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Performance of Gravel Aggregates in Superpave Mixes with 100/95 Angularity

Final Report

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16. Abstract

The research conducted in this study evaluated the asphalt mixture performance of various gravel and crushed stone sources consisting of different levels of crushed face counts, as determined by ASTM D5821. Along with ASTM D5821, two additional aggregate angularity tests were conducted to evaluate the angularity and texture of the coarse aggregates; 1) AASHTO T326, *Uncompacted Void Content of Coarse Aggregates*, and 2) Aggregate Imaging System (AIMS). Each of the asphalt mixtures designed and tested during the study used three different asphalt binders; 1) Neat PG64-22, 2) Polymer Modified PG64-22 meeting NYSDOT Elastic Recovery specifications (called PG64-22 ER), and 3) Polymer Modified PG76-22. The permanent deformation properties of the different asphalt mixtures were measured using the Asphalt Pavement Analyzer (AASHTO TP63) and the Asphalt Mixture Performance Tester, AMPT (AASHTO TP79) with confining pressure applied to the specimens.

The aggregate testing concluded that ASTM D5821 correlated poorly to both AASHTO T326 and the AIMS system. There were also situations when aggregates had identical crushed counts but different levels of uncompacted voids contents. Meanwhile, the asphalt mixture testing demonstrated that the AMPT using confining pressure correlated well to the uncompacted void content results of AASHTO T326. Unfortunately, stresses applied in the APA did not mobilize the asphalt mixtures enough to differentiate the differences in aggregate angularity. Both asphalt tests were sensitive to asphalt binder high temperature stiffness, as determined by AASHTO TP70, *Multiple Stress Creep Recovery Test*. The final statistical analysis of the data resulted in a table that would allow NYSDOT to interchangeably "swap" aggregate angularity, as determined by AASHTO T326, and the non-recoverable creep compliance (J_{nr}), as determined by AASHTO TP70 to ensure gravel aggregate HMA mixes perform as well as crushed stone aggregate HMA mixes.

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Thomas Bennert, Senior Research Engineer at CAIT, was the principal investigator with the work being done under his general supervision. Allen Cooley of Burns Cooley Dennis, Inc. supervised the Micro-Deval testing and provided valuable input on the Literature Review and Final Report. Emad Kassem of Texas A&M University supervised the Aggregate Imaging System testing.

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EXECUTIVE SUMMARY

The NYSDOT currently employs ASTM D5821, *Percent of Fractured Particles in Coarse Aggregate*, as a means of indexing the angularity of coarse aggregates used in hot mix asphalt (HMA). Based on previous research, NYSDOT now allows the use of 100/98 coarse aggregates for HMA placed as the surface course on pavements of traffic levels greater than 30 million ESALs. However, this initial reduction from 100/100 to 100/98 has left the NYSDOT wondering if further reductions in coarse aggregate angularity, as determined using ASTM D5821, can be allowed for these heavy volume pavement sections.

The research conducted in this study evaluated the asphalt mixture performance of various gravel and crushed stone sources consisting of different levels of crushed face counts, as determined by ASTM D5821. Along with ASTM D5821, two additional aggregate angularity tests were conducted to evaluate the angularity and texture of the coarse aggregates; 1) AASHTO T326, *Uncompacted Void Content of Coarse Aggregates*, and 2) Aggregate Imaging System (AIMS). Each of the asphalt mixtures designed and tested during the study used three different asphalt binders; 1) Neat PG64-22, 2) Polymer Modified PG64-22 meeting NYSDOT Elastic Recovery specifications (called PG64-22 ER), and 3) Polymer Modified PG76-22. The permanent deformation properties of the different asphalt mixtures were measured using the Asphalt Pavement Analyzer (AASHTO TP63) and the Asphalt Mixture Performance Tester, AMPT (AASHTO TP79) with confining pressure applied to the specimens.

The testing results showed that the measurements determined from ASTM D5821 did not correlate to the other angularity and texture test procedures (AASHTO T326 and AIMS). Coarse aggregates having identical fractured face counts resulted in much different measurements when tested with AASHTO T326 and the AIMS device. Additional aggregate angularity testing also showed that AASHTO T326 was sensitive and performed rationally with respect to slight additions to rounded coarse aggregate particles. Meanwhile, the AIMS testing showed conflicting results with slight additions to rounded aggregates were introduced into the coarse aggregate samples. This was attributed to the manual placement of crushed gravels on the 2-D measurement tray.

When evaluating the permanent deformation properties of the asphalt mixtures, it was immediately obvious that the test results of the Asphalt Pavement Analyzer were more sensitive to the asphalt binder high temperature stiffness than the aggregate angularity. This may have been due to the test parameters, especially test temperature, used in the study. The Flow Number results, measured in the AMPT, were found to correlate well to the aggregate angularity measured in AASHTO T326. The test results of the AMPT also indicated that as the asphalt binder high temperature stiffness increased, the sensitivity to the aggregate angularity decreased, clearly indicating that the permanent deformation was a function of both the aggregate angularity and binder stiffness. Statistical analysis of the AMPT results resulted in a final recommendation table that would allow asphalt mixture suppliers to interchangeably “swap” coarse aggregate angularity, as determined in the AASHTO T326, and asphalt binder PG grade and/or the non-recoverable creep

compliance (J_{nr}) properties. Unfortunately, due to the lack of any correlation between aggregate angularity and asphalt mixture permanent deformation performance, it was recommended for the NYSDOT to discontinue the use of ASTM D5821 as a means of indexing coarse aggregate angularity for pavements having greater than 30 million ESALs and adopt AASHTO T326 to index the angularity properties of crushed gravels in New York.

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CHAPTER 1

1.1 INTRODUCTION

Aggregates comprise of over 80% of the hot mix asphalt (HMA) mixture by volume. Therefore, it is obvious that the aggregate characteristics are a major factor in the performance of HMA. In the Superpave mixture design system, aggregate criteria were included to assure proper performance of the HMA. These criteria included: coarse aggregate angularity (percent of fractured faces), fine aggregate angularity (percent uncompacted voids in the fine aggregate), flat and elongated particles, clay content, and gradation parameters. The recommended limits set by the Strategic Highway Research Program (SHRP) on these aggregate criteria were established by a group of experts based on years of previous research and experience utilizing the Modified Delphi approach (Cominsky, 1994). The main premise behind the aggregate criteria was to provide an angular aggregate skeleton to maximize internal shear strength, and hence, mixture stability.

Rounded aggregate provides minimal shear interlock between particles and will easily "roll" over one another allowing the asphalt mixture to simply flow during loading. Increasing the amount of fractured faces in the coarse aggregate, thereby increasing its angularity, will improve the stability of the asphalt mixture. The Superpave criteria, established by the SHRP group of experts, recommended increasing the amount of fractured faces for coarse aggregate (+ 4.75mm sieve) with increasing traffic. Angularity requirements also increase for layers near the pavement surface. Table 1.1 shows the initial Coarse Aggregate Angularity Criteria established by the SHRP Aggregate Expert Group for the implementation of Superpave. It should be noted that actual numbers recommended were not based upon any formalized research, but were simply based on past experiences of the expert group.

