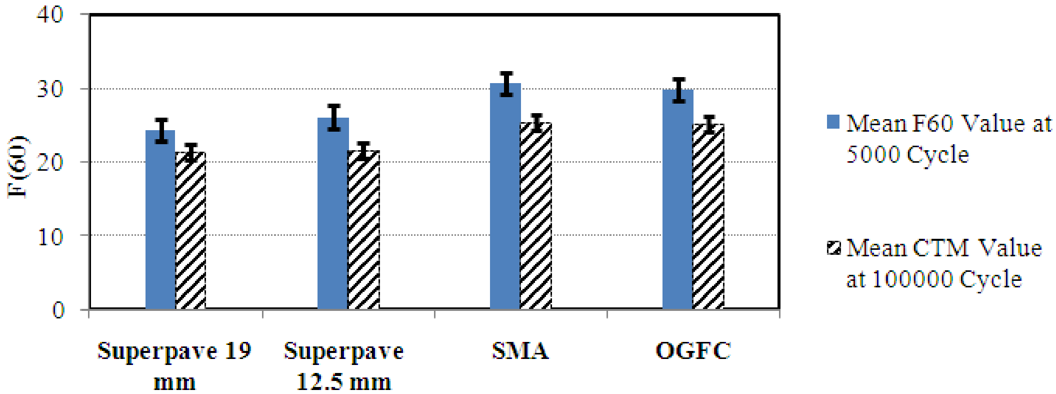

Figure 24
Mean F60 values by mix type at polish cycle 5000 and 100,000

A statistical ranking was performed based on the terminal F(60) values and the results are shown in Table 18. The mixes within the same category does not show any significant difference in their F(60) values.

Table 18
Statistical ranking of mix designs by F(60)

No.	Mix Designation	Ranking
1	OGFC-SS	A
2	SMA-SS	A
3	SP12.5-SS	B
4	SP19-SS	B
5	OGFC-LS+SS	B
6	SMA-LS+SS	B
7	SMA-LS	B
8	OGFC-LS	C
9	SP19-LS+SS	C
10	SP12.5-LS+SS	C
11	SP19-LS	C
12	SP12.5-LS	C

As discussed earlier, the potential combination of aggregate source and mix type can be evaluated using the IFI friction number, F(60). Using DF20 as a surrogate for micro-texture and the MPD for macro-texture, a relationship between F(60) and DF20 can be constructed based on different MPD levels. Figure 25 presents such a relationship obtained from test results of this study. As shown in Figure 25, to achieve the design level F(60) of 30, different pairs of DF20 and MPD are needed. This basically provides a guideline for evaluating the levels of micro-texture (DF20) and macro-texture (MPD) needed to achieve the design friction level established for a project, and potentially selecting low friction resistant aggregates in a mix design. As indicated by the NCHRP 1-43 study, a similar combination of PSV (micro-texture) and MPD (macro-texture) could be estimated and used (Hall et al., 2009).

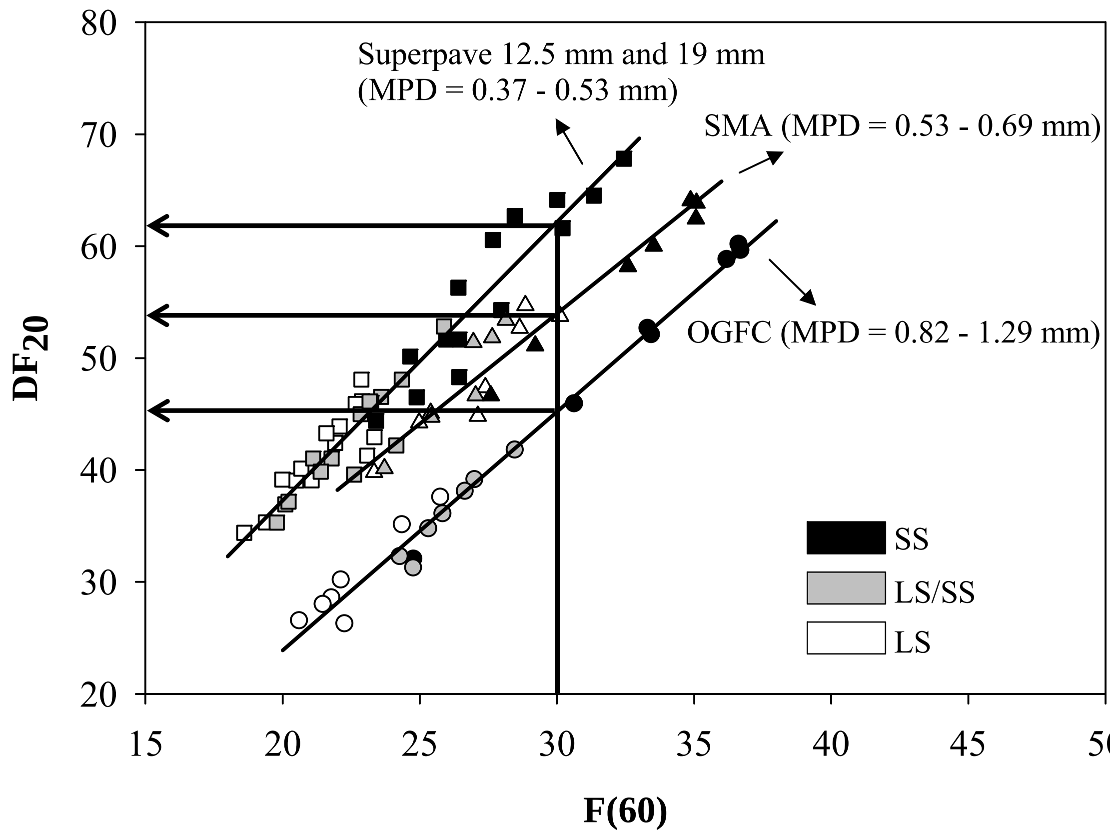

Figure 25
DFT (20) vs. F(60)

Development of Friction Guidelines for Surface Asphalt Mixture Design

The previous results have clearly shown that the influence of certain aggregate types on mixture friction resistance [i.e., F(60)] depends on the mixture type. With the high macro-texture associated with the OGFC and SMA mixture types, a satisfactory friction resistant surface mixture may be designed with relatively low polishing resistant aggregates or aggregate blends. Therefore, a successful mixture friction design guide should consider the combination effects of both mix design and aggregate type on the mixture friction resistance, not just the aggregate type alone as indicated by the current LADOTD friction guideline. In addition, mixture friction resistance is a function of traffic load and usually measured by friction trailer in terms of FN values in the field. To develop a new friction guideline for the surface asphalt mixture design in Louisiana, the following relationships were introduced based on the test results obtained in this study.

Relationship between SN and F(60)

ASTM E 1960 provides the conversion equation of F(60) based on the friction-trailer measured FN and MTD (mean texture depth) as below:

$$F(60) = -0.023+0.607\times FN_{50R}*e((80-60)/Sp)+0.098\times MTD \qquad (15)$$

where,

FN_{50R} = Friction Number at 50 mile/hr measured by friction-trailer,
Sp = Speed Constant (-11.6+113.6×MTD), and
MTD = Mean Texture Depth.

LTRC maintained a multi-year inventory data of friction trailer measurements (SN50R) for 12 measurement sites on Interstate I-10 and I-12. The control sections of those sites had the control numbers of 454-02, 450-08,450-11, and 450-13. Based on the projects' mix design information, all wearing course mixtures of those pavements tested were a Superpave 19-mm mixture, designed with a similar aggregate gradation and aggregate blends as considered in the laboratory friction tests of this study. In addition, the aggregate sources used were also similar to those used in this study. To develop a direct relationship between FN and F(60), this dataset was used and the result is presented in equation (16). Since all FN_{50R} measurements contain only one source of mixture type (i.e., Superpave 19 mm), the influence of the macro-texture on mixture friction resistance may be cancelled out in the developed relationship. When more field data are collected, the direct relationship between the FN and F(60) can be also developed for other mix types used in Louisiana. The developed FN vs. F(60) will be used in the laboratory friction mix design described later. The predicted F(60) used equation (16) is presented in Figure 26. Figure 26 shows that F(60) of 26 is equivalent to an FN_{50R} value of 40.

