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16. Abstract
This report summarizes findings and recommendations regarding the usage of manufactured fine aggregates in portland cement concrete pavement (PCCP). The supporting research included both field and laboratory testing of aggregates and concrete properties that relate to skid resistance. Results show good correlation between friction values obtained using a Dynamic Friction Tester (DFT) and the micro-Deval test for fine aggregates (ASTM D7428). After comparing skid trailer values with CTM and DFT values, a correlation between the skid trailer and the DFT was established. Recommendations on how to blend carbonate sands with low acid insoluble residues to achieve good friction are presented in chapter 1 of this document. In chapter 2, a model for estimating skid numbers on concrete pavements with a mortar surface finish is presented.

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Recommendations for Achieving Adequate Surface Friction and Predicting Skid Values in Class P Concrete Containing Manufactured Fine Aggregates

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Table of Contents

Chapter 1. Recommendations for Achieving Adequate Surface Friction in Class P Concrete Containing Manufactured Fine Aggregates.. 1
1.1 Introduction...1
1.2 Significance and Use 2
1.3 Test Methods to Evaluate Aggregates and Estimate Concrete Surface Skid Performance 3
1.4 Field Evaluation...4
1.4.1 Test Equipment Correlation 4
1.4.2 Fine Aggregate Type Comparison 6
1.5 Laboratory Testing...7
1.6 Recommendations..11

Chapter 2. Establishing a Friction Prediction Model for PCC Pavements 15
2.1 Introduction..15
2.2 Field Data Analysis...15
2.3 Laboratory data Analysis 17
2.3.1 Effect of mixture variables on surface friction 17
2.3.2 Relationship between DFT and MD 19
2.3.3 Relationship between DFT and AI 20
2.4 Prediction Model for Computing $SN(50)_{smooth}$ 21
2.5 Verification of Prediction Model and Development of Design Charts 24
2.6 Conclusions..27

References.. 28

List of Figures

Figure 1.1: Wear Index vs. Siliceous Particle Content (Balmer and Colley, 1966) 2

Figure 1.2: Correlation between **$SN(64)_{ribbed}$** and **DFT60** (metric units) [6] 4

Figure 1.3: Computed vs. Measured $SN(50)_{smooth}$ 5

Figure 1.4: DFT60 vs. Measured $SN(50)_{smooth}$ 5

Figure 1.5: MPD vs. Measured $SN(50)_{smooth}$ 6

Figure 1.6: MFA vs. Blended Sands Skid Performance 7

Figure 1.7: DFT60 after 160,000 TWPD cycles vs. AI 8

Figure 1.8: AI vs. MD.. 9

Figure 1.9: Wear Index obtained for Different Mineralogy (Balmer and Colley, 1966) 10

Figure 1.10: DFT60 after 160,000 TWPD cycles vs. MD.. 11

Figure 1.11: Testing Polish Resistance of Fine Aggregates in PCCP 12

Figure 1.12: AI Values for Blends of Aggregates Meeting the 12% MD Limit 13

Figure 2.1: Computed Skid Numbers for Trial Field Sections as a Function of time 15

Figure 2.2: Computed Skid Numbers for Trial Field Sections as a Function of ESALs............. 16

Figure 2.3: Relationship between DFT60 after 160,000 TWPD cycles and AI 17

Figure 2.4: Finishing Crew Spray the surface of the 100% MFA section.................................. 18

Figure 2.5: Surface Texture between Wheel Paths (left) and on the Wheel Path (right) for the 100% MFA Sections (Section 1) 18

Figure 2.6: Relationship between DFT60 after 160,000 TWPD cycles and MD........................ 20

Figure 2.7: Relationship between DFT60 after 160,000 TWPD cycles and AI 21

Figure 2.8: Computed Skid Numbers as a Function of ESALs (Sections 1 and 2)..................... 22

Figure 2.9: Computed Skid Numbers as a Function of ESALs (Section 3) 22

Figure 2.10: Computed Skid Numbers as a Function of ESALs (Section 4) 23

List of Tables

Table 2.1: DFT60 Results for Concrete Slabs made with TXI Bridgeport FA 19
Table 2.2: Verification of the Prediction Model 24
Table 2.3: Design Chart Assuming $SN(50)smooth = 25$, well finished, and using AI............ 25
Table 2.4: Design Chart Assuming $SN(50)smooth = 25$, poorly finished, and using AI 25
Table 2.5: Design Chart Assuming $SN(50)smooth = 25$, well finished, and using MD 26
Table 2.6: Design Chart Assuming $SN(50)smooth = 25$, poorly finished, and using MD 26

Chapter 1. Recommendations for Achieving Adequate Surface Friction in Class P Concrete Containing Manufactured Fine Aggregates

1.1 Introduction

The mineralogy of coarse aggregate is vital for obtaining good skid performance in asphalt concrete. In PCC, however, the mineralogy of the fine aggregate is more important for obtaining good friction. The coarse aggregate only becomes an influencing factor in cases where the top surface of the pavement has been severely abraded or when coarse aggregate is intentionally exposed. Folliard and Smith (2003) identified fine aggregate mineralogy and hardness as important factors for obtaining good surface friction after the texture of a pavement is abraded. Since it is difficult to directly measure the resistance of fine aggregate to polishing, other indicator tests have been used. The most widely used test is the acid insoluble residue test (AI). The test assesses the presence of noncarbonated material in the fine aggregate; materials that have high carbonate content yield low residue because they dissolve in acid, while materials with low carbonate content yield a high residue. It is believed that the presence of acid insoluble material in the sand fraction generally improves skid resistance [Folliard and Smith 2003]. In PCC pavements, the fine aggregates exposed on the surface constitute the micro-texture (wavelength < 0.5 mm, amplitude = 1 to 500 μm). Micro-texture is important to maintain adequate friction in dry-weather conditions and wet-weather conditions when speeds are less than 45 mph (72 km/h) [Hall et al. 2009].

Many states have either banned the usage of carbonate fine aggregates in PCC pavements or have required blending those aggregates with harder aggregates to meet certain limits. In 1958, the need for skid resistant pavements was recognized by the First International Skid Prevention Conference [Balmer and Coley 1966]. After this conference, state agencies started developing equipment to test skid both in the laboratory and in the field [Balmer and Coley]. In 1958, Shupe and Lounsbury showed a correlation between calcium carbonate content of aggregates and skidding susceptibility [Balmer and Coley 1966]. Gray and Renninger (1965) recognized the contribution of siliceous sand particles in skid resistance and pioneered the acid insoluble residue test to analyze the amount of siliceous materials in the aggregates [Balmer and Coley 1966]. Balmer and Colley (1966) correlated results of a laboratory concrete skid performance test to the acid insoluble residue of the aggregates tested (Figure 1.1). They concluded that 25% siliceous fine aggregate content was satisfactory for skid performance with most aggregates. Most specifications base their limits on the study done by Balmer and Colley. Some specifications require a minimum of 25% siliceous sand content in pavement concrete, while other specifications have set limits based on acid insoluble residue (AI) values.

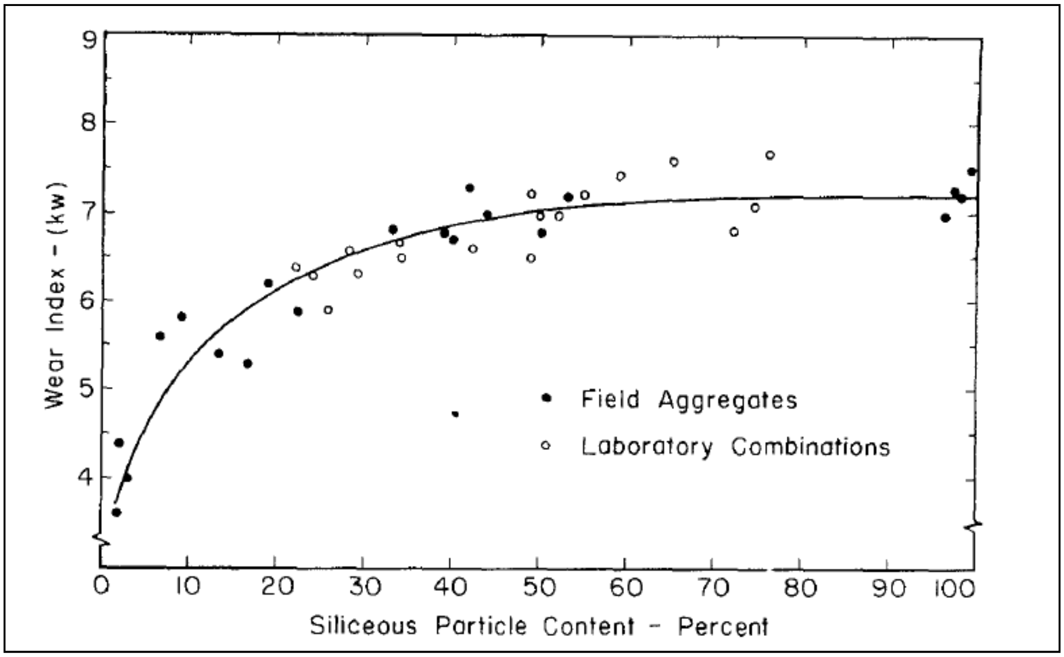

Figure 1.1: Wear Index vs. Siliceous Particle Content (Balmer and Colley, 1966)

Studies done after 1966 had similar conclusions as the study done by Balmer and Colley. Renninger and Nichols (1977) found good correlation between skid resistance (as determined by the British Pendulum Tester) and acid insoluble residue. As part of a study that evaluated micro-texture and macro-texture on PCC pavements in the United States, Hall and Smith (2009) found that tougher, more durable aggregates retain higher friction values. They found that the usage of limestone in Kansas and Illinois resulted in greater rates of micro-texture deterioration compared to the usage of high silica granite aggregates in Minnesota.

1.2 Significance and Use

Item 421 of the TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges requires that fine aggregates used in Class P Concrete meet a minimum acid insoluble residue (AI) limit of 60%. The AI test (Tex-612-J) indirectly evaluates the hardness of fine aggregates by assessing the presence of noncarbonated material. Since a more concentrated hydrochloric acid is used in the TxDOT test, all carbonate aggregates fail the AI test.

Districts in Texas, such as Dallas and Fort Worth, do not have sufficient sources of fine aggregates that meet the AI requirements. In order to meet an AI of 60%, the Dallas and Fort Worth Districts have to haul aggregates from distant pits and blend them with their local sources. The concern with using fine aggregates that do not meet AI limits is that those aggregates might result in poor skid performance. If more local carbonaceous aggregates are to be used in PCC pavements, it is important to investigate whether or not AI values for fine aggregates accurately relate to or predict the skid performance of PCC pavements.

An alternative method of evaluating and blending fine aggregates for pavement concrete is presented at the end of this document. This method aims at better quantifying the hardness of aggregates through their resistance to abrasion and crushing rather than their resistance to acid.

1.3 Test Methods to Evaluate Aggregates and Estimate Concrete Surface Skid Performance

Aggregates were tested for AI using the test described in Tex-612-J. The micro-Deval (MD) test described in ASTM D 7428 was used to evaluate the resistance of fine aggregates to abrasion and crushing. Although TxDOT uses Tex-461-A to evaluate coarse aggregates by MD, there is no state method to evaluate fine aggregates by MD.

The Locked-Wheel Skid Trailer (ASTM E 274) is the most common method used to evaluate skid resistance on pavements in the United States. The method consists of measuring the locked-wheel friction (100% slip condition) of a trailer towed behind a truck at a speed of 40 mph (64 km/h) or 50 mph (80 km/h). The trailer administers a water spray to the pavement in front of the tire to simulate wet conditions. The resulting friction force acting between the test tire and the pavement surface is used to determine the skid resistance which is reported as a skid number (SN). Higher SN values signify higher skid resistance.