Table 1.1 – Initial Superpave Coarse Aggregate Angularity Criteria

Traffic (millions of ESAL's)	Depth from Pavement Surface	
	< 100 mm	> 100 mm
< 0.3	55/-	-/-
< 1.0	65/-	-/-
< 3.0	75/-	50/-
<10.0	85/80	60/-
<30.0	95/90	80/70
<100	100/100	95/90
>100	100/100	100/100

Note: "85/80" denotes that 85% of the coarse aggregate has one or more fractured faces and 80% has two or more fractured faces.

A number of research studies have shown that the percent of crushed/angular coarse aggregate particles increase mixture stability, thereby verifying the concept of the Superpave aggregate requirements. One of the more comprehensive studies was published by Cross and Brown (1992). Cross and Brown (1992) reported on a study that included 42 pavement sections in 14 different states. At each location, rut depth measurements, mix design information, construction records, traffic counts, and pavement samples were collected and recorded. All of the collected data was analyzed and compared with the field rutting information. Of all the material and mixture properties studied, coarse and fine aggregate angularity correlated best to pavement rutting (i.e. – as aggregate angularity increased, pavement rutting decreased).

Although the intent of the aggregate specifications was to ensure a high level of internal shear strength by maximizing aggregate angularity, in some states, it precluded many suppliers from using native gravel sources for HMA in asphalt pavements designed for traffic levels in excess of 30 millions ESAL's. According to Superpave, coarse aggregates to be used in an asphalt mixture where the traffic level is greater than 30 million ESAL's must have a coarse aggregate angularity of 100/100. This requirement eliminates all gravel sources from being used on these pavements because they would never meet the 100/100 angularity requirement. Therefore, a NYSDOT supplier/contractor would be required to bring in crushed stone to meet the 100/100 requirement. This would obviously increase the price of the HMA due to the additional material and shipping costs of the crushed stone.

This exact situation arose in 2003 when an aggregate supplier in New York State approached the NYSDOT about using a 100/98 gravel for a HMA mix that was to be placed on a pavement with traffic levels exceeding 30 million ESAL's. The supplier, Lopke Products from Binghamton, NY, contracted the National Center for Asphalt Technology (NCAT) to conduct rutting-type testing, utilizing the Asphalt Pavement Analyzer, on two sources of coarse aggregates; a 100/100 and a 100/98. Based on the rutting tests conducted by NCAT, it was statistically determined that the 100/100 and 100/98 gravels produced similar rut resistant mixtures (Prowell, 2003). Based on this study, the NYSDOT revised its current specifications to allow 100/98 coarse aggregates for asphalt mixtures placed on pavements exceeding 30 million ESAL's.

In 2004, Lopke Products again utilized the laboratory services of NCAT to evaluate angularity issues (Prowell, 2003; Prowell et al., 2005). Although the main premise behind the 2004 study was to evaluate levels of fine aggregate angularity, additional work regarding the coarse aggregate angularity was also conducted. As reported by Prowell et al. (2005), three different levels of coarse aggregate angularity were evaluated; two crushed gravels having a coarse aggregate angularity of 100/90 and 100/95, and a limestone aggregate of 100/100. All three coarse aggregates were blended with the identical limestone manufactured sand to produce a 12.5mm nominal maximum aggregate size (NMAS) gradation according to the Superpave specifications. The final mixture gradations and volumetric properties of the 12.5 NMAS were all similar and the differences were found to be insignificant. Statistical comparisons of the Asphalt

Pavement Analyzer rutting results indicated that the results between the three different HMA mixtures were similar. Meaning that the performance of the HMA mixtures were not affected by the differences in the coarse aggregate angularity values. Identical findings were also determined using three different 25mm NMAS HMA mixtures produced with different coarse aggregate angularity values.

1.2 RESEARCH NEED STATEMENT

As previously mentioned, Superpave currently utilizes ASTM D5821, *Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate*, to determine the amount of angular coarse aggregates. However, many researchers and practioners question the precision of the test method and its correlation to rutting measurements in hot mix asphalt. Current Superpave specifications require a greater amount of coarse aggregate angularity as the traffic levels increase. For pavements carrying greater than 30 million ESAL’s, NYSDOT requires fractured faces, as determined by ASTM D5821, to be 100/98. This was recently reduced from 100/100 based on research work conducted at NCAT described previous. Further work regarding the fractured faces and alternative methods to assess coarse aggregate angularity may indicate that NYSDOT could reduce their current requirements even more. However, further research is needed to validate this approach.

CHAPTER 2

STATE OF PRACTICE

2.1 INTRODUCTION

The project statement for NYSDOT C-06-20 identifies Task 2 as a review of existing literature on the measurement of coarse aggregate angularity, in particular ASTM D5821, and its influence on the rutting performance of hot mix asphalt. Of particular importance during the research project was to acquire a more accurate and detailed summary of the research performed in the field of aggregate angularity measurements and their influence and sensitivity to permanent deformation.

A tremendous volume of literature was identified related to the measurement of aggregate angularity and its influence on HMA stability and rutting susceptibility. In particular, four recent National Cooperative Highway Research Program (NCHRP) reports dealt with this issue in detail, or parts there of. Highlights and excerpts of these reports were utilized, as well as other relevant literature collected through various journals, conference proceedings and technical reports. Although technical publications pertaining to aggregate angularity measurements and their impact on hot mix asphalt performance were collected from journal articles dating back sixty (60) years, (i.e. - Campen and Smith, 1948, "A Study of the Role of Angular Aggregates in the Development of Stability in Bituminous Mixtures", AAPT Vol. 17), to aid in limiting the size of the Literature Search to the twenty (20) pages specified in the proposal, only relevant literature in the past 10 years was utilized for this Literature Review.

To help the reader, subsequent sections of this chapter were organized by topic. The first section discusses ASTM D5821 "Percent of Fractured Particles in Coarse Aggregate". The second section discusses different methodologies and test procedures to measure aggregate angularity and its influence on the permanent deformation properties of hot mix asphalt. The third and final section is a summary of the literature search. Individual summaries are provided for each reference.