$$F(60) = 0.649\ FN_{50R} + 0.0572 \qquad (16)$$

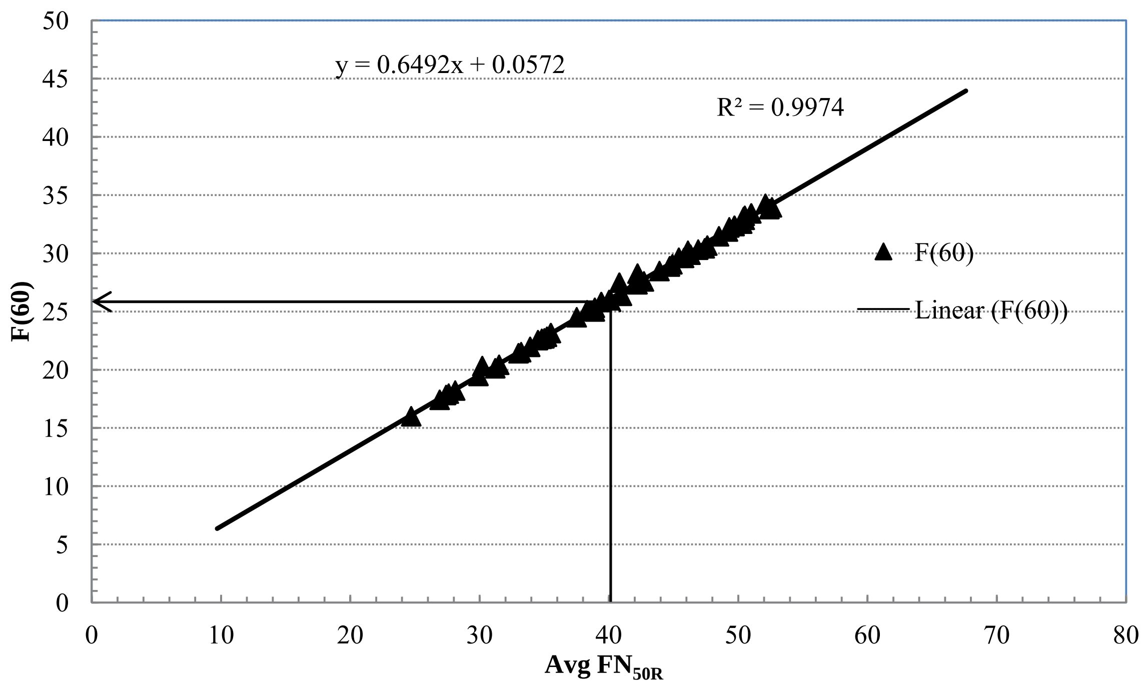

Figure 26
Conversion of field FN_{50R} into F(60)

Relationship between F(60) and PSV

Figure 27 shows the relationships between PSV and F(60) for different mix types considered in this study. For the given PSV value (or aggregate type), Figure 27 provides a direct estimation of F(60) for different mix types selected during a mix design. Figure 27 also indicates that, to achieve a same level of F(60), different mix types require different aggregate blend's PSV. Since only three data points were available for each mix type in this study, a polynomial function was chosen to fit the data curves. The prediction equations of F(60) based on different PSV values for the four mix type considered are presented next:

$$F(60) = 0.067(PSV)^2 - 3.84PSV + 74.46 \qquad \text{for Superpave 19 mm} \qquad (17)$$

$$F(60) = 0.106(PSV)^2 - 6.19PSV + 108.75 \quad \text{for Superpave 12.5 mm} \quad (18)$$

$$F(60) = -0.121(PSV)^2 + 9.417PSV - 153.52 \quad \text{for SMA} \quad (19)$$

$$F(60) = -0.066(PSV)^2 + 5.99PSV - 101.65 \quad \text{for OGFC} \quad (20)$$

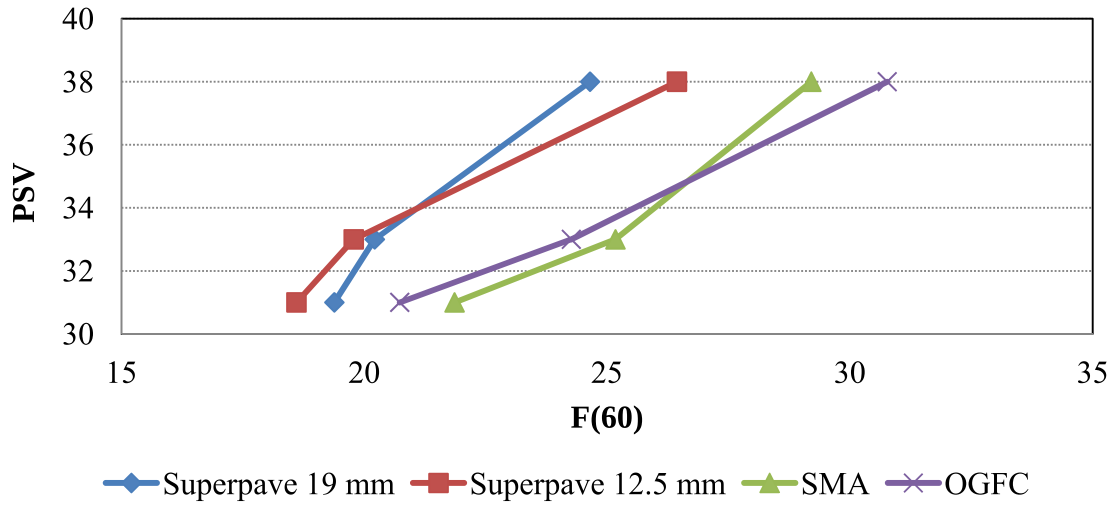

Figure 27
A relationship between F(60) and PSV

Prediction of F(60) at Different Polishing Cycles using DF_{20} and MTD

The data shown in Figure 23 was converted further into a prediction equation of F(60) at different polishing cycles based on DF20 and MPD measurements obtained in this study. The analysis included all four mix types and three aggregate blend's data. The 5000 cycle was considered as the initial cycle for all the mixes. A non-linear regression analysis in SAS was performed on the dataset and the following equation was obtained:

$$F(60) = (2.18 + 13.5 \times MPD + 0.38 \times DF_{20}) \times e^{(-1.73E\text{-}06 \times Cycle)} \quad (21)$$

Coefficient of Determination (R^2) = 88%

where,

F(60) = IFI at speed 60 km/hr,

MPD = Mean Profile Depth by Circular Texture Meter,

DF20 = Dynamic Friction Test value at speed 20 km/hr, and

Cycle = Polishing Cycle.

In general, the coefficient of determination is 0.88 for the relationship developed. The equation developed will be used in a laboratory friction mix design procedure described later.

Relationship between Traffic Loading and the Polishing Cycle (N)

Masad et al. developed a relationship between in-situ traffic loading and the laboratory polishing cycles (Masad, 2009):

$$TMF = 35600/(1+15.96\exp(-4.78\text{x}10^{-2}\ N) \quad (22)$$

where,

N is polishing cycle, and

TMF is traffic multiplication factor (TMF = AADT x Years in Service/1000).

Friction Guidelines for Surface Asphalt Mixture Design

Based on the relationships developed between lab and field friction-related characteristics, this study presents a design procedure for predicting the FN of asphalt mixtures that can be used as the friction guideline in a wearing course mixture design in Louisiana. The design procedure as outlined below consists of the following steps:

- Determine the friction demand for a specific mix design and select a design FN (e.g., FN = 38) at an anticipate design traffic volume index (TMF).
- Determine the required $F(60)_{des}$ based on the design FN using equation (16).
- Select a mixture type (i.e., Superpave 19 mm or 12.5 mm, SMA, and OGFC).
- Calculate the required PSV based in mix type and the required F(60) using equations (17-20).
- Choose aggregate blend based on the required PSV (note: the PSV values shown in the QPL list may be used; the blend PSV may also be determined for the blending chart developed by Ashby [Ashby, 1980]).
- Perform mix design and prepare testing slabs.
- Measure the DFT and MPD on the slabs.
- Calculate the laboratory polishing cycle N using equation (22) based on TMF.
- Calculate the $F(60)_{slab}$ based on DFT20, MPD, and N using equation (21).
- If $F(60)_{slab} > F(60)_{des}$, design is completed.

If not, go back to either the step of “choose mix type” or “choose aggregate blend.”

CONCLUSIONS

The surface friction properties of 12 mixtures including four HMA types and three aggregate blends were characterized in this study. The polishing resistance and texture characteristics of two selected aggregates (one low-polishing and one high-polishing) were determined using the British Pendulum, Micro-Deval, and AIMS devices. A three-wheel accelerated polishing device was used to polish laboratory fabricated mixture slabs, and the DFT and CTM were used to determine the changes of micro- and macro-textures of slab surface due to the accelerated polishing.