The Locked-Wheel Skid Trailer can only be used in the field, and for this reason other devices such as the Circular Track Meter (CTM) and the Dynamic Friction Tester (DFT) have been developed to evaluate texture and friction in the laboratory as well as in the field. The DFT is an apparatus that measures the friction-speed relationship on a pavement surface for speeds ranging from 0 to 80 km/h (micro-texture). The DFT measures the torque needed to stop three small spring-loaded standard rubber pads rotating in a circular path. The torque measured is then converted to a friction value. Water is also introduced during testing to simulate wet conditions. The CTM is a device that utilizes a displacement sensor that is mounted on an arm that rotates in a circular path and measures the mean profile depth (MPD) of a pavement (macro-texture). The CTM can be used in the field and laboratory to evaluate macro-texture.

Values obtained from the DFT and CTM can be used to compute an equivalent skid number (SN). The correlation between different texture and friction devices was established by the Permanent International Association of Road Congresses (PIARC) in 1992 [7]. PIARC developed the International Friction Index (IFI), which is an index for comparing and harmonizing friction measurements with different equipment to a common calibrated index. For example, to compute the equivalent skid number (SN) measured by a locked-wheel skid trailer at 50 mph using a smooth tire, the following equations can be used:

$$SN(50)_{smooth} = \left(\frac{F60-0.045}{0.925} \times \frac{1}{e^{\frac{20.47}{S_P}}}\right) \times 100 \qquad \text{(eq. 1)}$$

$$F60 = 0.081 + 0.732 \times DFT20 \times e^{-40/S_p} \qquad \text{(eq. 2)}$$

$$S_P = 14.2 + 89.7MPD \qquad \text{(eq. 3)}$$

where F60 and S_p are the International Friction Index (IFI) parameters, DFT20 is the coefficient of friction at 20 km/hr obtained from the DFT, MPD is the texture reading measured using the CTM, and $SN(50)_{smooth}$ is the calculated skid number at 50 mph using a smooth tire.

$SN(50)_{smooth}$ was calculated and compared for the field data only. For the laboratory testing the values of the DFT60 were compared. The reason this was done was because the goal of the lab test was to evaluate fine aggregates prone to polishing. The CTM measures macro-texture (wavelength of 0.02 in. to 2 in.), while the DFT evaluates micro-texture (wavelength < 0.02 in.). Since the texture created by the presence of fine aggregates fits more in the micro-texture range, the DFT values are able to better evaluate the polishing of fine aggregates. DFT60 was chosen instead of DFT20 because research done by the National Center for Asphalt Technology (NCAT) shows that DFT60 correlates well with locked wheel skid trailer values using ribbed tires (ASTM E 501) (Figure 1.2). Using ribbed tires in a skid trailer is a better way of evaluating micro-texture (smooth tire values represent the combined effect of micro-texture and macro-texture).

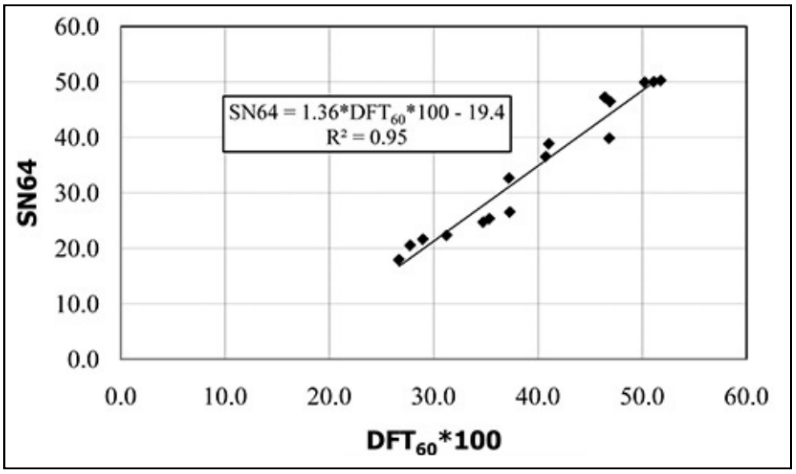

Figure 1.2: Correlation between **SN(64)$_{\text{ribbed}}$** and **DFT60** (metric units) [6]

1.4 Field Evaluation

Five sites in the Fort Worth area were evaluated (a total of twelve sections). The first was constructed in 2008 and consisted of four sections, three of which were made with 100% manufactured limestone aggregate having different microfine contents (aggregate passing the No. 200 sieve). The second consists of three sections constructed in 1995 using blends of sands that did not meet the 60% AI limit. The other three sites consist of pavement and bridge sections that were built using materials that meet the 60% AI limit. The reason sections made with materials meeting 60% AI were evaluated was to establish a better correlation between laboratory and field testing equipment (skid trailer, DFT, CTM).

1.4.1 Test Equipment Correlation

CTM, DFT, and skid trailer measurements were taken on twelve different PCC pavements sections to evaluate the correlation between different testing equipment (all twelve sections had a carpet drag and tined finish surface). Three to four CTM and DFT measurements were taken in the wheel path of each of the test sections; measurements were taken 50 to 200 feet apart, depending on how long each section was. A skid trailer equipped with smooth tires was then used to skid the same sections at 50 mph. Using equations 1, 2, and 3, the average equivalent skid numbers for each section was computed. Figure 1.3 shows a comparison between computed skid values and the actual skid values measured using a skid trailer.

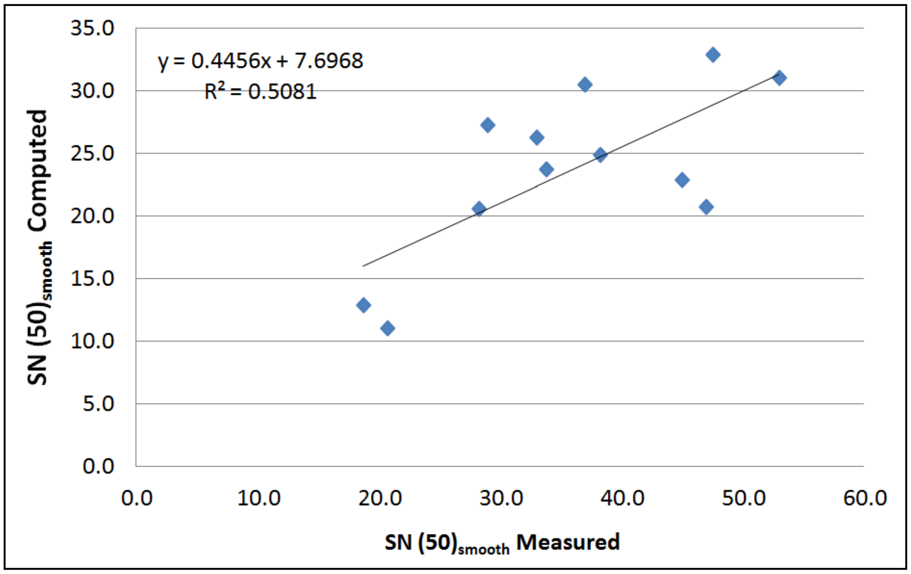

Figure 1.3: Computed vs. Measured SN(50)$_{smooth}$

Results from Figure 1.3 indicate that the IFI formula was not able to predict the measured skid number. Figure 1.4 shows that a better correlation was obtained when the measured skid trailer values were compared to friction values at 60 km/hr (DFT60).

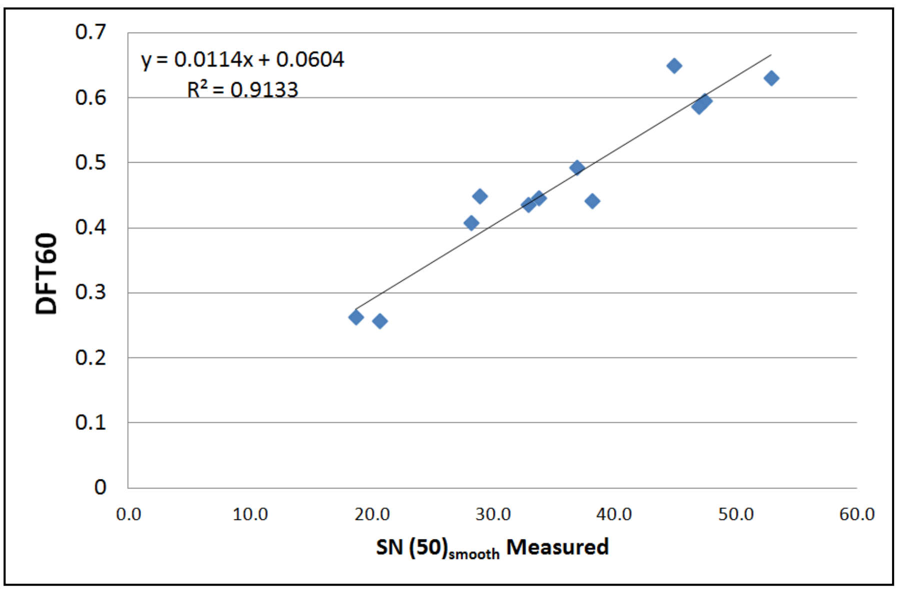

Figure 1.4: DFT60 vs. Measured SN(50)$_{smooth}$

Figure 1.5 shows that there is no correlation between SN(50)$_{smooth}$ and the MPD measured by the CTM. The poor correlation between SN(50)$_{smooth}$ and MPD, which is used to compute the IFI parameters, is the reason why a poor correlation between the measured and IFI computed SN(50)$_{smooth}$ exists.

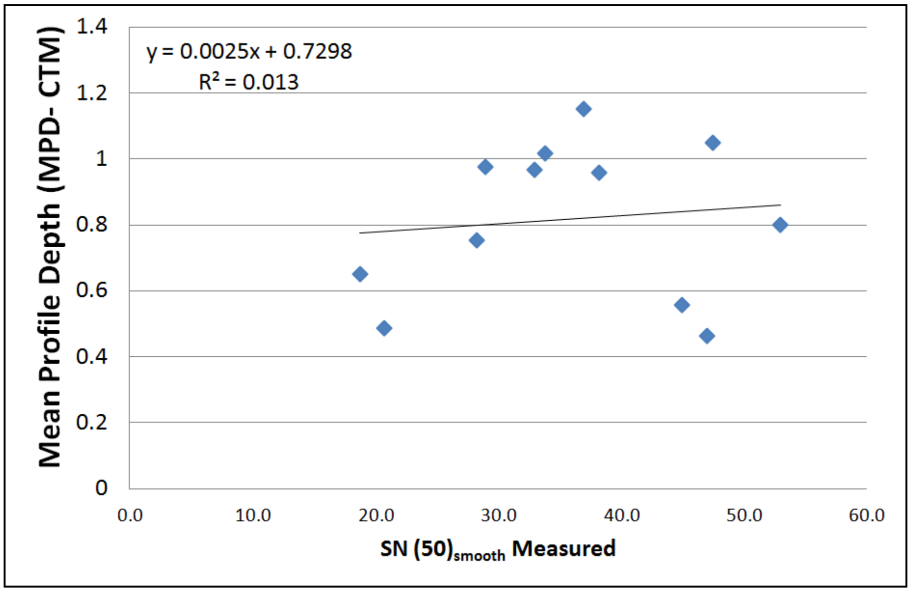

Figure 1.5: MPD vs. Measured $SN(50)_{smooth}$

Based on the results obtained in Figures 1.3, 1.4, and 1.5, the best method to compute $SN(50)_{smooth}$ using laboratory equipment is by using the following equation:

$$SN(50)_{smooth} = \frac{DFT60 - 0.0604}{0.0114} \qquad \text{(eq. 4)}$$

Where DFT60 is the average coefficient of friction at 60 km/hr measured using the DFT at different locations along a concrete section.