2.2 ASTM D5821 "Percent of Fractured Particles in Coarse Aggregate"

1. **Cross, S. and E.R. Brown, 1992, "Selection of Aggregate Properties to Minimize Rutting to Heavy Duty Pavements",** ***Effects of Aggregates and Mineral Fillers on Asphalt Mixture Performance, ASTM STP 1147,*** **American Society of Testing and Materials, Philadelphia.**

Cross and Brown (1992) conducted a study on the selection of aggregate properties to help in minimizing rutting. The study consisted of 42 different pavement sections in 14 states, where 30 of the 42 pavement sections exhibited premature rutting. Extracted cores from the pavements were evaluated for density, asphalt content, gradation, and various aggregate properties that included two fractured faces for coarse aggregates. Using the data generated only for the pavement sections where the in-place air voids where greater than 2.5%, the authors developed the following relationship that was found to be statistically significant ($\alpha = 0.01$).

$$\frac{\text{Rut Depth (mm)}}{\sqrt{\text{ESAL}}} = 0.0338 - 0.0025 \times (\text{Percent of 2 Crushed Faces}) \qquad (1)$$

Even though the relationship only generated an $R^2 = 0.42$, the Coarse Aggregate Angularity, as determined using the Crushed Face Count was incorporated into the Superpave Mixture Design.

2. Hand, A. J. Epps, and P. Sebaaly, 2000, "Precision of ASTM D5821 Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate", *Journal of Testing and Evaluation,* American Society for Testing and Materials, Vol. 28, No. 2, pp. 67 – 75.

Hand et al. (2000) conducted a round-robin study to determine the precision of ASTM D5821. The study was initiated due to concerns of insufficient fractured faces in the original gravel source used at WesTrack. Ten (10) laboratories tested four (4) aggregates used at WesTrack. The data collected through that study resulted in the precision statement shown in Table 2.1.

Table 2.1 – Precision Statement for Both One or More and Two or More Fractured Faces (from Hand et al., 2000)

Property and Index Type	Standard Deviation (%)	Acceptable Range of Two Results
	One or More Fractured Faces	
Single Operator Precision	1.1%	3.0%
Multi-Laboratory Precision	1.8%	5.1%
	Two or More Fractured Faces	
Single Operator Precision	1.8%	5.1%
Multi-Laboratory Precision	2.9%	8.2%

3. **Carlberg, M., C. Berthelot, and N. Richardson, "In-Service Rut Performance of Saskatchewan Highways and Transportation Asphalt Concrete Mixes", In *Canadian Technical Asphalt Proceedings 2002*, Calgary, Alberta.**

In a similar study to Hand et al. (2000), Carlberg et al. (2002) conducted a multi-laboratory study to determine the precision of ASTM D5821. The study used thirty-four (34) "well-trained observers" evaluating two samples of partially crushed gravel. The results of the study indicated that the multi-laboratory standard deviation of two or more fractured faces was 5.2% for "well-trained observers". The acceptable range between two properly conducted tests by two "well-trained observers" was reported to be 14.7%.

4. **Prowell, B., J. Zhang, and E.R. Brown, 2005, *NCHRP Report 539: Aggregate Properties and the Performance of Superpave-Designed Hot Mix Asphalt*, National Cooperative Highway Research Program, Transportation Research Board, Washington D.C., 90 pp.**

As part of their study, Prowell et al., (2005) developed and distributed a survey to the state agencies to determine what aggregate specifications are currently being used. Survey results regarding ASTM D5821, "Percent of Fractured Particles in Coarse Aggregate", showed that only 39% of the agencies specify the criteria outlined in the Superpave Design Method (AASHTO M323), with six states lowering the fractured-face requirements. As stated by Prowell et al. (2005);

> "This is most likely in recognition of the fact that it is nearly impossible to achieve 100% particles with two or more crushed faces with crushed gravel sources. Although there is extensive research that indicates improved rut resistance with increased percentages of fractured faces, little work has been done to investigate the effect at high levels of fractured faces (between 95% to 100%)."

2.3 Test Methods of Aggregate Angularity and Influence on the Rutting of Hot Mix Asphalt

5. **Ahlrich, R., 1996, "Influence of Aggregate Properties of Performance of Heavy-Duty Hot Mix Asphalt Pavements", In *Transportation Research Record 1547*, TRB, National Research Council, Washington, D.C., pp. 7 – 14.**

Ahlrich (1996) developed an uncompacted voids test for coarse aggregate that was similar to AASHTO T304 which is used to measure fine aggregate angularity in the Superpave mix design system. The premise behind that test's development was to provide a means of indexing aggregate angularity that was related to HMA performance, subjective with minimal user bias, and less labor intensive than current aggregate angularity indexing methods (i.e. – ASTM D3398, Index of Aggregate Particle Shape and Texture). Ahlrich (1996) found that the coarse aggregate uncompacted voids (currently

AASHTO T326) correlated well with percent fractured faces and ASTM D3398 (Index of Aggregate Particle Shape and Texture), as well as the confined, repeated load permanent deformation test results conducted on compacted hot mix asphalt specimens of varying coarse aggregate mineralogy and angularities.

6. Kandhal, P. and F. Parker, Jr., 1998, *NCHRP Report 405: Aggregate Tests Related to Asphalt Concrete Performance in Pavements*, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C.

Kandhal and Parker, Jr.(1998) generated a comprehensive assessment of aggregate indexing methodologies and how they relate to HMA performance for both coarse and fine aggregates. The work conducted by Kandhal and Parker, Jr. (1998), as part of NCHRP Project 4-19, recommended different performance-related aggregate tests to evaluate aggregates for their potential use in HMA. In their report, Kandhal and Parker, Jr. (1998) considered permanent deformation, as well as fatigue cracking and surface defects, although the permanent deformation results will only be discussed here. The performance relationships generated were based on laboratory tests conducted in the Superpave Shear Tester (SST) and the Georgia Loaded Wheel Tester, which is a predecessor to the Asphalt Pavement Analyzer.

Nine (9) aggregate tests were performed to evaluate coarse aggregate shape, angularity, and texture:

- Index of Aggregate Particle Shape and Texture (ASTM D3398)
- Image Analysis (Georgia Institute of Technology)
- Flat and Elongated and Flat or Elongated Particles by ASTM D4791
- Flakiness Index (British Standard 812)
- Elongation Index (British Standard 812)
- Percent Fractured Particles in Coarse Aggregate (ASTM D5821)
- Uncompacted Voids in Coarse Aggregate (Currently AASHTO T326)
- Uncompacted Voids in Coarse Aggregate – Shovel Techniques (AASHTO T19)

The research conducted by Kandhal and Parker, Jr.(1998) indicated that for coarse aggregates, the modified uncompacted voids test [previously developed by Ahlrich (1996) and currently specified as AASHTO T326 correlated best to HMA permanent deformation with the flat or elongated particle test of a 2:1 ratio providing the second best correlation. Unfortunately, ASTM D5821 was only performed on three gravel sources included in the nine different aggregate sources, and therefore, excluded from the statistical analysis.

Along with their conclusions, Kandhal and Parker, Jr.(1998) also recommended that the aggregate tests identified as part of NCHRP 4-19 should be validated using either full-scale accelerated load tests. The full-scale accelerated testing would eventually become NCHRP 4-19(b), "Aggregate Tests for HMA Mixtures in Pavements" (White et al., 2006).

7. Prowell, B., 2003, *Rutting Evaluation of Lopke Aggregate Blends, NCAT Report 03-06*, National Center for Asphalt Technology, Auburn, AL, 23 pp.