The laboratory results indicated the DFT measurements were fairly sensitive to the coarse aggregate types (related to micro-texture), but were not very sensitive to different mix types or aggregate gradations (related to macro-texture). The analysis of CTM measured MPD results indicated a strong relationship between the macro-texture and mixture type. Further analysis was based on the International Friction Index − the F(60) values. The F(60) results indicated that generally it is possible to blend low and high friction aggregates together to produce an asphalt mixture with an adequate field friction resistance.

A set of prediction models for estimating of mixture frictional properties was developed. Based on the developed prediction models, a laboratory mix design procedure that addresses the surface friction resistance of an asphalt mixture in terms of both micro- and macro-surface textures was presented. The developed frictional mix design procedure allows estimating a friction-demand based, design FN value for an asphalt mixture during the mix design stage.

RECOMMENDATIONS

- LADOTD should consider implementing the frictional design procedure developed in this study by selecting several currently used, typical wearing course mixtures to predict the available FN values using the backward steps of the frictional mix design procedure developed in this study and to check the results with field measured FN values.

- LADOTD should consider implementing the results of the NCHRP 1-43, *Guide for Pavement Friction*, for the management of pavement friction on existing highways in which three to five site categories based on friction demand levels may be established and the corresponding intervention and investigatory levels of FN values for each category may be determined to guide the frictional mix design.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
AIMS	Aggregate Imaging System
BPT	British Pendulum Tester
BPN	British Pendulum Number
CTM	Circular Track Meter
DFT	Dynamic Friction Tester
F(60)	Friction Number at 60 km/hr
FHWA	Federal Highway Administration
FN	Friction Number
FR	Friction Rating
HMA	Hot Mix Asphalt
IFI	International Frictional Index
JMF	Job Mix Formula
LADOTD	Louisiana Department of Transportation and Development
LTRC	Louisiana Transportation Research Center
LWST	Locked Wheel Skid Tester
MPD	Mean Profile Depth
MTD	Mean Texture Depth
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
NTSB	National Transportation Safety Board
OGFC	Open Graded Friction Course
PSV	Polish Stone Value
QPL	Qualified Product List
SAS	Statistical Analysis System
SGC	Superpave Gyratory Compactor
SMA	Stone Matrix Aggregate
Superpave	Superior Performing Pavement
TWPD	Three-Wheel Polishing Device
US	United States

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APPENDIX A

Evaluation of Field Friction Resistance Data

This section presents the evaluation of a historical friction inventory database for Louisiana pavements. The historic pavement friction inventory database is comprised of the Locked Wheel Skid Test (LWST) data for different pavement sections at various dates. The field test data have also been analyzed to find the threshold values of friction resistance and a relationship to predict field friction resistance from the mixture and aggregate properties.

Extensive work was done by LADOTD engineers to prepare the database for this study. The database was prepared by matching the database from the mix data from Pavement Management Section (PMS) database to the LWST test records (Historical Friction Database), which contains large amounts of test data since 1982. The nine-digit project number was used to match the PMS database to the LWST records. The LWST machine tests the wet friction resistance of the pavement by measuring the torque when the wheels are locked. According to ASTM E 274, the test speeds of LWST can be 30, 40, or 50 mph and this test can be performed with both smooth and ribbed tire. This study has only considered the ribbed tire LWST tests. In the historical friction database, most of the test data were measured at the test speed of 40 km/hr, whereas the rest of the data were at 50 km/hr. Also in this study, a FN_{40} denotes the friction resistance taken at the test speed of 40 km/hr with a ribbed tire.

Pavement Sections

A total of 294 road sections were considered. Test sections included a total of 1068 data records. The database includes 3 mix designs and 34 aggregate types. The road sections are distributed across nine districts in Louisiana and categorized by three functional classes of highways, which are interstates, U.S. highways, and LA highways. Further, aggregate types are categorized by the friction rating established by the LADOTD specification for the aggregate friction.

Table 19 shows the number of road sections distributed in each district in Louisiana. The New Orleans and Alexandria areas have the majority of selected sections. This selection of sections was also based on the availability of friction resistance data for particular sections, so some of the districts have fewer sections than others.

Table 19
Number of studied road sections in each district

District	Number of Test Sections
02	58
03	17
04	18
05	37
07	38
08	57
58	31
61	26
62	12

Aggregates Used

Table 20 includes all the aggregates used in the surface mix of the studied road sections. A total of 34 aggregates were used in those sections. The aggregates are categorized in different categories by their friction rating provided by LADOTD and their source code. The same aggregate types from different sources behave differently because of different physical and chemical properties; in this study each aggregate type from different sources has been treated as different aggregate types. Out of these 34 aggregates, only 21 aggregates were evaluated because of availability of their friction resistance data for at least two sections.

The road sections fall into three major functional classes of roadways. Table 21 shows the breakdown of road sections by the functional class with the friction resistance data statistics. The breakdown of the database into major functional class is associated with the identification of traffic groups in which the variation of friction resistance can be considered homogenous since the polishing of road surface aggregates are related to the traffic volume. Interstates show the highest average and median value of friction resistance. The interstates have high traffic volume, so the frictional consideration of an interstate pavement is more important.

To have a better understanding of the selected friction resistance data, a data distribution chart (Figure 28) is plotted. Figure 28 is the distribution of the selected data records for friction resistance at speed 40 km/hr. The majority of sites have the friction number between 30 and 50 (FN_{40}). There are 56 sections having the same SN_{40} equal to 37, which is the maximum number of sections having same friction number.

Table 20
Aggregate source used in pavement sections

No.	Source Code	Aggregate Type	LADOTD Friction Rating
1	A133	Chert Gravel	3
2	A602	Chert Gravel	3
3	A812	Chert Gravel	3
4	A607	Chert Gravel	3
5	A812	Chert Gravel	3
6	AA79	Chert Gravel	3
7	AB07	Chert Gravel	3
8	AX03	Chert Gravel	3
9	AX07	Chert Gravel	3
10	AX23	Chert Gravel	3
11	AX52	Chert Gravel	3
12	AA78	Chert Gravel	4
13	AA15	Granite	3
14	A037	Limestone	3
15	AA50	Limestone	3
16	AB29	Limestone	3
17	AB37	Limestone	3
18	A056	Limestone	4
19	AA56	Limestone	4
20	AA44	Novaculite	2
21	A079	Oolitic Stone	3
22	AA55	Oolitic Stone	4
23	AA92	Rhyolite Traprock	2
24	A061	Sandstone	1
25	A066	Sandstone	1
26	A067	Sandstone	1
27	A089	Sandstone	1
28	AA35	Sandstone	1
29	AA66	Sandstone	1
30	AB13	Sandstone	1
31	AB40	Sandstone	1
32	AA52	Sandstone	2
33	A033	Syenite Granite	3
34	AA43	Syenite Granite	3

The selected road sections included three pavement surface course mix types, i.e., 3, 8 and 8F. These mix designs have been historically used for wearing courses in Louisiana. The historic friction database also included friction data for airport runway wearing courses, which are not included in this study. Other new types of mix designs could not be considered in the selection because of either unavailability of friction resistance data or not having multiple year data.

Table 21
Pavement sections by their function class

Type	No. of Sections	Average	Median	Min	Max	Std Dev
Interstate	28	42.05	42.66	24.48	57.29	7.02
US Highway	119	39.27	39.22	25.52	55.79	5.82
LA Highway	147	37.64	37.45	24.82	56.24	5.74

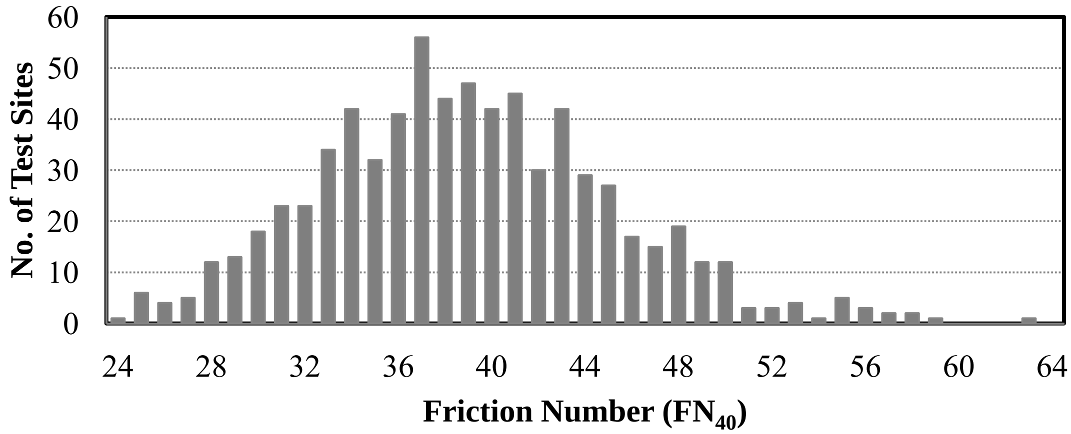

Figure 28
Distribution of friction resistance data

Mixture Requirement

The three mix design types 3, 8, and 8F, are the three traditional mix designs used to design pavements in Louisiana. Gradation of these three mix design types have been given in Table 22. The mix types 08 and 8F are intended to provide better friction and designed for higher volume of traffic.