Note that equation 4 was established for concrete surfaces that have a mortar finish. It is not known whether or not equation 4 accurately predicts $SN(50)_{smooth}$ for other types of finishing such as diamond ground or exposed aggregate.

1.4.2 Fine Aggregate Type Comparison

Five field sections in two different locations in the Ft. Worth district were evaluated. Those sections were chosen because they were the only known sections in Texas that were made with materials that did not meet the TxDOT AI limit of 60%. The first location had two sections that were constructed with 100% limestone MFA, while the second location contained three sections made from three different blends of siliceous sand and limestone MFA. The difference between the two sections made with 100% MFA (AI ≈ 0%) was in gradation, not source; section 1 had 5% aggregates passing the No. 200 sieve (microfines) while section 2 had 10%. Those two sections were constructed in 2008 as part of an implementation project that involved using manufactured fine aggregates containing high microfines. The other three sections were constructed in 1995 using blends of sands that do not meet the 60% AI limit (AI of 29, 35, and 40%).

Abrasion/wear of concrete pavements is primarily caused by trucks and not by regular vehicles, which is the reason the 18-kip equivalent single axle load (ESAL) count should be used to compare wear instead of the average daily traffic (ADT) count. For both sections the ESAL count was obtained for the year 2011. ESAL counts for other years could not be obtained, for this reason the ESAL count for previous years will be assumed to be equal to that of 2011. The 100% MFA sections were constructed in the outside lane, while the blended sand sections were constructed in the inside lane. The outside lane receives most of the heavy trucks; for this reason (and based on AASHTO design recommendations) the inside lanes were assumed to receive 20% of the ESAL count, while the outside lanes were assumed to receive the other 80%.

Results shown in Figure 1.6 were computed using equation 4, and they represent the average of three measurements taken on the wheel path of each of the sections evaluated. Although the estimated ESAL count for the blended sections is twice that of the 100% MFA sections, the skid values obtained for the 100% manufactured limestone sections are around half those of the blended sand sections. The blended sand section with the highest siliceous sand content (or highest AI content) had the highest skid value. Moreover, Figure 1.6 shows that even when only 40% siliceous sand is used (AI ≈ 29%), good skid can be achieved.

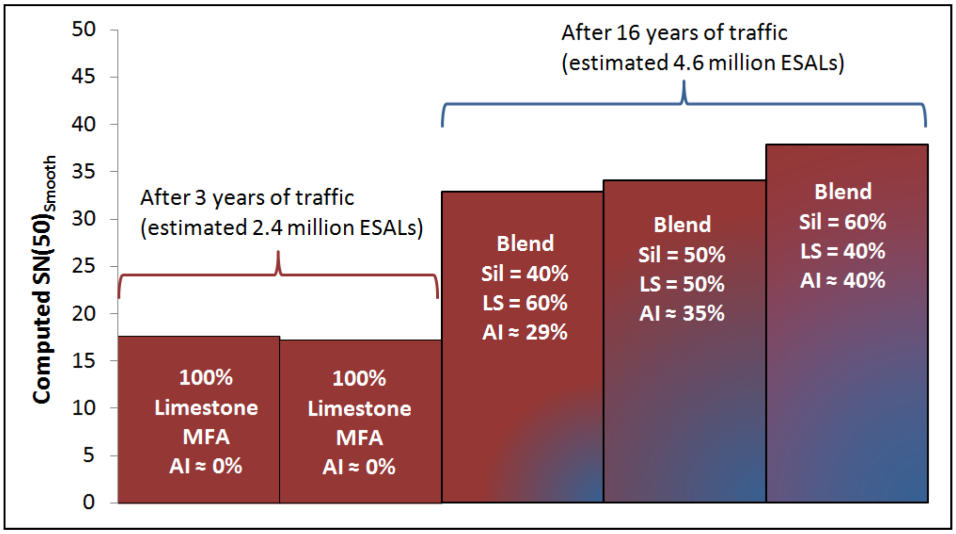

Figure 1.6: MFA vs. Blended Sands Skid Performance

Using a 100% manufactured limestone fine aggregate likely resulted in more loss of skid resistance than when some siliceous sand was present. Blending a limestone aggregate with a small percentage of siliceous fine aggregate can have a high impact on skid performance. Skid performance seems to increase as a result of using blends of aggregates with higher siliceous content.

1.5 Laboratory Testing

The goal of the laboratory testing was to evaluate the polish resistance of concrete slabs made with different fine aggregates and to relate those results to aggregate tests. The CTM and DFT were used along with a three-wheel polishing device (TWPD) to evaluate the polish resistance of a laboratory concrete specimen. The TWPD simulates the wear caused by traffic. The wheels used on the TWPD were hard polyurethane casters loaded to exert an average stress of 50-60 psi on the concrete specimen. For each sand or blend of sands two slabs were tested. The volumetric mixture proportions for all tested specimens were the same. Note that the effect of changing mixture proportions was also evaluated but the results showed that changing proportions had little or no effect on friction.

In Figure 1.7, results of DFT60 after 160,000 polishing cycles on concrete specimens are compared to the AI values of aggregate used. Figure 1.7 shows that some of the carbonate aggregates that had low AI performed as well as the aggregates that had a high AI. There does seem to be a relation between AI and the performance of siliceous and blended aggregates; as the AI decreases, DFT60 values after 160,000 cycles decrease for siliceous and blended aggregates. The relation between AI and DFT60 values for carbonate aggregates (limestone or dolomite) is not clear. A few of the aggregates with AI < 60 % maintained a relatively high DFT60 value after 160,000 polishing cycles.

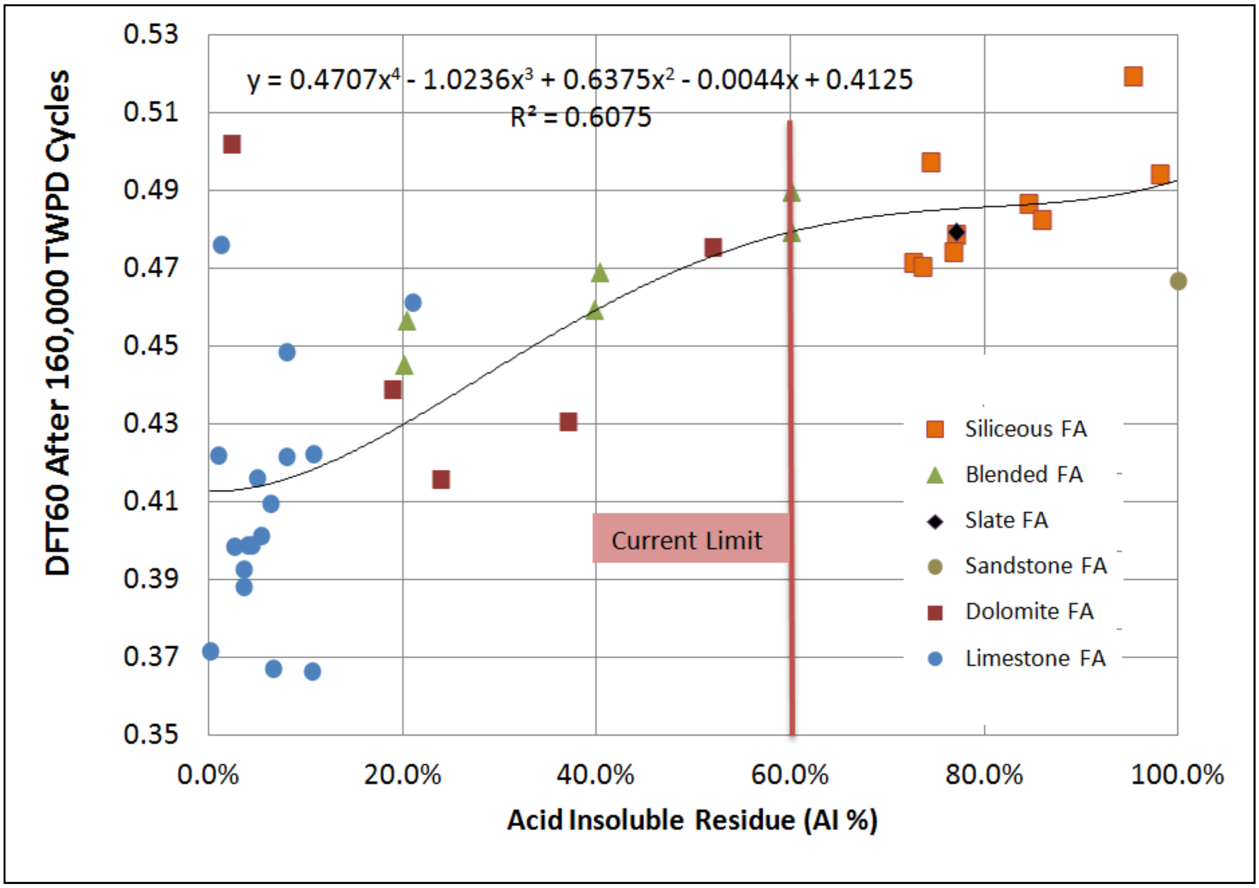

Figure 1.7: DFT60 after 160,000 TWPD cycles vs. AI

An alternative way of evaluating aggregates for polish resistance was considered for the laboratory testing. Fine aggregates were tested using the MD test (ASTM D 7428). Values of AI and MD are compared in Figure 1.8.

There is good correlation between the AI test and the MD test for aggregates that have an MD less than 24%. AI does not differentiate between hard and soft carbonates, because it is a chemical test and not a mechanical test. Except for dolomites and dolimitic limestones, carbonates are generally softer than other aggregates used in concrete, and as a result AI is a very conservative test that disqualifies all carbonates regardless of their hardness. Except for two dolomites and one sandstone, most fine aggregates that met the AI requirement of 60% also met micro-Deval limit of 12% (intersecting red lines in Figure 1.8).