The study evaluated the rutting performance of three levels of coarse aggregate angularity; as-received, 95 percent two crushed faces, and 100 percent two crushed faces. Testing was conducted on [NYSDOT approved?] approved 12.5mm NMAS and 25mm NMAS Superpave mixes using the crushed gravel source of Lopke Contracting in New York State. Rut testing was performed using the Asphalt Pavement Analyzer (APA) at the optimum asphalt content for each mixture and performed using the protocol established in NCHRP 9-17, "Accelerated Laboratory Rutting Tests: Asphalt Pavement Analyzer" (Kandhal and Cooley, 1998).

The test results showed that all mixes evaluated either met or performed better than the NCHRP 9-17 APA rutting criteria of less than or equal to 4.5mm. Variability of the test results were first determined using the F-test and then determined if the APA rut results were statistically equal using the Student t-test. The statistical analysis showed that the APA rutting results of the as-received and 100/95 crushed gravel were statistically equal to the two face crushed limestone mixture. Table 2.2 shows the crushed face count results as determined by ASTM D5821 for the different mixtures evaluated, along with their respective APA rut data.

Table 2.2 – Crushed Face Count and APA Rut Depths (Adapted from Prowell, 2003)

NMAS	Gravel HMA Mix Type	ASTM D5821 Results	APA Results (mm)
12.5mm	Lopke As-Received	100/90.4	4.09
	Lopke 100/95	100/98	4.32
	Limestone 100/100	100/100	2.70
25mm	Lopke As-Received	100/91.8	3.8
	Lopke 100/95	100/99.1	4.53
	Limestone 100/100	100/100	3.56

8. Al-Rousan, T., E. Masad, L. Myers, and C. Speigelman, 2005, "New Methodology for Shape Classification of Aggregates", In *Transportation Research Record 1913*, TRB, National Research Council, Washington, D.C., pp. 11 – 23.

The authors presented a methodology for the classification of aggregate shape properties. The classification methodology is based on the direct measurements of the aggregate form (three dimensions), angularity, and texture (Figure 2.1). The computer analysis methods are reported to be simple, have physical meanings that can be interpreted easily and has no user bias.

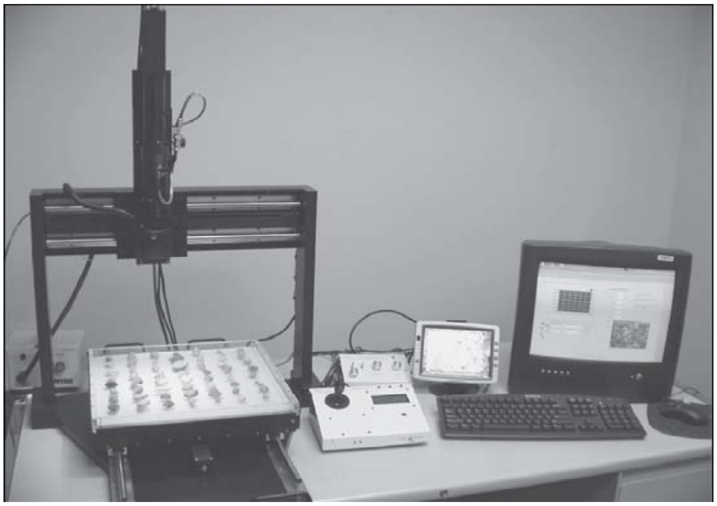


Figure 2.1 – Aggregate Imaging System (AIMS) and Its Components

One of the more significant findings during the initial development and analysis was the AIMS ability to capture aggregate texture and that aggregate texture varies considerably among different aggregate samples.

The classification results are presented in terms of the distribution of shape properties within an aggregate sample (Figure 2.2). According to the authors, this

> "… gives the methodology the capabilities to (a) explore the influence of different processes such as crushing and blending on aggregate shape, (b) conduct quality control to detect changes in the distribution of any of the shape characteristics, (c) relate the distribution of different shape characteristics to performance, and (d) develop specifications based on the distribution of aggregate shape characteristics rather than average indices."

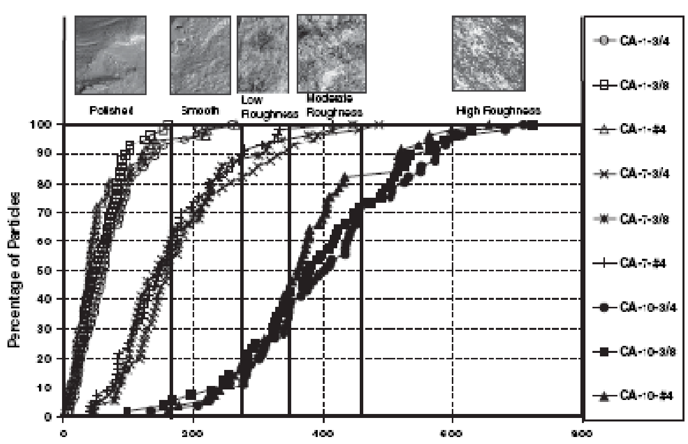


Figure 2.2 – Illustration of Texture Index Distribution from the AIMS (after Al-Rousan et al, 2005)

9. **White, T., J. Haddock, and E. Rismantojo, 2006, *NCHRP Report 557: Aggregate Tests for Hot-Mix Asphalt Mixtures*, National Cooperative Highway Research Program, Transportation Research Board, Washington D.C., 48 pp.**

The objective of this research was to use accelerated pavement testing techniques to conduct the rutting, fatigue, and moisture susceptibility validation experiments identified in NCHRP Project 4-19 by Kandhal and Parker, Jr. (1998). The validation effort involved subjecting HMA mixtures prepared with various aggregates to full-scale accelerated pavement testing and measuring their performance according to one of three HMA failure modes: (1) rutting; (2) moisture susceptibility; and (3) fatigue.

The full-scale rutting results showed that the coarse aggregate uncompacted voids (AASHTO T326) was found to be the best single predictor of rutting performance of the coarse-graded mixtures as indicated by the descriptive ranking. The test appears to capture information related to particle shape and texture and rutting decreases as the coarse aggregate uncompacted void content increases. A relationship between traffic and coarse aggregate seemed less sensitive for uncompacted void content values in the range of 40 to 45 percent. The relationship becomes stronger in the coarse aggregate uncompacted void content range of 45 to 50 percent. Previous testing in Purdue's Accelerated Pavement Tester (APT) has indicated that one APT pass is equivalent to approximately 2,500 ESAL's. The author's applied this relationship to the coarse aggregate uncompacted void content/wheel pass data, a performance limit occurs at 100,000 ESAL. For expected traffic below 100,000 ESAL's, a minimum coarse aggregate uncompacted void content of 40 percent would be required. A coarse aggregate uncompacted void content of at least 45 percent would be required for traffic above 100,000 ESAL's.