Table 22
Mixture requirements (LADOTD specification, 2000 edition)

Mixture Design				
U. S. sieve % passing	Type 3	Type 8	Type 8F	Mix Tolerance[1]
	Wearing Course (Mix code 05) ¾ inch Nominal Size	Wearing Course (Mix Code 20) ¾ inch Nominal Size	Wearing Course (Mix Code 22) ¾ inch Nominal Size	
1 ½ in.	-	-	-	±6
I in.	100	100	100	±6
¾ in.	90-100	90-100	90-100	±6
½ in.	89 Max.	89 Max.	89 Max.	±6
3/8 in.	-	-	-	±6
No. 4	-	-	-	±6
No. 8	23-49	23-49	23-49	±5
No. 16	-	-	-	±3
No. 30	-	-	-	±3
No. 50	-	-	-	±3
No. 100	-	-	-	±2.0
No. 200	2.0-8.0	2.0-8.0	2.0-8.0	±2.0
Extracted Asphalt %	-	-	-	±0.4
% Crushed Min.	85	90	90	
Aggregate[2]	A,B,C	A,B,C	A,B,C	
Mar. Flow 1/100 in.	6-15	6-15	6-15	
% Air Voids	3.0-5.0	3.0-5.0	3.0-5.0	
% VMA, Min.	13.0	13.0	13.0	
% Rap, Max.[3]	0.0	0.0	0.0	
Binder Type	PG 70-22m	PG 76-22m	PG 76-22m	

[1] Job Mix Formula based on approved mix design

[2] A - Gravel, B – Slag, C – Stone approved for wearing course

Data Summary

Table 23 shows the summary of friction database by functional class, mix type, and friction rating of aggregate type.

Table 23
Summary of the LWST data

Function Type	Mix Type	Friction Rating	N	Mean	Min.	Max.	Std. Dev.
Interstate	**Type 03**	FR II	1	50.99			
		FR III	70	36.91	28.71	49.33	4.99
		FR III + FR IV	5	42.82	42.66	42.97	0.15
		FR IV	3	42.82	42.66	42.97	0.15
	Type 08	FR I +FR III	37	42.73	28.10	49.95	6.56
		FR III	30	35.24	26.20	52.73	6.09
	Type 8F	FR I	6	48.93	46.45	50.95	3.67
		FR I +FR III	3	38.81	34.40	41.90	3.92
		FR II	12	47.35	38.90	57.28	6.14
		FR III	6	41.81	40.89	42.42	1.12
		FR II +FR III	4	46.98	43.90	49.70	2.62
LA HWY	**Type 03**	FR II	5	37.51	29.64	42.70	6.14
		FR II+ FR III	1	48.54			
		FR III	232	37.83	25.13	57.88	5.37
		FR III + FR IV	4	37.14	32.63	40.84	4.06
	Type 08	FR II	9	42.26	39.17	45.86	5.87
		FR II + FR III	66	38.98	29.21	55.11	4.91
		FR III	21	40.16	30.28	45.42	3.74
		FR I + FR III	12	46.67	44.42	48.65	4.26
	Type 8F	FR I + FR III	12	40.67	32.63	44.18	3.40
		FR II	7	49.23	47.56	54.80	2.62
		FR III	16	39.48	29.64	50.29	7.04

Table 23
Summary of the LWST data (contd.)

Function Type	Mix Type	Friction Rating	N	Mean	Min.	Max.	Std. Dev.
US HWY	Type 03	FR I	2	56.35	51.00	61.70	7.57
		FR II	35	43.49	39.95	47.93	3.62
		FR III	176	38.34	27.35	60.47	6.18
		FR II + FR III	12	33.92	32.60	36.47	1.31
		FR II + FR IV	1	42.84			
	Type 08	FR I	11	49.97	47.09	54.13	5.51
		FR I + FR II	1	40.88			
		FR II	49	41.70	32.89	47.09	3.55
		FR I + FR III	83	42.25	30.79	56.44	6.78
		FR I + FR IV	1	34.90			
		FR III	24	43.61	33.97	57.67	6.61
		FR II + FR III	5	40.33	38.68	41.20	1.00
	Type 8F	FR I	6	43.81	40.26	49.63	7.75
		FR I + FR III	14	40.28	35.06	44.10	4.50
		FR II	22	46.46	41.33	52.69	3.64
		FR III	45	39.69	29.60	52.16	5.94

Traffic Volume

The friction resistance data set has been further analyzed for comparison purposes under three roadway functional classes. Interstates have the highest volume of traffic followed by US highways and state local highways denoted as LA highways. These three groups of traffic sets have been considered to represent three different polish effects to the surface aggregate of the asphalt pavements. Table 24 illustrates the average daily truck traffic (ADTT) for three functional classes.

Table 24
ADTT by functional class

Functional Class	Count	Average	Min.	Max.	Std. Dev.
Interstate	180	13378	2723	45091	9730
LA Highways	409	968	115	4990	799
US Highways	479	2360	200	10053	1749

For the analysis of deterioration in friction resistance versus the polish cycle, the polish effect has been evaluated in terms of Traffic Index (TI). The traffic index basically represents the annual average daily truck traffic. The definition of the traffic index is given in the following equation:

$$\text{Traffic Index (T.I.)} = \text{Age of Pavement (in days)} \times ADT \times TTP \times 365 / 1 \times 10^6 \qquad (23)$$

where,
T.I. = Traffic Index (Million Vehicles),
ADT = Average Daily Traffic (No. of Vehicles), and
TTP = Truck Traffic Percentage of ADT (%).

The high friction aggregate indicates high initial friction resistance compared to medium and low friction aggregate. At a high traffic index, which represents a high polish cycle, the medium and low friction aggregate shows the same polishing effect. A traffic index of 50 is approximately equivalent to the polishing of an interstate pavement section for 10 years with ADTT around 14,000. It is quite evident that the pavement sections with high friction aggregate outweigh the pavements with medium or low friction aggregate in terms of retaining the friction resistance.

Mix Design

Figure 29 shows the box plot of the mix types and the friction resistance. It can be observed from Figure 29 that significantly large variations of the measured FN_{40} values exist among all mix types considered. Such high friction number variations may be partially due to individual traffic polishing history and partially due to different aggregates used in the mix design (Table 23). In general, the median value for type 8F is highest with less standard deviation than the other two types, whereas the median value for type 8 is slightly higher than type 3.

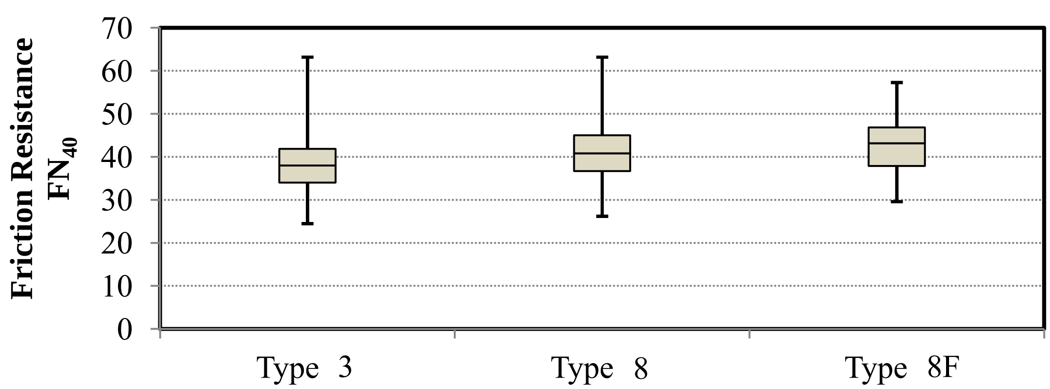

Figure 29
Box plot for the different mixtures by friction resistance

To further evaluate the mix types, the friction resistance data were sorted by the aggregate type and the average FN_{40} value for each aggregate type was plotted. Figure 30 illustrates friction performance of different aggregate types by mixture type. For this analysis, the different sources of the same type of aggregate were considered the same. Aggregate Rhyolite Traprock shows the highest average FN_{40} among all the aggregate types. For types 8 and 8F, the friction number for a same aggregate type is higher than that of type 3.Within same mixture types, the aggregates show a different friction resistance. The two aggregates, limestone and sandstone in particular, show more difference in their friction resistance for mix 8F. These observations clearly indicate that the mixture design plays an important role in the variability of friction resistance.