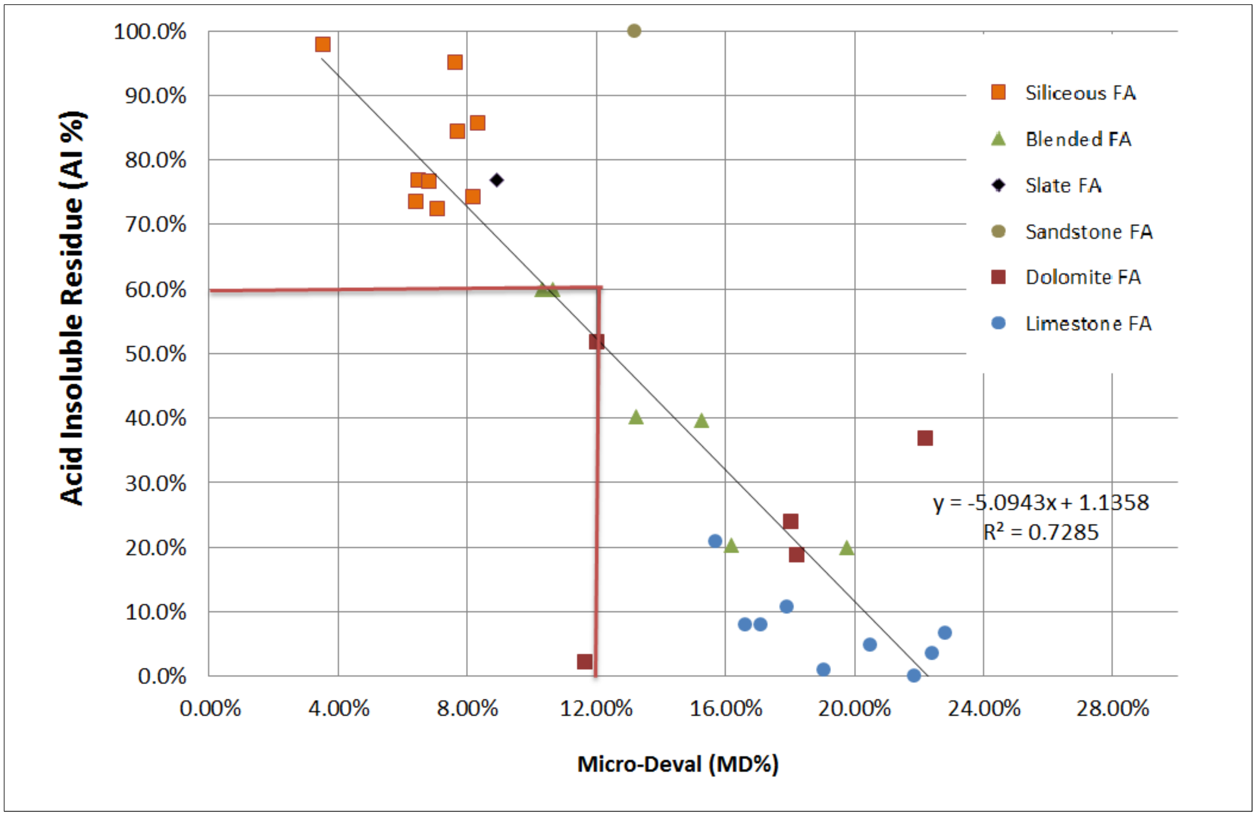

Figure 1.8: AI vs. MD

It should be noted that the AI test is only a surrogate test for evaluating the polish resistance of fine aggregates in PCCP, and the test was originally developed based on an observation that an increase in non-carbonate content improves skid resistance (Balmer and Colley). The concrete test results obtained by Balmer and Colley in 1966 also seem to indicate that dolomites perform better than limestone fine aggregates (note limestones are referred to as calcites in this paper – Figure 1.9)

Fine Aggregate No.	Principal Constituents, %	Rating of Field Performance	Wear Index, kw
1......	90 calcite	poor	3.6
2......	70 calcite 24 dolomite	poor	4.4
3......	90 calcite	poor	4.0
4......	80 dolomite	poor	5.6
5......	75 dolomite	poor	5.8
6......	70 dolomite	poor	5.4
7......	60 calcite 16 silt and clay	poor	5.3
8......	80 calcite 15 quartz	fair	6.2
9......	65 calcite 12 dolomite	poor	5.9
10......	50 calcite 33 mica and quartz	excellent	6.8
11......	55 dolomite 39 quartz, quartzite, and feldspar	excellent	6.8
12......	55 calcite and dolomite 40 quartz, mica, and epidote	excellent	6.7
13......	45 calcite 42 quartz and feldspar	excellent	7.3
14......	50 dolomite 44 quartz	excellent	7.0
15......	45 dolomite 45 quartz	excellent	6.8
16......	45 dolomite 45 quartz	excellent	7.2
17......	30 graywacke 55 quartz	excellent	7.0
18......	75 quartz 17 feldspar	excellent	7.3
19......	72 quartz 20 feldspar	excellent	7.2
20......	99 quartz	excellent	7.5

Figure 1.9: Wear Index obtained for Different Mineralogy (Balmer and Colley, 1966)

Figure 1.10 shows a linear relationship between DFT60 after 160,000 TWPD cycles and MD for aggregates having an MD value between 12% and 24%. According to Hudec and Boateng (1995), high MD loss values indicate the presence of shale or chert. This might explain why some aggregates had a high micro-Deval loss but still performed well. DFT60 values increase as the MD percent loss decreases for aggregates that have an MD loss lower than 24%; aggregates with an MD loss higher than 24% or lower than 12% do not seem to follow that trend.

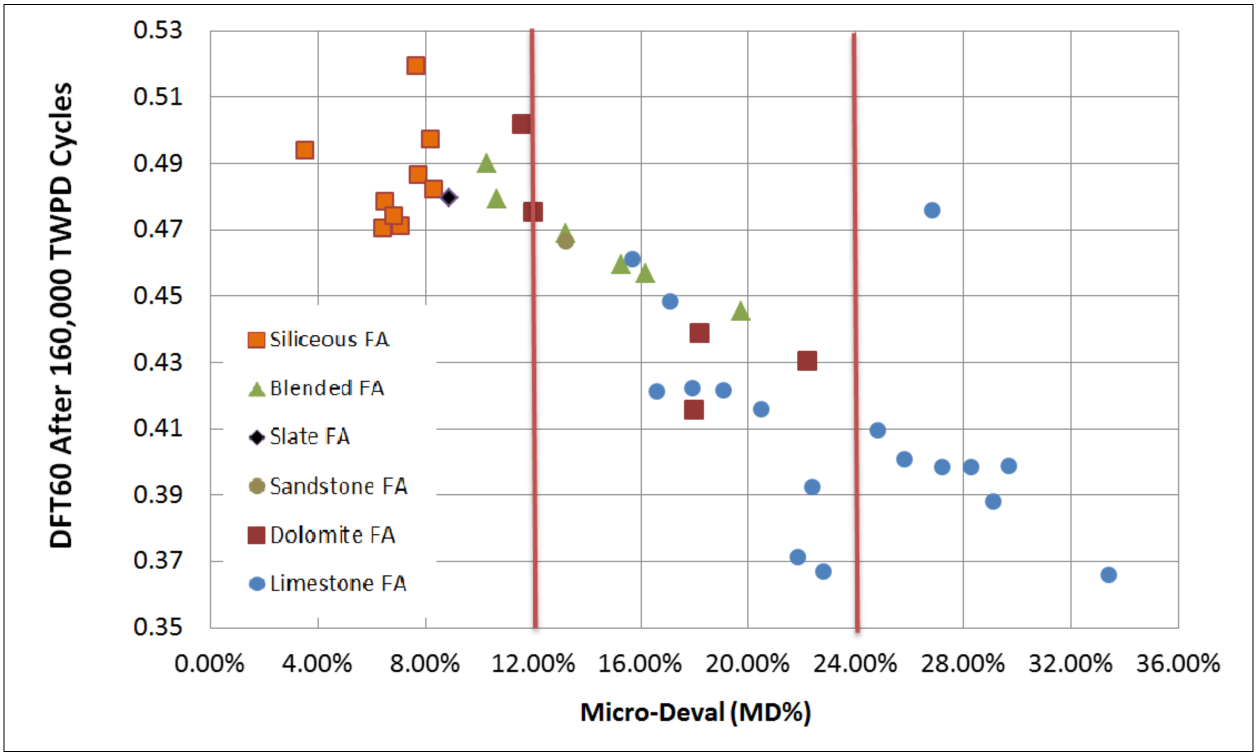

Figure 1.10: DFT60 after 160,000 TWPD cycles vs. MD

1.6 Recommendations

The following method is recommended as an alternative preliminary procedure for accepting and blending aggregates for class P concrete:

1) Test unblended fine aggregate(s) using Tex-612-J (acid insoluble residue).
   a) If AI ≥ 60%, no need for further testing of fine aggregates for polish resistance.
   b) If AI < 60%, further testing of fine aggregates is needed.
2) Test fine aggregates using the micro-Deval (MD) test (ASTM D 7428).
   a) If the micro-Deval percent loss of a fine aggregate is less than 12% (MD < 12%), blend this fine aggregate with at least 40% of a fine aggregate that has an AI ≥ 60%.
   b) If the micro-Deval percent loss of a fine aggregate is greater than 12% (MD ≥ 12%), then blend this fine aggregate such that the equivalent micro-Deval percent loss of the combined fine aggregate is less than 12% (MD < 12%):

$$[(\%Agg1) \times (\%loss\ of\ Agg1)] \times [(\%Agg2) \times (\%loss\ of\ Agg2)] < 12\%$$

Note that all aggregates have to be tested prior to blending. Aggregate test values obtained from testing blended fine aggregates using Tex-612-J and ASTM D 7428 should not be used to identify polish resistant aggregates in PCCP.

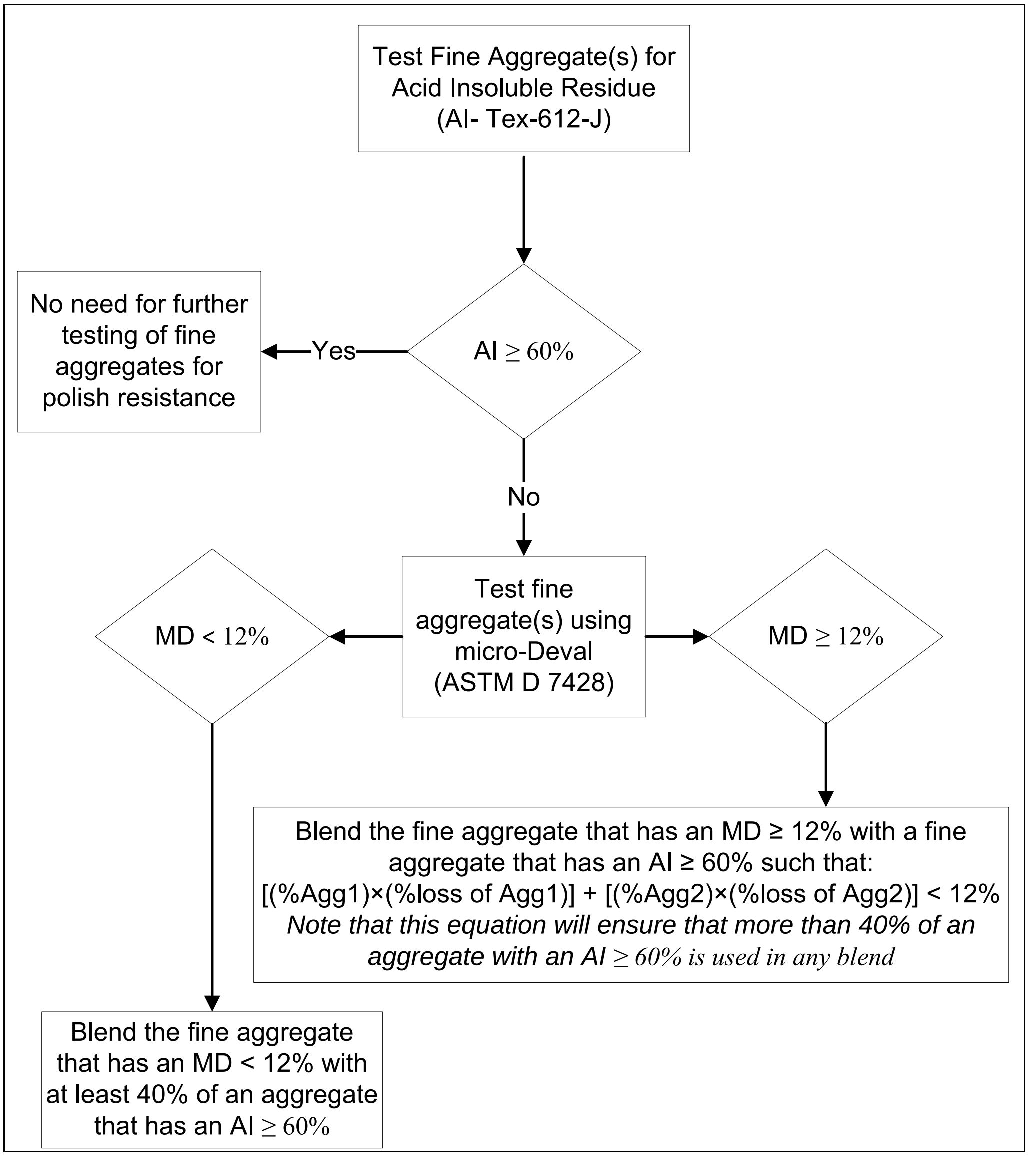

Figure 1.11: Testing Polish Resistance of Fine Aggregates in PCCP

If this method of blending is used instead of the current specifications, then more manufactured carbonate sand will be allowed in pavements if the manufactured sand itself is hard, or if it is blended with harder siliceous sands (hardness is evaluated by the MD test).