10. **Christensen, D., R. Bonaquist, and A. Cooley, 2006, *Quarterly Report for NCHRP Project 9-33, A Mix Design Manual for Hot Mix Asphalt*, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C.**

As part of the National Cooperative Highway Research Project (NCHRP) 9-33, "A Mix Design Manual for Hot Mix Asphalt", the research team of Christensen et al. (2006) was required to evaluate the aggregate requirements in a similar manner to the Strategic Highway Research (SHRP) work during the development of Superpave. Based on the vast literature review conducted, the researchers recommended Tables 2.3 and 2.4 as test methods and specification criteria for coarse aggregate shape and angularity, respectively.

Table 2.3 – Criteria for Flat or Elongated Particles (After Christensen et al., 2006)

Design ESAL's (millions)	Maximum Percent of Flat or Elongated Particles	
	(5:1, Superpave)	(2:1, NCHRP 9-33)
< 0.3	----	----
0.3 to 3.0	10	50
3.0 to 10.0	10	50
10.0 to 30.0	10	50
> 30.0	10	50

Criteria are presented as percent flat or elongated particles by mass.

Table 2.4 – Coarse Aggregate Angularity, AASHTO T326 (Christensen et al., 2006)

Design ESAL's (millions)	Depth of Pavement Layer from Surface (mm)	
	0 to 100	> 100
< 0.3	40	----
0.3 to 3.0	40	----
3.0 to 10.0	45	40
10.0 to 30.0	45	45
> 30.0	45	45

Criteria are presented as percent air voids in loosely compacted coarse aggregate.

11. Bennert, T. and M. Bryant, 2006, *Inter-relationship Between Fine Aggregate Angularity and High Temperature PG Grade to Mitigate HMA Rutting*, Internal Research Conducted at the Center for Advanced Infrastructure and Transportation (CAIT).

This study did not specifically evaluate the influence of coarse aggregate angularity on mix performance; however, the study conducted by Bennert and Bryant (2006) did illustrate the potential for a fine aggregate angularity/PG grade swap for providing HMA rutting resistance. Figures 2.3 and 2.4 show Asphalt Pavement Analyzer and SST Repeated Shear test results of HMA mixtures with different fine aggregate angularity (FAA) values, as determined by AASHTO T304, and different PG binder grades; PG64-22, PG70-22, and PG76-22. The test results show that as FAA increases, the amount of rutting/permanent strain decreases. However, the magnitude of the effect is minimized as the high temperature PG grade increases. The test results also show that at an FAA of approximately 47% or greater, the influence of the PG grade is minimized.