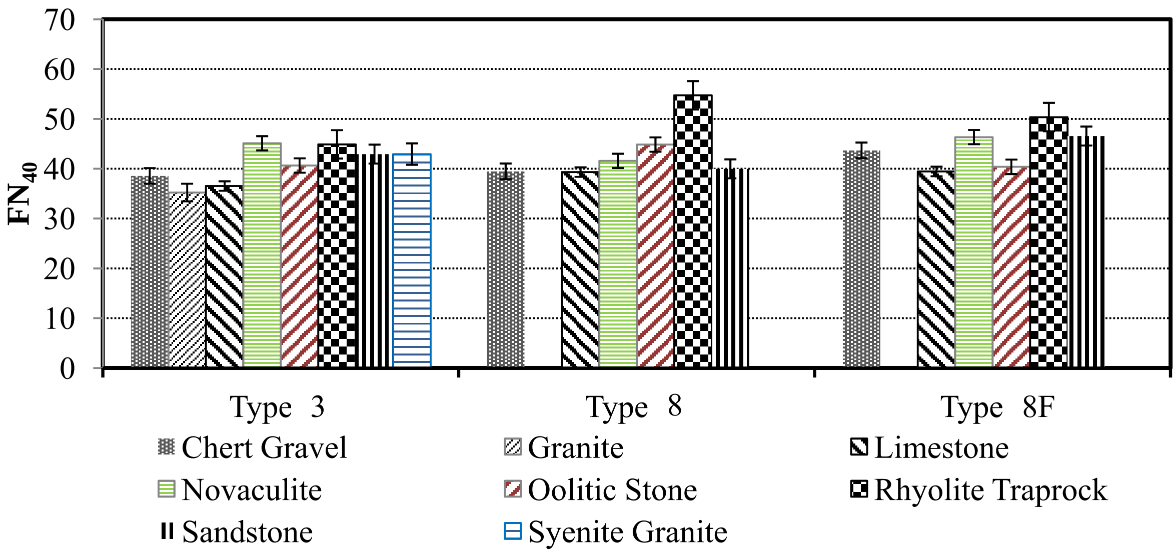

Figure 30
Different aggregates friction resistance by mixtures types

Aggregate Type

In order to evaluate different aggregate types, some typical aggregates used in pavement construction in Louisiana were studied for performance and degradation evaluation. Table 25 shows a listing of a total of 21 typical aggregates used in Louisiana pavements. Individual aggregates were given alphabetic symbols based on geological types. Each alphabet denotes a type of aggregate; whereas, digit suffixes denote different sources of same aggregate types.

Figure 31 shows average friction resistance for typical aggregates by mix types. The mix 8F is higher than mix type 3 for all the aggregates compared except Oolitic stone and Rhyolite

Traprock. Also mix type 8 shows a higher value of friction resistance than type 3 except for Novaculite. Limestone from different sources show similar results, but sandstone A067 has a much higher average friction resistance than sandstone from other sources.

Figure 32 presents the comparison of different aggregate types and sources by the traffic volume group. Low friction group aggregates, such as limestone, show better performance when used in low volume traffic groups, i.e., LA highways and US highways; whereas, high friction aggregates such as sandstone, show higher performance when used on interstates than the other two highways.

Table 25
Typical aggregates selected for comparison

Aggregate Type	Source Code	Symbol
Chert Gravel	A133	A1
Chert Gravel	A602	A2
Chert Gravel	A812	A3
Chert Gravel	A132	A4
Chert Gravel	A607	A5
Granite	AA15	B1
Limestone	A037	C1
Limestone	AA50	C2
Limestone	AB37	C3
Limestone	AB29	C4
Novaculite	AA44	D1
Oolitic Stone	A079	E1
Rhyolite Traprock	AA92	F1
Sandstone	A067	G1
Sandstone	AA52	G2
Sandstone	A061	G3
Sandstone	AA66	G4
Sandstone	AB13	G5
Syenite Granite	A033	J1

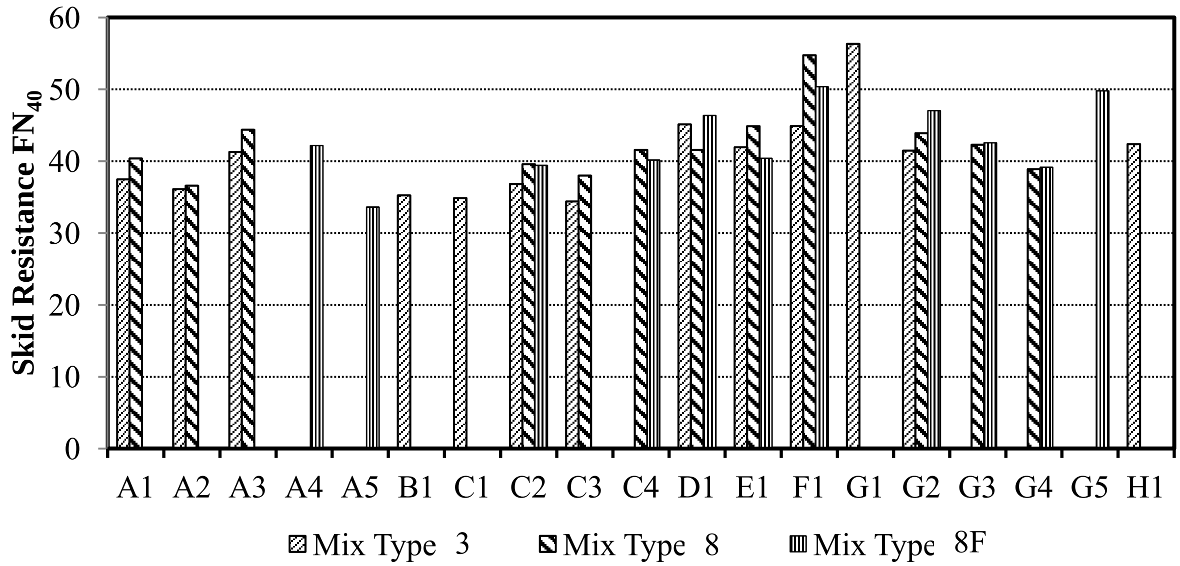

Figure 31
Average friction resistance by mixtures types for typical aggregates

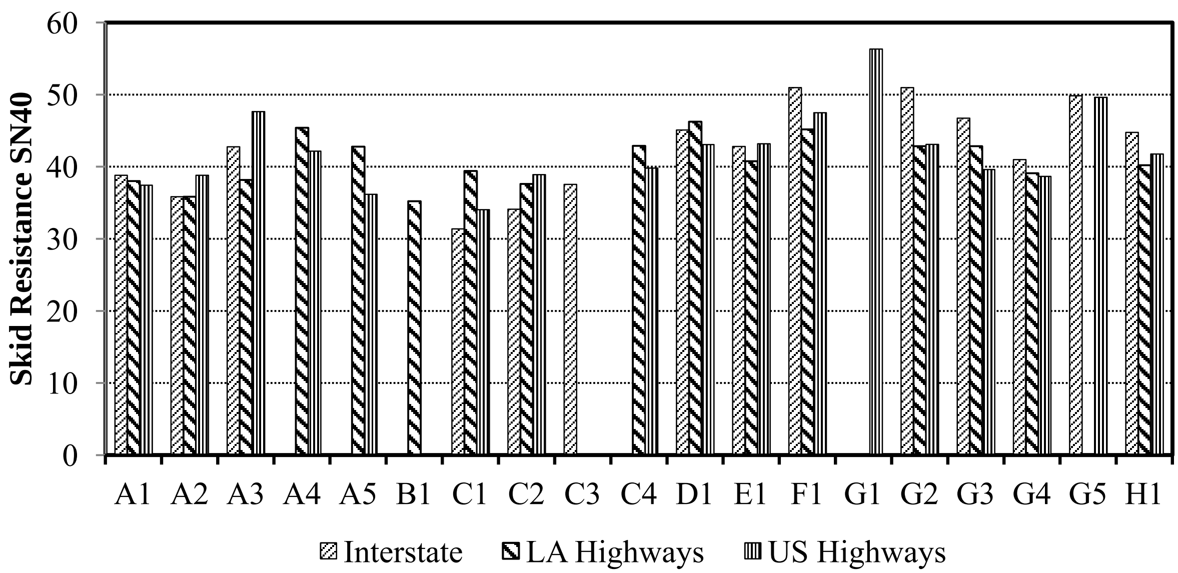

Figure 32
Average friction resistance by traffic volume types for typical aggregates

Aggregate Blend in the Asphalt Mixes

This analysis attempts to evaluate the combination of limestone with another aggregate mix in achieving initial friction. Figure 33 shows the variation in initial friction resistance with the variation in limestone percentage in a limestone only mixes. The FN_{40} shows a decreasing trend with the increase of limestone coarse aggregate.