If blends of the siliceous and limestone aggregate tested during this research project were to be blended to meet a MD loss of less 12%, then the minimum AI that can be obtained from such blends will be greater than 40% (Figure 1.12).

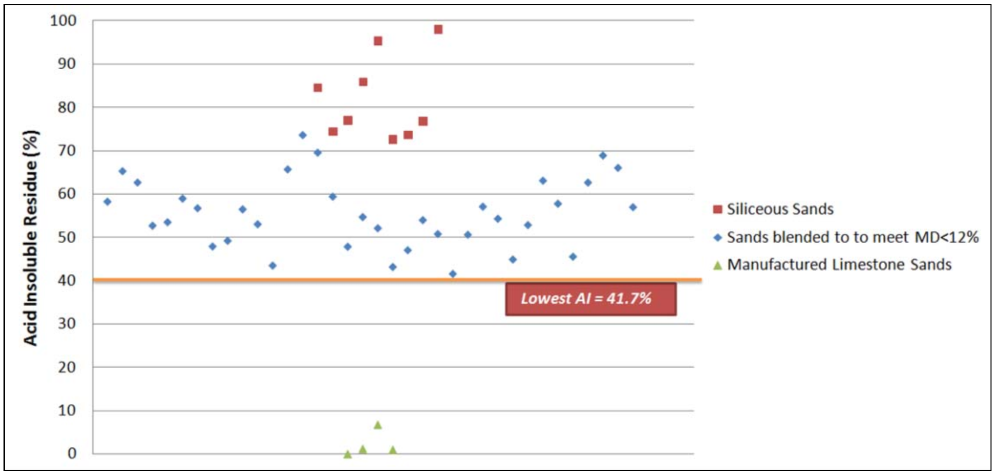

Figure 1.12: AI Values for Blends of Aggregates Meeting the 12% MD Limit

Chapter 2. Establishing a Friction Prediction Model for PCC Pavements

2.1 Introduction

A model for predicting skid values for concrete pavements is presented in this chapter. The model was derived using data obtained from the monitored field sections as well as the friction testing conducted at the laboratory. Since all the sections evaluated had a tined and carpet drag finish, the model derived is only suitable for concrete pavements that have a mortar surface exposed to traffic (the effect of exposed coarse aggregate is not accounted for in this model).

Among the twelve sections evaluated, only four sections have been monitored for an extended period of time. Five of the twelve sections were recently constructed and have not been exposed to a significant amount of traffic; for this reason, data obtained from those sections was not adequate for computing the model. The three sections constructed in 1995 were only used to verify the model; although friction measurements were taken between wheel paths to estimate initial conditions of the sections, it is not clear whether or not those measurements represent the initial skid resistance of the pavement.

2.2 Field Data Analysis

The test sections constructed in 2008 were monitored for three years. CTM and DFT measurements were taken during four visits to the site and the values were converted using equation 4 (presented in Chapter 1). Measurements taken between the wheel paths are generally not exposed to traffic and were assumed to represent the initial conditions of the pavement (June, 2008). The data obtained are shown in Figure 2.1.

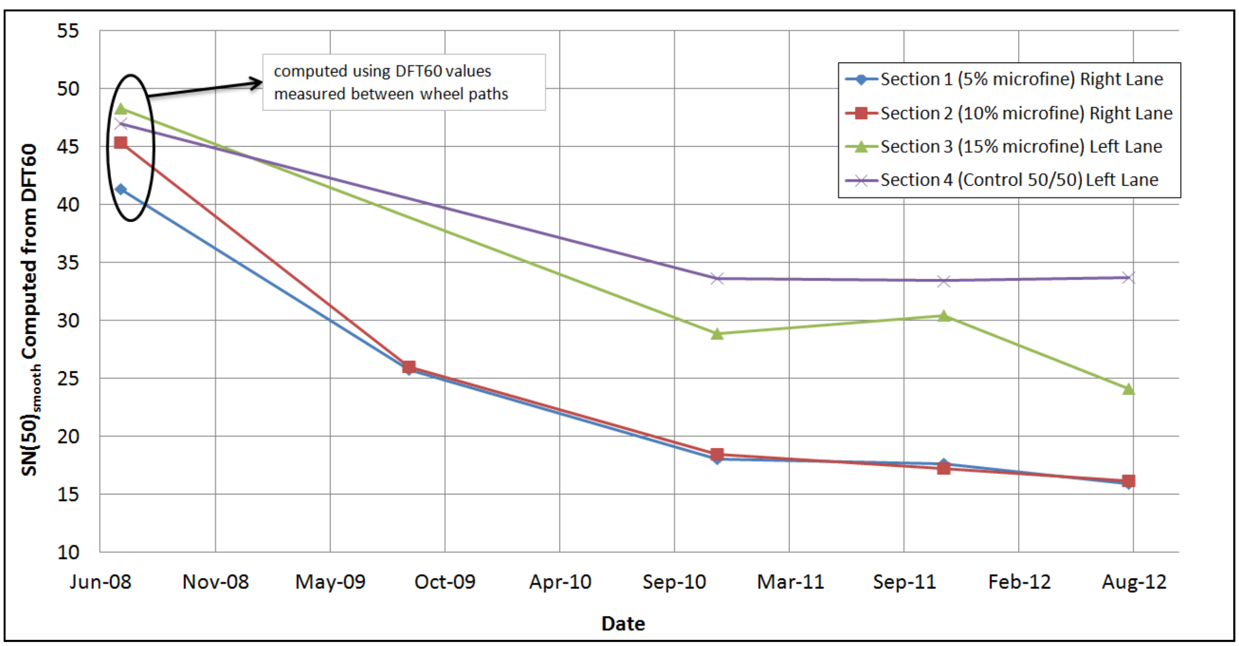

Figure 2.1: Computed Skid Numbers for Trial Field Sections as a Function of time

Sections 1 and 2 were constructed using 100% manufactured limestone aggregate on the outside lane. Section 1 had 5% microfine content while section 2 had 10%. Section 3 was constructed on the inside lane using 100% carbonate aggregate but had 15% microfines (same fine aggregate source as sections 1 and 2). Section 4, the control section on the inside lane, which had a blend of 50/50 is estimated to have an AI of 40%, while the AI of the other three sections can be assumed to be equal to 0%. The manufactured sand used in all the test sections was TXI Bridgeport.

Figure 2.1 shows that a change in microfine content (gradation change) did not have any effect on skid resistance. Even though section 2 had slightly higher initial skid values, sections 1 and 2 had very similar skid values after more than 1 year of traffic. Results obtained from Figure 2.1 also show that trucks have a higher wearing and polishing effect on pavements. The inside lane and the outside lane might have the same average daily traffic (ADT) count but they almost assuredly have different ESAL counts. If the ESAL count is assumed to be the controlling factor in wear and if it is assumed that section 3 had a wear factor similar to sections 1 and 2, then the ESAL distribution between the two lanes can be estimated by matching the wear rate of section 3 with sections 1 and 2. Figure 2.2 shows that a good fit can be obtained if a 77.5/22.5 split is assumed between the outside lane and the inside lane (this is very close to the 80/20 split assumed in Chapter 1).

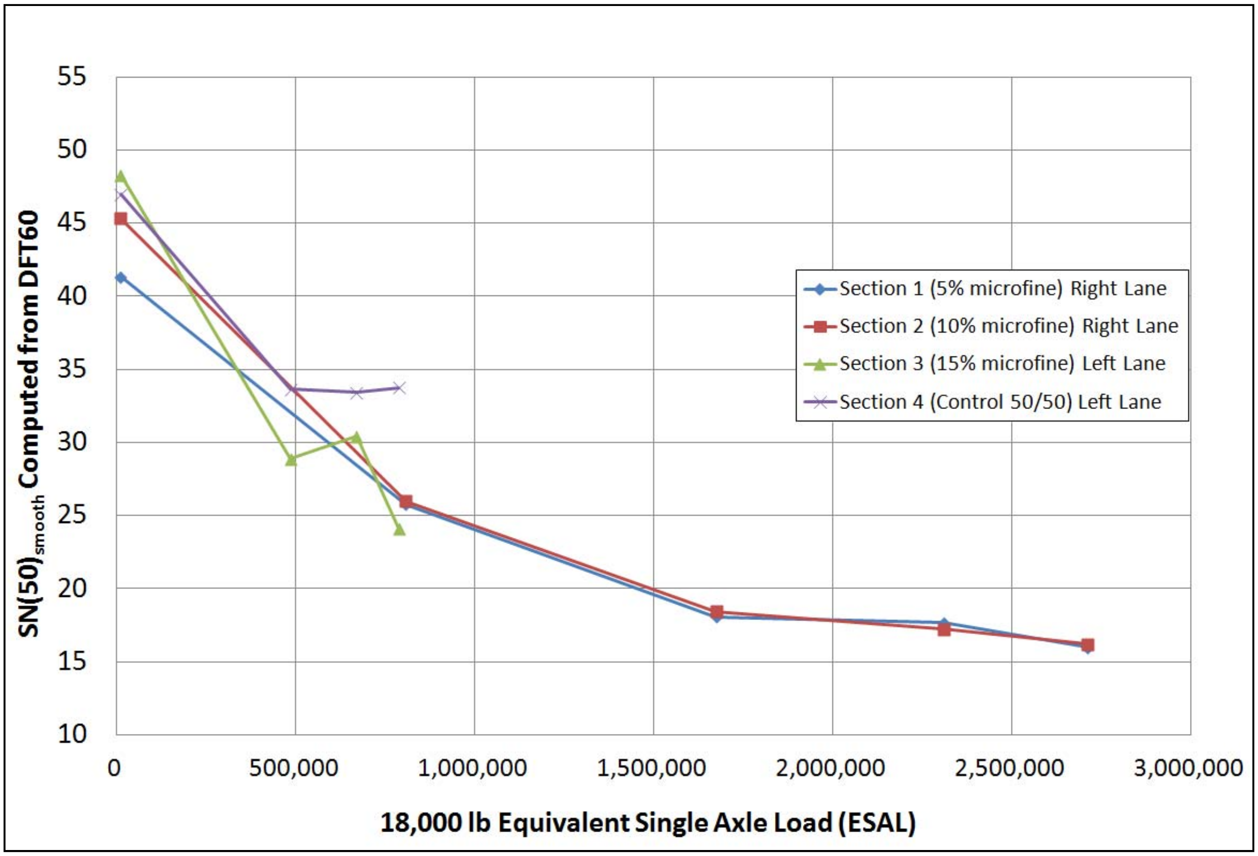

Figure 2.2: Computed Skid Numbers for Trial Field Sections as a Function of ESALs

2.3 Laboratory data Analysis

2.3.1 Effect of mixture variables on surface friction

The fine aggregate used in the test sections constructed in 2008 was obtained from the TXI Bridgeport quarry. Figure 2.3 shows the results of three micro-Deval coarse aggregate tests for TXI Bridgeport coarse aggregate samples obtained at different times in other research studies. The results show that the three samples obtained from the quarry in 2008, 2009, and 2012 did not vary significantly in hardness. In summer 2010, a sample of fine aggregate from TXI Bridgeport was obtained and tested using the TWPD, CTM, and DFT. The sample tested at the laboratory was obtained from the quarry on a different date than the date the test sections were constructed, but because the hardness of the aggregate has not significantly changed, the fine aggregate used in the field sections and the laboratory tests were assumed to be identical.