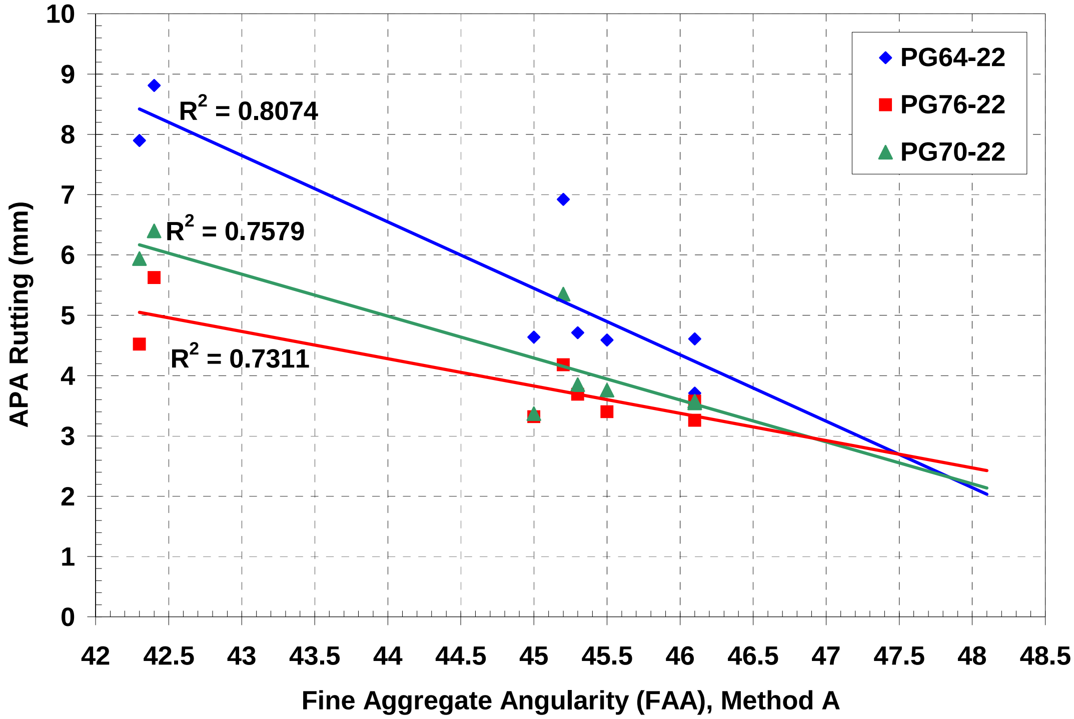


Figure 2.3 – APA Test Results Conducted at 64°C

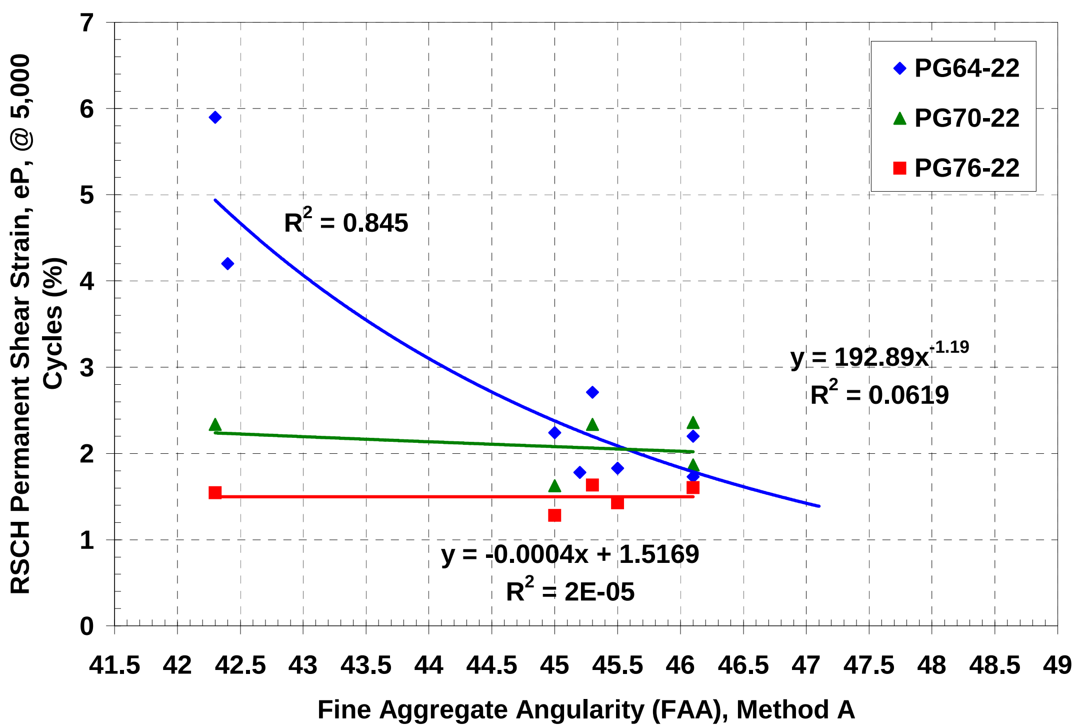


Figure 2.4 – SST Repeated Shear Conducted at 54.4°C

The laboratory rutting results of various fine aggregate angularities indicate that state agencies may be able to allow suppliers to “bump” PG grades to substitute for fine aggregates of marginal angularity, if it is cost effective. Although not measured in this study, it is hypothesized that the same methodology may be viable for coarse aggregates as well.

12. Huang, B., X. Chen, X. Shu, E. Masad, and E. Mahmoud, 2008, “Effects of Coarse Aggregate Angularity and Asphalt Binder on Laboratory-Measured Permanent Deformation Properties of HMA”, Presented at the 87th Annual Meeting of the Transportation Research Board, TRB, National Research Council, Washington, D.C.

In the present study, efforts have been made to identify the contributions of aggregate structure and asphalt binder to the rutting characteristics of a dense-graded surface HMA mixture. Coarse gravels at five different angularity levels (100, 85, 70, 50 and 35 percent fractured face counts) were used to produce mixtures with similar aggregate gradations. Three different asphalt binders (PG 64-22, PG 76-22, and PG 82-22) were used to make mixtures for laboratory rut evaluations. The Aggregate Imaging System (AIMS), Uncompacted Voids in Coarse Aggregate (VCA), and Triaxial Shear tests were conducted to evaluate the coarse aggregate angularity (CAA). The US Army Corps of Engineers’ Gyratory Testing Machine (GTM), Static Confined Creep and the Asphalt Pavement Analyzer (APA) tests were selected to characterize the rut resistance of asphalt mixtures.

The results from this study indicated that coarse aggregate AIMS, VCA and Tri-axial tests were related to the Coarse Aggregate Angularity (CAA) and laboratory measured rutting indices. At temperatures close to the binder’s upper grade limit, aggregate structures played a critical role for the rut resistance of HMA mixtures; whereas, at temperatures below the binder’s upper grade limit, the stiffness of asphalt binder played a more important role in the rut resistance of asphalt mixtures evaluated in this study.

The test results also showed that:

- The aggregate imaging system (AIMS), VCA and Tri-axial tests can be used to characterize the angularities of coarse aggregates.
- Creep and APA tests generally provided consistent ranking in evaluating the rutting performance of HMA mixtures.
- Aggregate structure and binder stiffness had significant effects on the rutting performance of HMA.
- CAA had significant effect on the laboratory rutting performance of HMA mixtures when a soft binder was used.
- Use of relatively hard asphalt binder could also lead to high rut-resistance HMA mixture and may “compensate” for the relatively low aggregate angularity.
- The traditional CAA (fractured face count?) had the strongest correlation with the laboratory static creep permanent strain; and

- The angularity index as measured by the AIMS had the strongest correlation with the APA rut depth;

An interesting result in the research clearly showed that as the PG binder grade increased, the effect of the aggregate angularity was not as significant. Figure 2.5 shows the Asphalt Pavement Analyzer (APA) rutting results versus the coarse aggregate angularity level, as determined using ASTM D5821. The results clearly show that APA rutting decreases as angularity and PG grade increases. However, as the PG increases to a PG76-22 and to a PG82-22, the influence of aggregate angularity decreased, illustrating the concept that there is an inter-relationship between aggregate angularity and high temperature PG grade with respect to reducing rutting. Unfortunately, the wide range of coarse aggregate angularity values do not provide insight as to the relative performance in the range of angularities to be evaluated in this study.

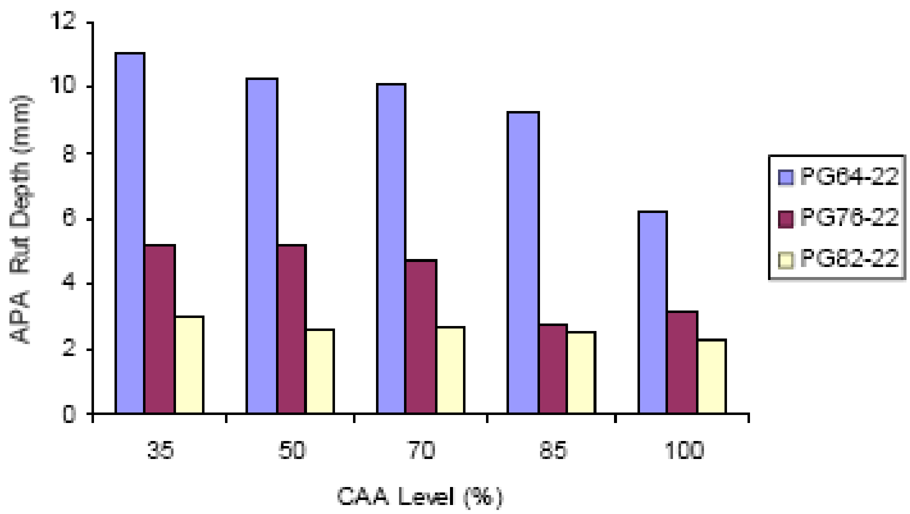


Figure 2.5 – APA Rutting Results of Test Specimens with Varying PG Grade and Coarse Aggregate Angularity (After Huang et al., 2008)

2.4 Summary of Literature Review Findings

A Literature Review was conducted to determine the state-of-the-practice regarding the measurement of coarse aggregate angularity (CAA) and the relationship between these tests and hot mix asphalt (HMA) rutting performance. The Literature Review provided an extensive amount of information regarding aggregate angularity, and therefore, only relevant work conducted within the past 10 years was reported.

It was evident from the Literature Review that a majority of the "coarse aggregate angularity influence on HMA rutting" dealt with either a very broad range of CAA values as determined using ASTM D5821 or simply the affect of aggregate gradation (i.e. – coarse versus fine-graded) with minimal attention to CAA. There existed limited data on CAA values between 100 to 90% crushed faces, as represented by the NCAT work conducted on Lopke aggregates in 2003 (Prowell, 2003). Therefore, it seems that not only is this study timely for the NYSDOT, but for the aggregate and HMA industry as well.