Figure 34 shows the initial friction resistance of a limestone and sandstone combined mix. The increase of limestone aggregate in the coarse aggregate shows a decrease in initial friction resistance.

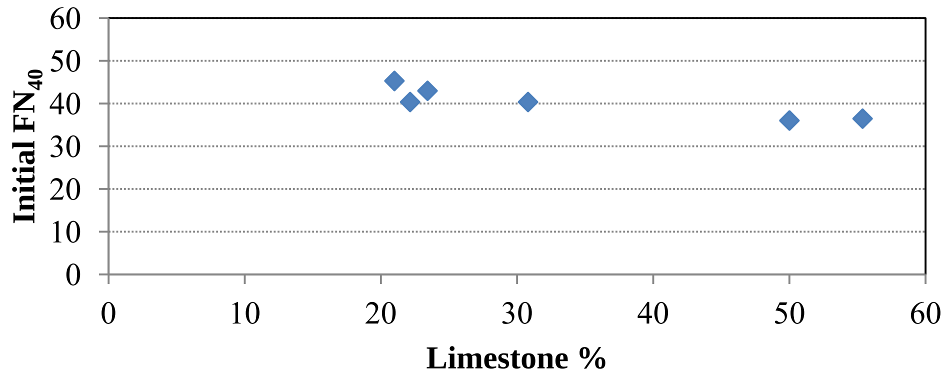

Figure 33
Initial friction resistance by coarse limestone % as the single aggregate type mix

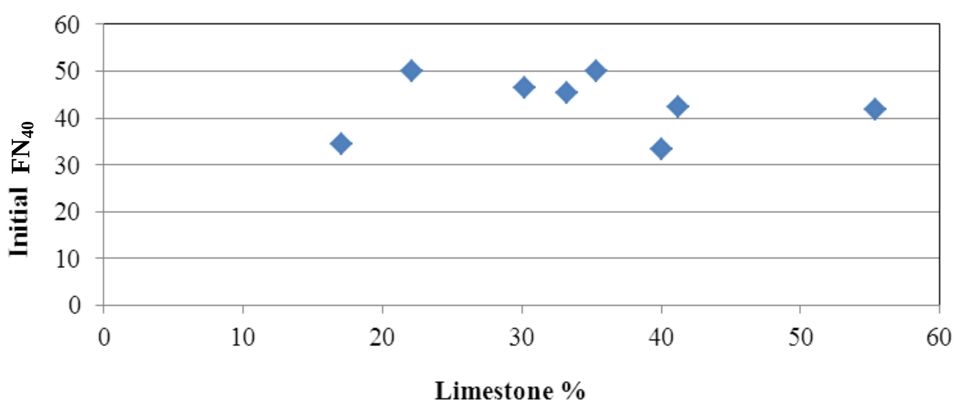

Figure 34
Initial friction resistance by limestone % in mixes combined with sandstone aggregate

Deterioration in Friction Resistance

The deterioration rate of friction resistance of aggregate is important in terms of retaining frictional resistance for a long term of pavement life. This study attempts to evaluate limestone, novaculite, and sandstone aggregate types that represent friction level I, II, and III, respectively. The polishing curves for these three aggregate types against the traffic index are plotted in Figures 35, 36, and 37. Note that each marker type in the figures represents a different pavement section. Since each pavement section has its own degradation curve, an average degradation curve as a fitted line for all source type and section has been plotted for each aggregate type.

Figure 35 represents the degradation curve for the limestone (friction rating III) aggregate type. The sources AB37 and AB29 show higher initial SN_{40} values than AA50. Figures 36 and 37 show the degradation in friction value for novaculite and sandstone aggregates, respectively.

The limestone mixes show the highest rate of depletion in friction compared with novaculite and sandstone aggregate types, as it shows significant loss of friction resistance early in life. The sandstone aggregate type has high retention of friction resistance; whereas, novaculite aggregate shows a degradation trend in between limestone and sandstone. The two sources of sandstone show different values of friction resistance with traffic, which may be due to different chemical composition of these two aggregates from different sources.

It can be inferred from these plots that the friction level of aggregates actually addresses the polishing rate of surfaces of asphalt mix. These plots also suggest that the combination of friction level III and I aggregates may produce a blend of aggregate mix that can represent the friction level II aggregate.

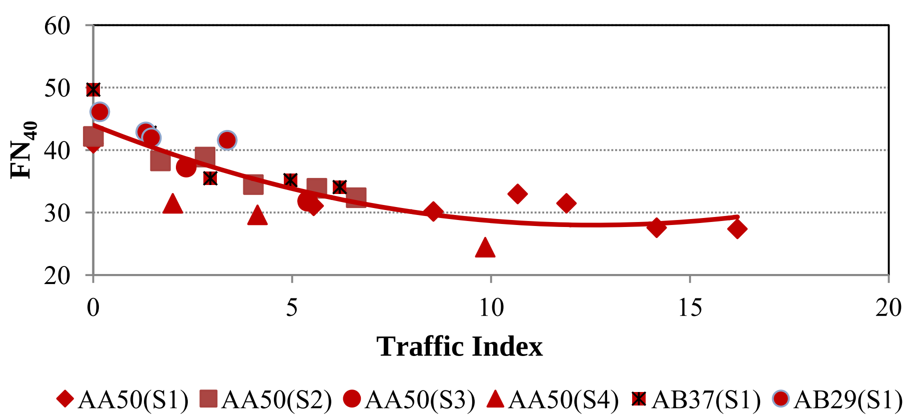

Figure 35
Degradation of limestone aggregates

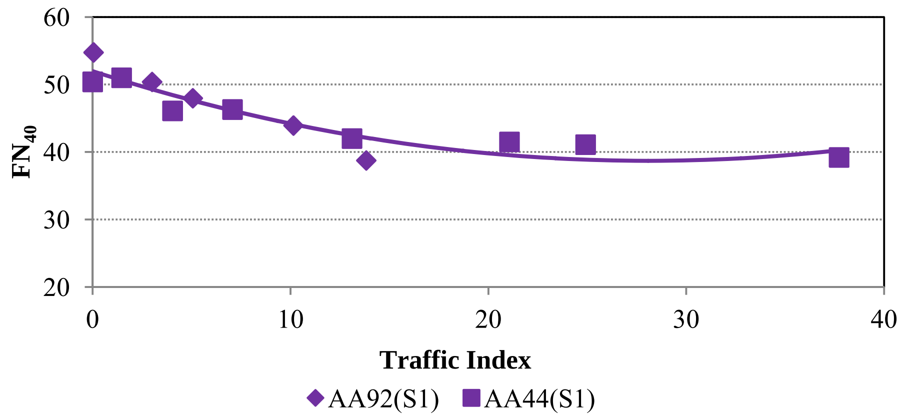

Figure 36
Degradation of novaculite aggregates

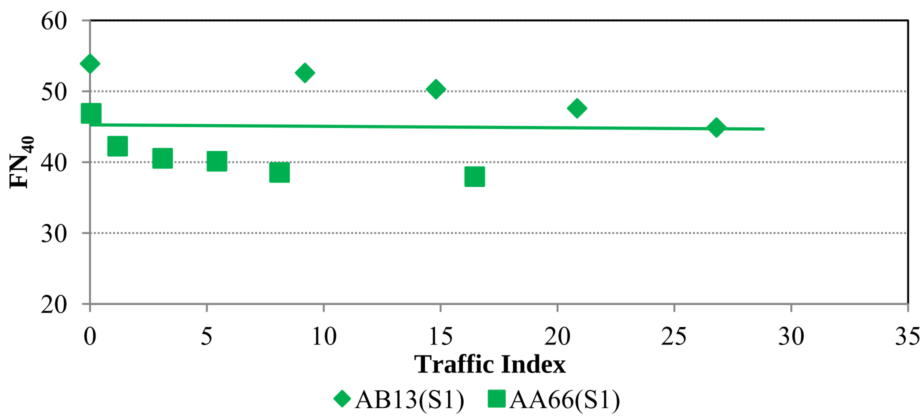

Figure 37
Degradation of sandstone aggregates

Investigatory and Intervention Level of Friction

Pavement Friction Threshold. Out of three methods recommended by a guide for friction (Hall et al., 2009) to determine threshold value of friction, the method 1 suggests establishing the intervention and investigatory level by drawing the age vs. friction resistance plot. The slope at which the friction resistance starts to decrease significantly is set as investigatory level, and the intervention level is set as a friction number of 5 to 10 percent less than that of the investigatory level.