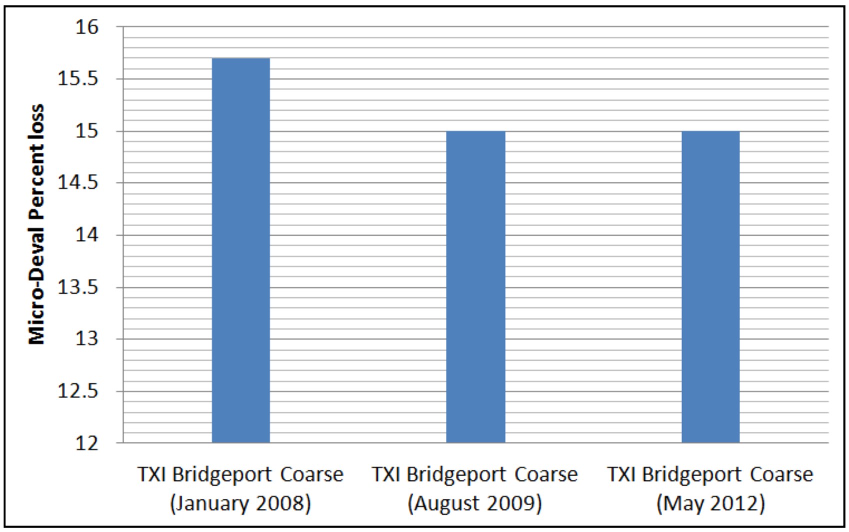

Figure 2.3: Relationship between DFT60 after 160,000 TWPD cycles and AI

When the test sections containing 100% MFA were constructed, finishing problems were encountered. The concrete received on site did not meet slump requirements and the finishing crew had trouble finishing it. To be able to finish the concrete, the finishing crew sprayed the surface with water and worked the water into the paste to make the surface more finishable (Figure 2.4).

Figure 2.4: Finishing Crew Spray the surface of the 100% MFA section

Figure 2.5 illustrates the difference in macro-texture for the surface of section 1 between wheel paths and on the wheel path. When the finishing crew sprayed the concrete surface with water, they were able to finish it and create the required macro-texture, but because water was added to the paste and increased the water-cement ratio, the finished surface was not durable and that is it the reason it polished at a greater rate.

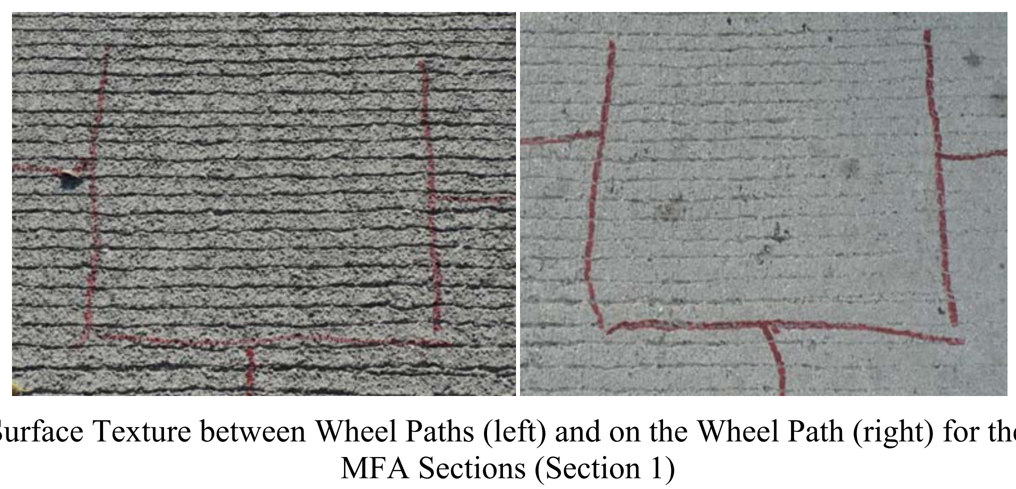

Figure 2.5: Surface Texture between Wheel Paths (left) and on the Wheel Path (right) for the 100% MFA Sections (Section 1)

As described in Chapter 1, the CTM, DFT, and TWPD were used to evaluate fine aggregates for polish resistance. Several test slabs were made with TXI Bridgeport fine aggregate; In addition to the baseline (control) mixture, several variables were used:

- Use of fly ash;
- Low and high sand content;
- Low and high paste content; and
- Two amounts of water added to the surface.

Table 2.1 shows the results obtained. Slabs that started with a higher DFT60 value did not necessarily end up with a higher DFT60 value after 160,000 TWPD cycles. Moreover, it has been found earlier in this research that the type of finishing applied on the mortar, whether it is a carpet, turf, or broom finish does not have a significant effect on the DFT60 value after 160,000 polishing cycle. These results proved that changing mixture proportions had only a minor effect on the DFT60 value after 160,000 polishing cycles, but adding water to surface and working it into the paste during finishing caused the DFT60 value to drop by around 10%. Although it is unknown how much water was added to the surface of the field sections, the samples made at the laboratory with the addition of water on the surface better represent the field test sections.

Table 2.1: DFT60 Results for Concrete Slabs made with TXI Bridgeport FA

TXI Bridgeport Mixtures	DFT60 initial values (0 cycles)	DFT60 after 160,000 Cycles
Baseline	0.676	0.422
Baseline (30% fly ash)	0.753	0.409
Low sand content	0.629	0.414
High sand content	0.718	0.421
Low paste content	0.694	0.433
High paste content	0.750	0.429
Average	0.703	0.421
Baseline + water added to surface (11 oz/yd^2)	0.687	0.39
Baseline + water added to surface (22 oz/yd^2)	0.836	0.373
Average of Samples with added water to the surface	0.761	0.3815

2.3.2 Relationship between DFT and MD

In Figure 1.10, the values of DFT60 after 160,000 polishing cycles were compared to the micro-Deval percent loss of the fine aggregates. Fine aggregate with a micro-Deval loss higher than 24% or lower than 12% did not follow the trend of decrease in performance with an increase in loss; these limestone fine aggregates might have had materials such as chert that caused them to experience higher loss in micro-Deval compared to other limestone fine aggregates. For this reason they were not used to obtain the relationship shown in Figure 2.6.

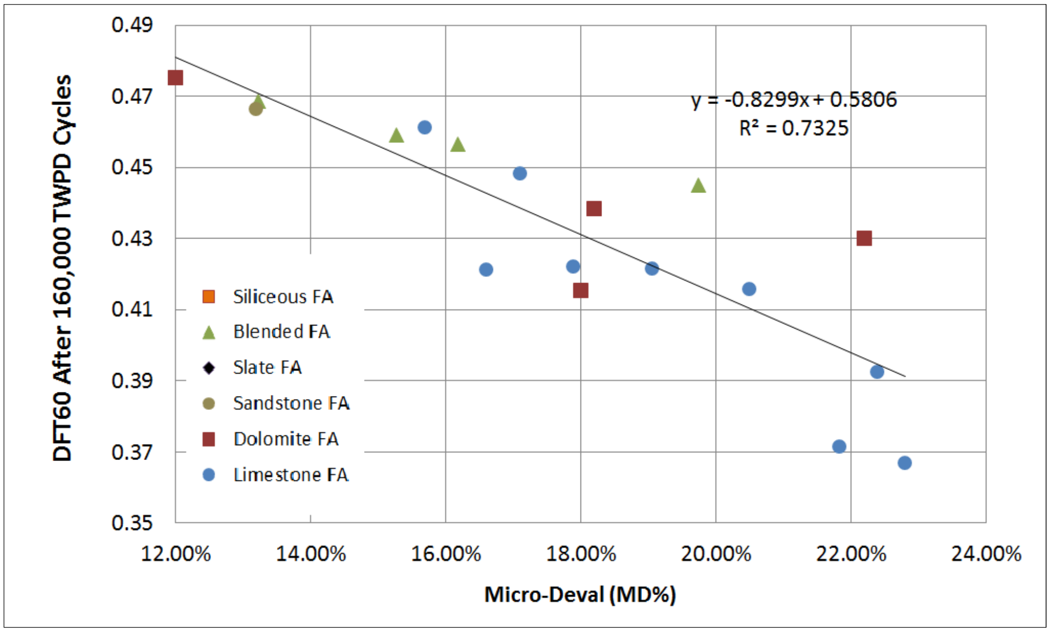

Figure 2.6: Relationship between DFT60 after 160,000 TWPD cycles and MD

DFT60 at a 160,000 cycles can be estimated from MD using the following equation:

$$DFT60 = -0.83 \times MD_{eq} + 0.58 \qquad \text{for } MD_{eq} \geq 12 \qquad \text{(eq. 5)}$$

$$MD_{eq} = (\%\ aggregate\ 1\ \times MD_{aggregate\ 1}) + (\%\ aggregate\ 2\ \times MD_{aggregate\ 2}) \qquad \text{(eq. 6)}$$

MD_{eq} is the equivalent micro-Deval percent loss of two or more fine aggregates. MD_{eq}, $MD_{aggregate\ 1}$, and $MD_{aggregate\ 2}$ are in percent for the equations presented, i.e. if MD_{eq} was equal to 12%, use $MD_{eq} = \frac{12}{100} = 0.12$ in equation 5. Note that for $MD_{eq} < 12$, $DFT60 = 0.485$.

2.3.3 Relationship between DFT and AI

DFT60 after 160,000 polishing cycles was compared to the fine aggregates AI values in Figure 1.7. AI evaluates the carbonate content of a fine aggregate and this was the reason the AI values obtained for the dolomitic fine aggregates did not correlate with DFT60. To compute a better relationship between AI and DFT60 at 160,000 cycles, all the dolomites, dolomitic limestones, and carbonates that had DFT60 values comparable to siliceous and blended sands were not considered in Figure 2.7.