Based on the relevant literature collected and reviewed in this study, the following conclusions were drawn regarding coarse aggregate angularity:

2.4.1 ASTM D5821 – Percent of Fractured Faces in Coarse Aggregate

- During the Delphi process, which was the SHRP Expert Task Group's method for selecting appropriate aggregate requirements for Superpave, no test method was identified to rank coarse aggregate angularity. In fact, it was the FHWA's Office of Technology Applications that eventually recommended Pennsylvania DOT's Test method 621. Pennsylvania DOT Test Method 621, like its successor ASTM D5821, was based on the visual inspection of individual aggregates to determine the percentage of that aggregate stockpile/blend that contained fractured face(s). With any visually-based criteria, poorer levels of precision are expected due to user bias and error.
- Work conducted by Cross and Brown (1992) showed that although the Percent of Two Crushed Faces was statistically significant to the rutting performance of asphalt pavements, only a moderate correlation ($R^2 = 0.42$) existed between the Percent of Two Crushed Faces and Rutting Rate.
- Although a Precision Statement has never been officially implemented by ASTM regarding the repeatability of ASTM D5821, two studies provide insight into the relative repeatability of the test.
    - Using four different aggregate types, Hand et al., (2000) enlisted ten laboratories to conduct the ASTM D5821 *Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate* for each aggregate type. Hand et al., (2000) concluded that the acceptable range of results for two results conducted by multiple labs was 5.1% for One Fractured Face and 8.2% for Two Fractured Faces. Therefore, two laboratories evaluating the same aggregate may result in angularities of

  100/98 and 96/92 and still be within the acceptable range of results for ASTM D5821.
  - The Ministry of Ontario conducted a similar study, although they included 34 "well-trained" observers to conduct ASTM D5821 on two samples of partially crushed gravel. The results concluded that the acceptable range between two properly conducted tests by two "well-trained" observers was 14.7%.
- It is well documented that the extent of crushing, which increases angularity and texture of the aggregate, is related to the rutting performance of hot mix asphalt. However, the Literature Review does indicate that due to the poor repeatability of ASTM D5821, it may be difficult to determine at what level of Fracture Face Count is appropriate for specification.

2.4.2 Test Methods of Aggregate Angularity and Influence on the Rutting of Hot Mix Asphalt

- AASHTO T326, known as the Modified Uncompacted Flow Test for Coarse Aggregates, seems to provide a quick and repeatable means of indexing coarse aggregate angularity and has been found to be related to rutting in HMA. Originally developed by Alhrich (1996), the test is an enlarged version of AASHTO 304. Laboratory rutting tests conducted by Alhrich (1996) and Kandhal and Parker, (1998), as well as full-scale rutting tests conducted during NCHRP Project 4-19b (White et al., 2006), concluded that the measured rutting was indeed highly correlated to the coarse aggregate angularity (CAA) as measured using AASHTO T326.
- The Aggregate Imaging System, AIMS, (Al-Rousan et al., 2005) is another test that shows promise in indexing the angularity and texture of aggregates, coarse or fine. The AIMS system uses actual photos taken of the aggregate under different lighting conditions to index the particle angularity and surface texture via computer algorithms. By measuring angularity and texture in this manner, it is completely void of user bias. The study conducted by Huang et al., (2008) showed that the AIMS Angularity Index resulted in the strongest correlation to HMA rutting measured in the Asphalt Pavement Analyzer (APA). The study evaluated five different coarse aggregate sources that were selected to provide a very broad distribution of Fractured Face Counts.

2.4.3 HMA Permanent Deformation Rutting Tests to Evaluate Aggregate Influence

- A majority of the literature reviewed utilized one or both of the following; Uniaxial/Triaxial Test Apparatus and/or Loaded Wheel Track Test Apparatus. Work conducted by Ahlrich (1996) evaluated the influence of CAA with a confined triaxial test apparatus. The main idea behind the applied confining stress was to better simulate the confined nature of HMA in the field, while trying to mobilize the influence the shear resistance characteristics of the HMA aggregate (angular shape and surface texture). Loaded Wheel Tracking tests, such as the Asphalt Pavement Analyzer (APA), was used in a number of laboratory evaluations, including NCHRP Project 4-19 (Kandhal and Parker, 1998), the initial Lopke gravel study conducted by NCAT (Prowell, 2003), the inter-laboratory fine aggregate angularity study conducted at Rutgers University (Bennert and Bryant, 2006) and the recent CAA study conducted at Huang et al., (2008). Both HMA rutting tests appear to be sensitive enough to illustrate the differences in aggregate angularity.

- Two studies showed interesting results pertaining to the use of PG grade “bumping” to supplement aggregate angularity. Both studies, Bennert and Bryant (2006) and Huang et al. (2008), showed that marginal aggregate angularities, fine or coarse, may be able to be used if the high temperature asphalt binder PG grade is increased. This may allow suppliers with marginal aggregate angularity properties to still utilize their local materials by increasing the PG grade of their typically used asphalt binder.

CHAPTER 3

DETAILED WORKPLAN

A thorough Literature Review had been conducted on relevant published journal articles, technical reports, and conference presentations regarding the coarse aggregate angularity (CAA) and its influence on hot mix asphalt (HMA) rutting properties. Based on the Literature Review and recent meetings and conversations with the New York State Department of Transportation (NYSDOT) Technical Working Group (TWG), the following Workplan was developed and conducted.

Subtask 3a - Obtain samples of crushed gravel and artificially achieve different Crushed Face Counts **–** The Consultant obtained samples from four (4) NYSDOT approved gravel suppliers that represent the greatest volume of gravel usage and supply. To keep the identity of the suppliers confidential, the names of the gravel suppliers were withheld from the final report and are only identified as" G#".

1. G221G
   CAA: 1's: 100/99.5; 1A's: 100/99.9
2. G3
   CAA: 1's: 100/99.2; 1A's: 100/99.4
3. G1
   CAA: 1's: 100/92.9; 1A's: 100/98.6
4. G4
   CAA: 1's: 100/99.2; 1A's: 100/99.4

At each source, the Consultant obtained crushed gravel that conforms to NYSDOT 1 and 1A designation. The Consultant also obtained uncrushed gravel that also conformed to the 1 and 1A designation. In one case, uncrushed gravel was not able to be collected due to the suppliers' lack of materials (i.e. – G3). In this case, the rounded gravel from G4 was utilized to obtain the different levels of crushed face counts shown in Table 3.1.

After the aggregates were obtained, the Consultant blended crushed and uncrushed gravel to artificially achieve different coarse aggregate angularity (CAA) values, as determined by ASTM D5821 *Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate.* Based on initial testing of the four (4) crushed gravel sources by NYSDOT, it appears that most of the two faces crushed counts are approximately 99% (i.e. – 100/99 according to ASTM D5821). Therefore, it had been agreed upon between the Consultant and the NYSDOT TWG that the uncrushed gravels were to be substituted for crushed gravel at 3% intervals to obtain the CAA values as shown in Table 3.1.

Three (3) NYSDOT approved crushed stone sources were included as "baseline" data to illustrate a 100/100 crushed stone. The three (3) crushed stone sources and the 20 different gravel blends provide a total of 23 different aggregate blends that were evaluated under aggregate angularity testing and hot mix asphalt laboratory rutting tests.