The second method uses the age vs. friction resistance and the crash rate plot together. The investigatory and intervention levels are plotted by overlapping these two curves. The point on the crash rate curve at which the rate increases significantly is set as the intervention level. The point at which the friction resistance drops significantly in the age vs. friction resistance plot is set as the investigatory level. The third method uses the distribution of friction resistance data according to number of sites and the crash rate plot together. The investigatory level is set as the mean friction value minus 1.0~ 2.0 standard deviations of the distribution of sections and adjusts to where wet-to-dry crashes begin to increase considerably. On the other hand, the intervention level is set as the mean friction value minus 1.0~ 3.0 standard deviations and adjusts the level to a minimum satisfactory wet-to-dry crash rate.

This study has adopted the third method, but without the crash rate data to establish intervention and investigatory level by the plot of friction resistance distribution from historic pavement friction data inventory. The third method is more robust and provides a tool for decision-makers to consider the economic impact of friction level.

The investigatory and intervention level of friction resistance number for the pavements are fixed by plotting the friction resistance data distribution chart (Figure 38). The average FN value for this distribution is 38.92 with standard deviation 6.28; and taking a conservative approach, FN = 38 is established as the investigatory level. The intervention level is chosen as "investigatory level – 1.2 * standard deviation." The intervention level was estimated at FN = 32.

A high value of investigatory number would be un-realistic as LADOTD cannot perform the monitoring of a very large number of sections throughout the year. Also the crash rate plot gives a better realization of the friction level at which the rehabilitation must be needed. But in this study due to the lack of crash rate data, the threshold friction level is estimated in a slightly modified way than recommended by method three of the *Guide for Friction*. A better assessment of these values can be done by individually analyzing an array of friction demand category sections. But the establishment of friction demand categories for Louisiana pavements was beyond the scope of this study.

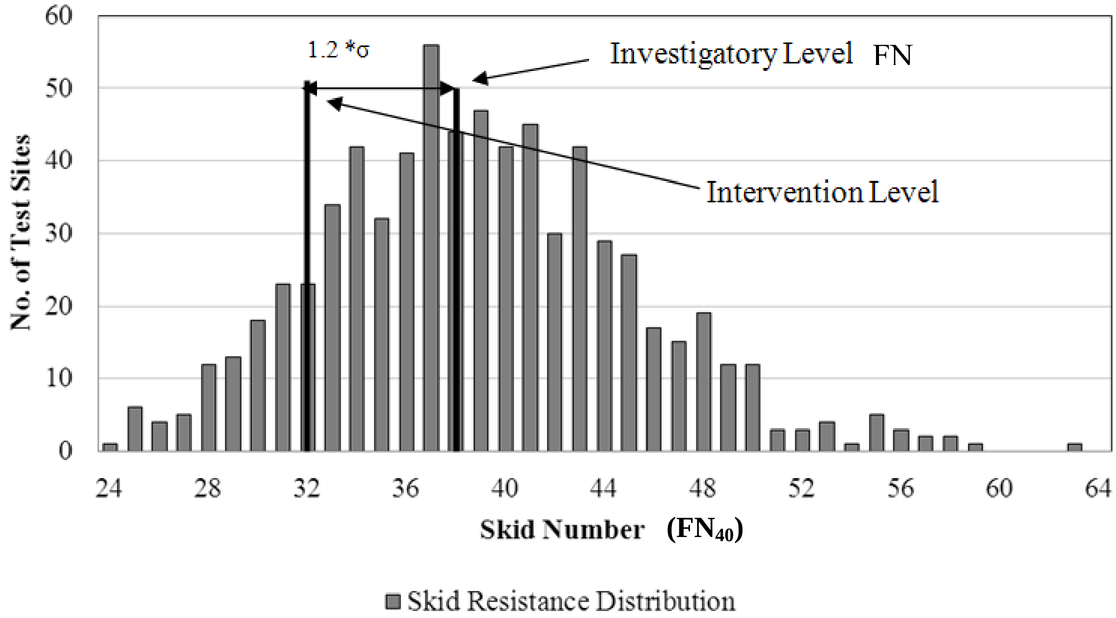

Figure 38
Estimation of investigatory and intervention level of friction

Further field evaluation is recommended for the fixation of threshold values of friction in the state of Louisiana. Since a statewide single threshold value would be impractical, a friction demand category should be established identifying different type of pavement sections such as bridge, corners, intersections, etc. For high volume roads, such as interstates, the threshold value would be highly useful to recommend rehabilitation for friction demand such as milling, grinding, etc.

Relationship between Friction Resistance and Mix and Aggregate Properties

Different aggregate properties and their influence on the friction resistance have been evaluated by the ANOVA analysis. Significance of different test variables along with traffic and polishing effect on friction resistance is shown in Table 26. P-values less than 0.05 denote the significance; i.e., FN value is affected by particular variable.

The significance test shows that traffic index, fine aggregate percentage in mix, SSD density, LA abrasion loss value, and silica percentage of aggregate influence the FN of asphalt mixes. This test was done for all the mix data so some of the variables did not show any influence on FN.

Table 26
Different variables and their significance on friction resistance

Variables	t-value	P-value
Traffic Index	-2.07	0.04
Coarse Aggregate Percentage	0.93	0.35
Fine Aggregate Percentage	1.40	0.001
Saturated Surface Dry Density	2.81	0.01
Water Absorption	1.69	0.09
Magnesium Soundness Loss	0.77	0.44
LA Abrasion Loss	3.48	0.01
Silica Percentage	3.23	0.001
Polish Stone Value	1.53	0.13

Further, a relationship of friction resistance with traffic volume and aggregate gradation has been investigated. The variables for the correlation of FN used are coarse aggregate index (CAI), fine aggregate percentage (FAP), and traffic index (TI). FN is assumed as the function of all the above indices, i.e.,

$$FN = f\{CAP, FAP, T.I.\}$$

where,
CAI = Coarse (crushed) aggregate index,
FAP = Fine aggregate percentage in the mix, and
T.I. = Traffic index.

The evaluation of friction resistance in terms of traffic and aggregate percentage in the mix was performed to investigate the mix effect on friction resistance.

FN Prediction Model

The SAS (version 9.2) software was used to find the multiple linear regression model for FN. Table 27 shows the Pearson's correlation coefficient of variables for different mixes and their corresponding p-values. The p-values less than 0.05 denote significance. Unlike the previous significance test (Table 26), coarse and fine aggregate percentages show influence on FN values, since this analysis was performed separately for each mix.

All three mixes showed a decreasing trend for mean friction resistance with an increase in traffic index. The friction resistance was found inversely proportional to the fine aggregate index more than with the coarse aggregate index. It can be inferred from these relationships that the friction resistance can be related with the type of asphalt mixes.

Table 27
Pearson's correlation coefficient p-value statistics

	FN	TI	CAI	FAP
Type 3	1	-0.2837	0.72137	-0.60855
P-value		*0.0001*	*<0.001*	*<0.001*
Type 8	1	-0.25146	0.55891	-0.33195
P-value		*0.0307*	*<0.001*	*0.0039*
Type 8F	1	-0.24302	0.52797	-0.26868
P-value		*0.0491*	*<0.001*	*0.0363*

All three mixes were treated separately to find their own models. The linear regression models for mixes 03, 08, and 8F are given as equation (2), (3), and (4) respectively.