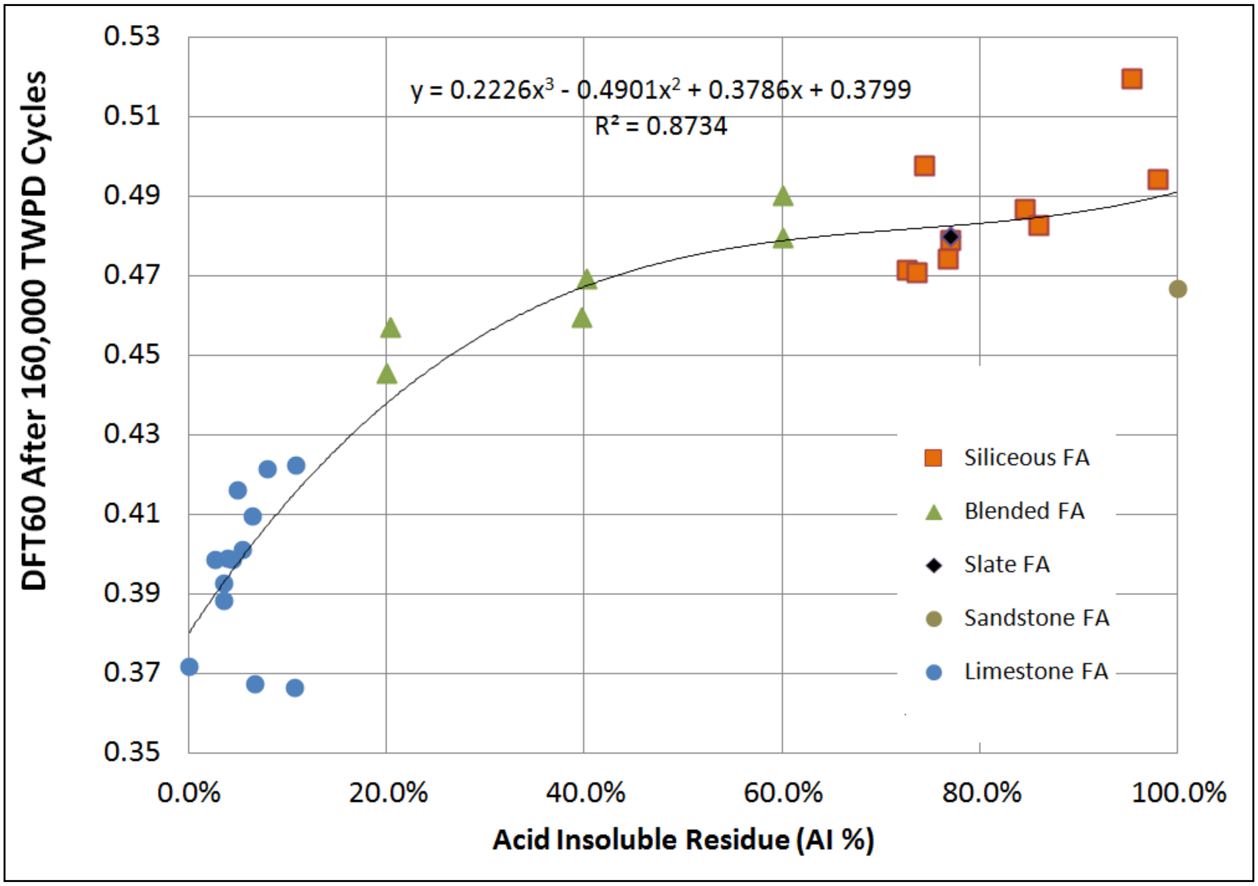

Figure 2.7: Relationship between DFT60 after 160,000 TWPD cycles and AI

DFT60 at a 160,000 cycles can be estimated from AI using the following equation:

$$DFT60 = 0.3799 + (0.3786 \times AI_{eq}) - \left(0.4901 \times AI_{eq}^{2}\right) + (0.2226 \times AI_{eq}^{3}) \qquad \text{(eq. 7)}$$

$$AI_{eq} = \left(\%\, aggregate\ 1\ \times AI_{aggregate\ 1}\right) + \left(\%\, aggregate\ 2\ \times AI_{aggregate\ 2}\right) \qquad \text{(eq. 8)}$$

AI_{eq} is the equivalent acid insoluble residue of two or more fine aggregates. AI_{eq}, $AI_{aggregate\ 1}$, and $AI_{aggregate\ 2}$ are in percent for the equations presented, i.e. if AI_{eq} was equal to 60%, use a value of $AI_{eq} = \frac{60}{100} = 0.6$ in equation 7.

2.4 Prediction Model for Computing SN(50)$_{smooth}$

Figures 2.8, 2.9, and 2.10 represent a wear model for the four different sections tested. Sections 1 and 2 have very similar values and can be represented in one equation. Section 3 can be represented in a model very similar to 1 and 2. Comparing the laboratory test for the slabs made with TXI Bridgeport where water was added to surface and the field results in Figures 2.8 and 2.9, show that 160,000 TWPD cycles are equivalent to around 700,000 ESALs.

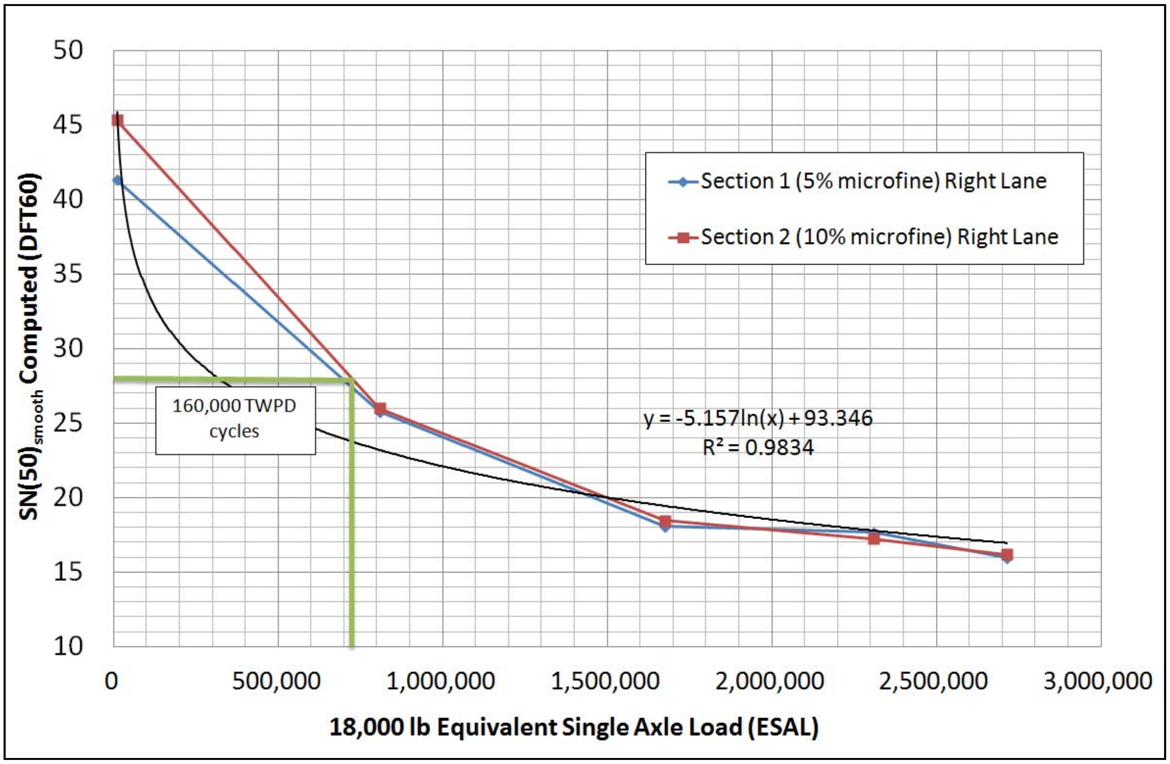

Figure 2.8: Computed Skid Numbers as a Function of ESALs (Sections 1 and 2)

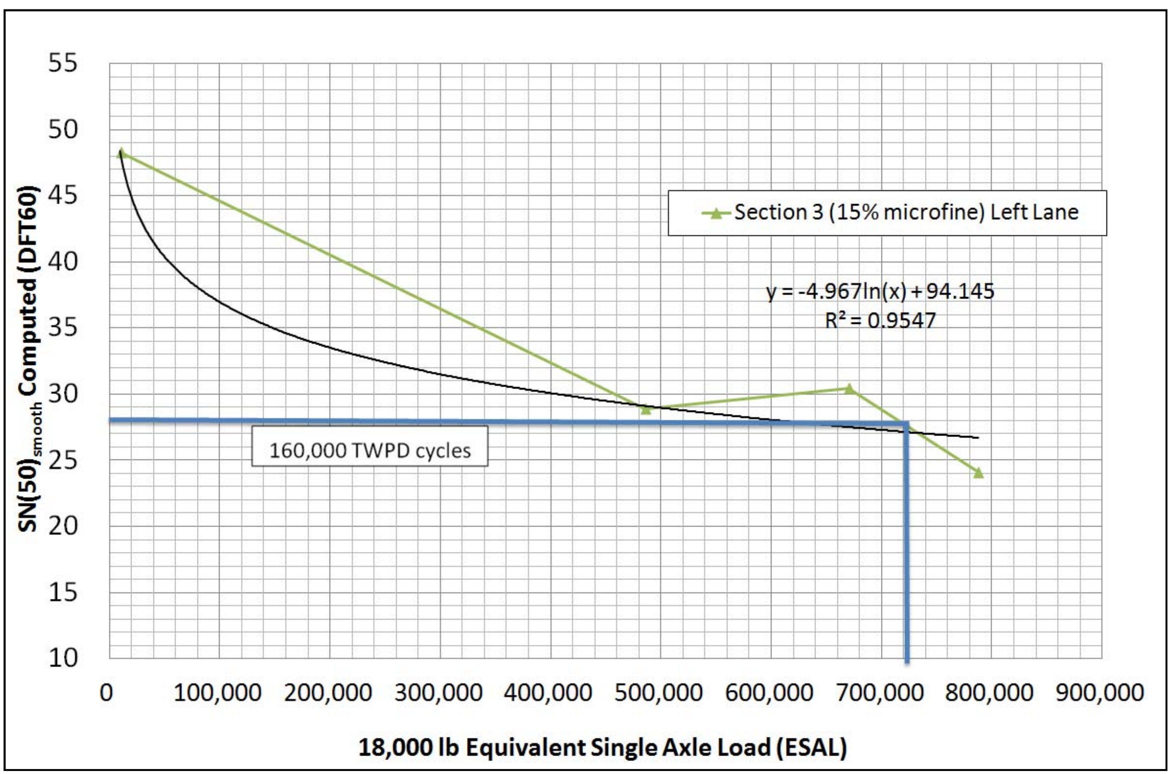

Figure 2.9: Computed Skid Numbers as a Function of ESALs (Section 3)

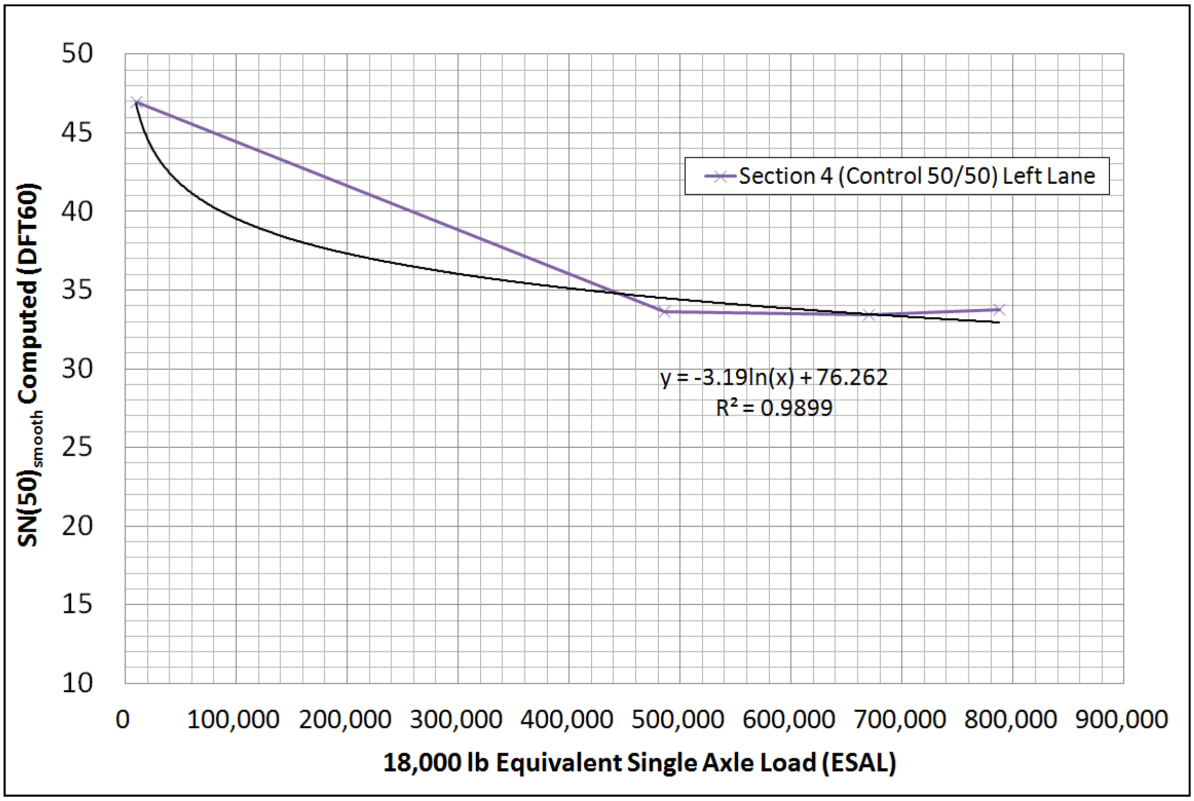

Figure 2.10: Computed Skid Numbers as a Function of ESALs (Section 4)