Mix Type 3

$$FN = 29.049 - 0.0722TI + 0.1475\ CAI - 0.2426\ FAP \quad (24)$$

$R^2 = 0.62$

Mix Type 8

$$FN = 37.342 - 0.0629TI + 0.0836\ CAI - 0.2537\ FAP \quad (25)$$

$R^2 = 0.42$

Mix Type 8F

$$FN = 39.499 - 0.2089TI + 0.064\ CAI - 0.2279\ FAP \quad (26)$$

$R^2 = 0.45$

where,
FN = Friction Number (Measured by LWST with Ribbed Tire) at speed 50 mph.
TI = Traffic Index,
CAI = Coarse Aggregate Index,
FAP = Fine Aggregate Percentage, and
PSV = Polished Stone Value (measured with British Pendulum Number).

The plots between actual and predicted FN values are shown in Figures 39, 40, and 41.

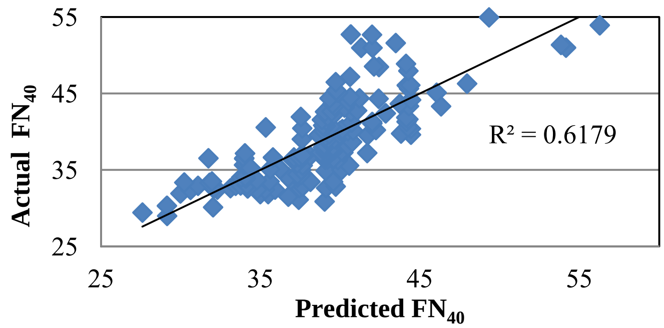

Figure 39
Actual FN vs. predicted FN from model (2) for Type 3

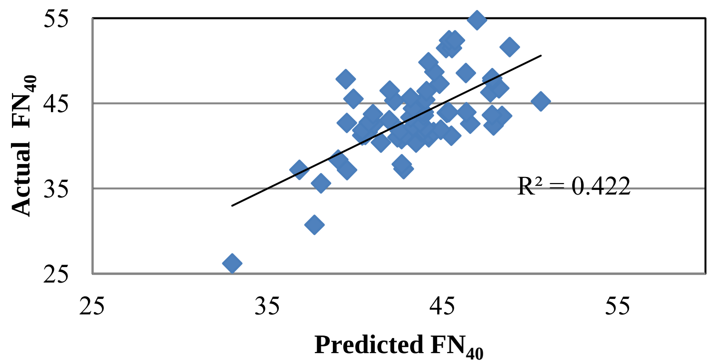

Figure 40
Actual FN vs. predicted FN from model (3) for Type 8

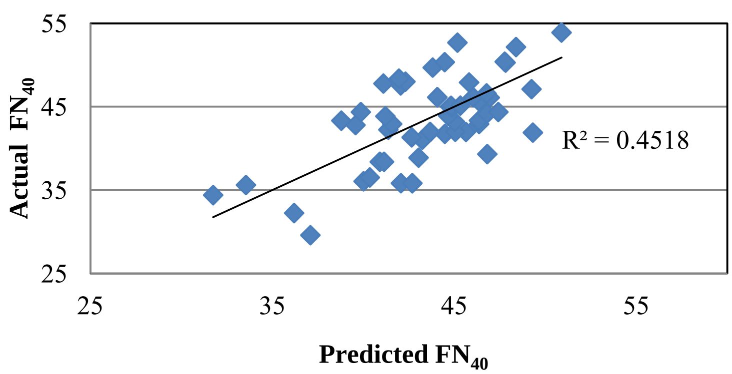

Figure 41
Actual FN vs. predicted FN from model (4) for Type 8F

Further these three relationships to predict FN_{40} are compared to evaluate the prediction pattern. Figure 42 shows the prediction lines for three mixes. The mixes 8 and 8F shows similar predicted FN values, both higher than the FN values for mix 3.

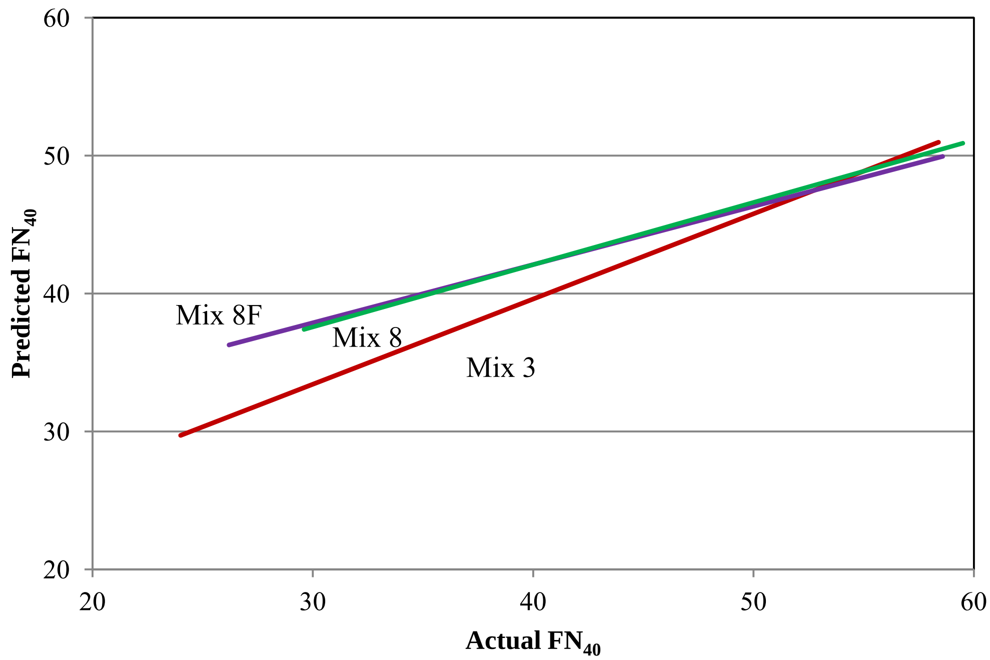

Figure 42
Prediction comparison of relationships for different mixes

Table 28 presents the field measured FN data that were used in developing the relationship between FN and F(60) in this study.

Table 28
Friction number database of Superpave mixes used in the study

Hwy	Ctrl. Sect.	Log Mile	Aggregate Type	Friction Rating	Avg. FN_{50}	Age (Yrs)	MTD (mm)	ADT	S_p	F(60)
Spring LA 3002-Walker LA 447										
I-12	454-02	1.91-7.68EB	Sandstone (AB13)	I	53.9	0.00		74022		
					52.6	1.57	3.50		385.43	33.95
					50.3	2.53	3.33		366.69	32.55
					47.6	3.56	3.75		414.17	30.67
					44.9	4.93	3.40		374.41	29.06
I-12	454-02	7.68-1.91WB	Sandstone (AB13)	I	52.8	0.00		74022		
					52.4	1.57	3.55		391.11	33.80
					49.2	2.53	3.26		358.28	31.87
					47.4	3.56	4.16		460.52	30.43
					42.7	4.93	3.61		398.04	27.58
Walker LA 447-Livingston LA 63										
I-12	454-02	7.68-4..51EB	Novascotia (AA44)	II	50.4	0.00	2.55	74022	277.85	33.10
					51	0.25	2.63		286.71	33.43
					46.1	0.69	2.65		289.78	30.22
					46.3	1.21	3.61		398.50	29.88
					42	2.24	3.02		331.47	27.35
					38.9	3.60	3.11		341.70	25.32
I-12	454-02	14.51-7.68WB	Novascotia (AA44)	II	50.5	0.00	2.49	74022	271.38	33.22
					52.1	0.25	2.51		272.97	34.25
					45.4	0.69	2.88		315.45	29.62
					44.7	1.21	3.53		388.84	28.89
					42.2	2.24	3.36		370.21	27.34
					38.5	3.60	3.20		351.58	25.03
Livingston LA 63-Tangipahoa Parish Line										
I-12	454-02	14.51-25.82EB	LS+SS (70/30) (AA50/AB 13)	I+III	49.7	0.00	2.99	43427	327.72	32.34
					48.5	0.96	3.19		350.78	31.46
					45.8	1.99	3.48		383.61	29.61
					43.9	3.35	3.26		358.51	28.47
I-12	454-02	25.82-4.51WB	LS+SS (70/30) (AA50/AB 13)	I+III	50.5	0.00	2.95	43427	323.63	32.87
					46.9	0.96	3.48		383.27	30.31
					44.7	1.99	3.62		399.06	28.86
					40.1	3.35	3.47		382.82	25.96