The equations from Figure 2.8 and Figure 2.9 can be combined and generalized for any sand by multiplying the slope of the log function by the DFT60 value of the laboratory slab that resembles the field test section and then dividing it by $\alpha \times DFT60_{160{,}000\ cycles}$

$$SN(50)_{smooth} = 93.75 - \frac{5.062 \times 0.381}{\alpha \times DFT60_{160{,}000\ cycles}} \ln(ESAL_{Total}) \qquad \text{(eq. 9)}$$

where α is a factor that accounts for poor finishing techniques; this includes spraying the concrete surface with water and working the water in the surface and/or poor application of drag finishing techniques (burlap, turf, or broom). It was shown in Table 2.1 that poor finishing techniques resulted in a reduction of friction after 160,000 cycles of about 10%. Unless the surface was poorly finished $\alpha = 1.0$; for poorly finished surfaces is assumed to be $\alpha = 0.9$. $ESAL_{Total}$ is total ESAL count that the lane has experienced while in service.

Combining eq. 5 and eq. 7 with eq. 9 will lead to the following:

$$SN(50)_{smooth} = 93.75 - \left\{\frac{1.93}{\alpha \times (-0.83 \times MD_{eq} + 0.58)}\right\} \times \ln(ESAL_{Total}) \qquad \text{(eq. 10)}$$

$$SN(50)_{smooth} = 93.75 - \left[\frac{1.93}{\alpha \times \left[0.3799 + (0.3786 \times AI_{eq}) - (0.4901 \times AI_{eq}^2) + (0.2226 \times AI_{eq}^3)\right]}\right] \times \ln(ESAL_{Total}) \qquad \text{(eq. 11)}$$

MD_{eq} is the equivalent micro-Deval percent loss of two or more fine aggregates. MD_{eq} is in percent for the equations presented, i.e. if MD_{eq} was equal to 12%, use a value of $MD_{eq} = \frac{12}{100} = 0.12$.
AI_{eq} is the equivalent acid insoluble residue of two or more fine aggregates. AI_{eq} is in percent for the equations presented, i.e. if AI_{eq} was equal to 60%, use a value of $AI_{eq} = \frac{60}{100} = 0.6$.

Equations 10 and 11 can be used to predict the skid number of any mortar finished pavement by knowing either the AI or MD and the total ESAL count for the lane. Note that neither equation presented takes into account the presence of exposed coarse aggregates, whether those were intentionally or unintentionally exposed.

2.5 Verification of Prediction Model and Development of Design Charts

Equation 11 was used to estimate the skid number of four of the sections evaluated. The results are shown in Table 2.2. Note that the exact ESAL count per lane is not known and was assumed to be 20% for sections in location 2. For location 1, 22.5% deduced in Figure 2.2 by matching wear rate between sections 1, 2, and 3. The values predicted in Table 2.2 were not exact for all the sections but were still able to give a conservative estimate of what skid numbers to expect for the different blends.

Table 2.2: Verification of the Prediction Model

Test Section	AI (%)	Measured $SN(50)_{smooth}$	Estimated Total ESALs at time of Skid Test	Estimated Using $SN(50)_{smooth}$ Model
Location 1 - Section 4 (Control 50/50)	40	38.3	670,000	38.3
Location 2 - Blend 60/40	40	37.0	4,600,000	30.4
Location 2 - Blend 50/50	35	29.0	4,600,000	29.6
Location 2 - Blend 40/60	29	33.0	4,600,000	28.5

Equations 10 and 11 can also be used to calculate the estimated number of years to reach a target skid number as a function of the number of daily lane ESALs and either the MD or AI of the fine aggregate. Tables 2.3, 2.4, 2.5, and 2.6 were developed assuming that the target skid number was $SN(50)_{smooth} = 25$. Note that the ESAL values shown in the table are ESALs per lane per day, i.e. the ESAL distribution per lane should be computed before using the tables.

Table 2.3: Design Chart Assuming $\boldsymbol{SN(50)_{smooth}} = \mathbf{25}$, well finished, and using AI

AI (%) / ESALs/lane (per day)	Estimated years to reach SN(50)smooth										
	1%	10%	20%	30%	40%	50%	60%	70%	80%	90%	100%
500	5	13	32	61	92	120	139	152	163	180	216
1000	2	7	16	30	46	60	70	76	82	90	108
1500	2	4	11	20	31	40	46	51	54	60	72
2000	1	3	8	15	23	30	35	38	41	45	54
2500	1	3	6	12	18	24	28	30	33	36	43
3000	1	2	5	10	15	20	23	25	27	30	36
3500	1	2	5	9	13	17	20	22	23	26	31
4000	1	2	4	8	12	15	17	19	20	23	27
4500	1	1	4	7	10	13	15	17	18	20	24
5000	0	1	3	6	9	12	14	15	16	18	22
5500	0	1	3	6	8	11	13	14	15	16	20
6000	0	1	3	5	8	10	12	13	14	15	18
6500	0	1	2	5	7	9	11	12	13	14	17

Table 2.4: Design Chart Assuming $\boldsymbol{SN(50)_{smooth}} = \mathbf{25}$, poorly finished, and using AI

AI (%) / ESALs/lane (per day)	Estimated years to reach SN(50)smooth										
	1%	10%	20%	30%	40%	50%	60%	70%	80%	90%	100%
500	1	3	7	12	18	22	25	27	29	32	38
1000	1	2	3	6	9	11	13	14	15	16	19
1500	0	1	2	4	6	7	8	9	10	11	13
2000	0	1	2	3	4	6	6	7	7	8	9
2500	0	1	1	2	4	4	5	5	6	6	8
3000	0	1	1	2	3	4	4	5	5	5	6
3500	0	0	1	2	3	3	4	4	4	5	5
4000	0	0	1	1	2	3	3	3	4	4	5
4500	0	0	1	1	2	2	3	3	3	4	4
5000	0	0	1	1	2	2	3	3	3	3	4
5500	0	0	1	1	2	2	2	2	3	3	3
6000	0	0	1	1	1	2	2	2	2	3	3
6500	0	0	1	1	1	2	2	2	2	2	3

Table 2.5: Design Chart Assuming $\boldsymbol{SN(50)_{smooth}=25}$, well finished, and using MD

MD (%)	Estimated years to reach SN(50)$_{smooth}$										
ESALs/lane (per day)	26%	24%	22%	20%	18%	16%	14%	12%	10%	8%	6%
500	2	4	8	14	25	45	82	148	175	175	175
1000	1	2	4	7	13	23	41	74	87	87	87
1500	1	1	3	5	8	15	27	49	58	58	58
2000	1	1	2	3	6	11	21	37	44	44	44
2500	0	1	2	3	5	9	16	30	35	35	35
3000	0	1	1	2	4	8	14	25	29	29	29
3500	0	1	1	2	4	6	12	21	25	25	25
4000	0	1	1	2	3	6	10	19	22	22	22
4500	0	0	1	2	3	5	9	16	19	19	19
5000	0	0	1	1	3	5	8	15	17	17	17
5500	0	0	1	1	2	4	7	13	16	16	16
6000	0	0	1	1	2	4	7	12	15	15	15
6500	0	0	1	1	2	3	6	11	13	13	13

Table 2.6: Design Chart Assuming $\boldsymbol{SN(50)_{smooth}=25}$, poorly finished, and using MD

MD (%)	Estimated years to reach SN(50)$_{smooth}$										
ESALs/lane (per day)	26%	24%	22%	20%	18%	16%	14%	12%	10%	8%	6%
500	1	1	2	3	5	9	16	27	31	31	31
1000	0	1	1	2	3	5	8	13	16	16	16
1500	0	0	1	1	2	3	5	9	10	10	10
2000	0	0	0	1	1	2	4	7	8	8	8
2500	0	0	0	1	1	2	3	5	6	6	6
3000	0	0	0	1	1	2	3	4	5	5	5
3500	0	0	0	0	1	1	2	4	4	4	4
4000	0	0	0	0	1	1	2	3	4	4	4
4500	0	0	0	0	1	1	2	3	3	3	3
5000	0	0	0	0	1	1	2	3	3	3	3
5500	0	0	0	0	0	1	1	2	3	3	3
6000	0	0	0	0	0	1	1	2	3	3	3
6500	0	0	0	0	0	1	1	2	2	2	2

2.6 Conclusions

The following conclusions can be drawn from Chapters 1 and 2 on achieving adequate surface friction:

- An equation to calculate the skid number based on the skid trailer using a smooth tire at 50 mph (SN (50) $_{smooth}$) was developed using values obtained by the dynamic friction tester (DFT60) at 60 kmh.
- Based on field test sections using limestone manufactured sands, ranging from 40 to 100 percent of the total sand, it was found that 100 percent sand gave low skid numbers compared to blended sands. The concrete test sections made with blended sands with as low as 40 percent siliceous content gave much higher skid numbers after 16 years of traffic (4.6 million ESALs).
- There is a reasonable correlation between surface friction based on the DFT and acid insoluble residue values (AI) for all aggregates and blends except dolomites and dolomitic limestones. For AI > 60 percent, friction values were high; but even for some fine aggregates for AI < 60 percent relatively high friction values were achieved.
- A good correlation exists between micro-Deval (MD) loss and AI for aggregates with MD less than 24 percent. Hard fine aggregates such as some dolomites and dolomitic limestones performed well when tested for friction using the TWPD.
- An alternative method to the current TxDOT acceptance procedure for use of fine aggregates in class P concrete is proposed based on MD. This method allows harder MFA that do not meet AI to be used at a higher replacement rate without causing reductions in skid on pavements. An MD limit of 12% loss was recommended for the new procedures described in Chapter 1.
- Using field and laboratory data, a model for predicting skid was established. The model allows $SN(50)_{smooth}$ to be estimated by knowing the total ESAL count and the AI or MD value of the pavement. More data should be collected with continued monitoring of existing sections or new sections prior to using the model as a design guide.
- The model established for predicting $SN(50)_{smooth}$ was used to develop design tables that can aid designers in choosing what AI or MD limits they need to follow knowing the ESAL count and the approximate number of years they need a pavement to maintain a desired value of $SN(50)_{smooth}$.

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