Table 3.1 – Proposed Coarse Aggregate Angularity Blends

ASTM D5821 Fractured Face Counts (Based on Theoretical Blending)

G1	G2	G3	G4
100/96	100/99	100/99	100/99
97/93	97/96	97/96	97/96
94/90	94/93	94/93	94/93
91/87	91/90	91/90	91/90
88/84	88/87	88/87	88/87

Subtask 3b: Determine shape characteristics of crushed stone and gravels – In the Superpave Design System, the required coarse aggregate property to ensure proper angularity, also called Consensus Properties, is the Coarse Aggregate Angularity (ASTM D5821). The Consultant evaluated the selected aggregates under ASTM D5821 and two other aggregate angularity test procedures.

NCHRP Projects 4-19 and 4-19(b) recommended including the Modified Uncompacted Void Content for Coarse Aggregate test (AASHTO T326). In fact, AASHTO T326 showed to have the best correlation to rutting among all aggregate tests conducted during NCHRP 4-19(b). NCHRP 9-33 has also recommended the Coarse Aggregate Angularity based on AASHTO T326. Therefore, the Consultant measured and recorded AASHTO T326 on the different aggregate sources and blends during the study.

The Consultant also included the Micro-Deval test (ASTM D6928) on the aggregate sources to index their durability. Although not a direct measurement of particle shape or texture, the Micro-Deval test has been found to be useful in predicting the durability of HMA as it can account for aggregate degradation during production and placement. The Micro-Deval test has also shown to be a useful screening tool for indexing the potential angularity polishing that occurs during the production and compaction process of the hot mix asphalt. This concept of polishing or angularity breakdown was also evaluated by first polishing the crushed gravels in the Micro-Deval testing and testing them in the Aggregate Imagining System (Gatchalian et al., 2006).

Shape, texture and angularity determined using the Aggregate Imaging System (AIMS) was also measured in the study. The AIMS quantitatively measures the entire distribution of different aggregate shapes and angularities for coarse and fine aggregates. The two most important parameters from the AIMS, both of which were measured in this study, are the shape parameter and angularity. Based on two-dimensional analysis, AIMS provides a percentage distribution of particles with different sphericities ranging from one (indicating a perfect sphere) to approximately zero (indicating a completely flat and elongated element). Since the AIMS method provides a quantitative means of evaluating shape, texture, and angularity, the Consultant incorporated the AIMS analysis into the study to compare to the other coarse aggregate angularity tests, as well as to correlate to the rutting tests.

Subtask 4a – Develop hot mix asphalt (HMA) mixture designs for sampled aggregates **-** Prior to the performance testing, the Consultant used each aggregate source and gravel blend to develop a hot mix asphalt design in accordance to the NYSDOT Superpave Specifications, *Materials Method 5.16 2006 – Superpave Hot Mix Asphalt Mixture Design and Mixture Verification Procedures.* To expedite the mixture design phase, the Consultant did not conduct Moisture Sensitivity testing (AASHTO T283). The Consultant also conducted only "verification" designs on the crushed gravel mixtures once the mixtures started using the blended, uncrushed gravel. Since each source had their own consistent bulk specific gravity and absorption properties for their gravel source, the optimum asphalt content determined during mixture design did not vary significantly due to different levels of Fractured Face Counts. Therefore, to expedite the mixture design phase, only verification designs were conducted on HMA mixture designs succeeding the initial design.

The Consultant blended the various gravel sources with a single fine aggregate to ensure no bias in the performance testing occurred and to isolate the properties of the coarse aggregates. The current fine aggregate source approved by the NYSDOT TWG for use was Dalrymple's 6-21F. Major efforts were conducted to try and keep a final aggregate gradation and amount of fine aggregate consistent for each mixture design conducted. For this study, a total of twenty-two (23) mix designs were conducted.

Subtask 4b – Permanent Deformation (Rutting) Testing **–** The Consultant conducted permanent deformation testing using two different test methods: 1) Asphalt Pavement Analyzer and 2) the Repeated Load Test using the Simple Performance Test specifications and confining pressure. The following describes the methodology used for each of the different permanent deformation test methods:

<u>Asphalt Pavement Analyzer</u> - The Asphalt Pavement Analyzer (APA) was previously used in the NCAT studies regarding the testing of the NYSDOT approved aggregate sources of varying coarse aggregate angularities. The test procedure used incorporated the recommendations from NCHRP Report 508 and entailed using an applied wheel load of 120 lbs (+/- 5 lbs) and a hose pressure of 120 psi (+/- 5 psi). The Consultant used a test temperature of 58^{o}C as this represents both the LTPPBind high PG grade, uncorrected for traffic and vehicle speed, and the test temperature previously used in the initial NCAT work.

<u>Repeated Load Simple Performance Test</u> - The Consultant also utilized the Asphalt Mixture Performance Tester (AMPT) to evaluate the permanent deformation properties

of the crushed gravel and stone mixes. The AMPT test device and test procedure conformed to the recommendations provided in Appendix D, Annex C of the *NCHRP Report 513, Simple Performance Tester for Superpave Mix Design: First Article Development and Evaluation*. However, unlike the APA test, the AMPT incorporated confining pressure to emphasize the potential internal aggregate shear development. It was anticipated that crushed gravels/stone with higher levels of angularity and surface texture should develop a greater resistance to permanent deformation due to the natural confining pressure of the pavement structure. A confining pressure of 20 psi and an applied cyclic load of 100 psi were used. The Consultant used a test temperature of 58^{o}C based on recommended LTPPBind high PG grade for New York State.

The Consultant also evaluated the permanent deformation characteristics of the different HMA mixtures using a neat PG64-22, a PG64-22 with an Elastic Recovery requirement of 60% (PG64-22 ER), and a polymer-modified PG76-22 asphalt binder. The PG64-22 with Elastic Recovery and the polymer-modified PG76-22 conformed to the NYSDOT specifications. The addition of the PG76-22 asphalt binder to the study allowed for the evaluation of a potential “bump” in asphalt binder grade when marginal crushed gravels are encountered.

CHAPTER 4

AGGREGATE TESTING

The aggregate testing consisted of measuring the angularity properties of the different aggregate sources. For angularity testing, three main test procedures were utilized;

1. Coarse Aggregate Angularity as determined using the Fracture Face Count (ASTM D5821);
2. Coarse Aggregate Angularity as determined using the modified Uncompacted Void Test procedure (AASHTO T326); and
3. Angularity and Texture measured in the Aggregate Imaging System (AIMS).

Aggregate samples were also measured before and after "abrasion" to determine how susceptible the crushed gravels are to polishing. This new test procedure developed by the Texas Transportation Institute (TTI) utilizes the Micro-Deval test to artificially polish the crushed aggregates. The AIMS device is used to first measure the angularity and texture of the aggregates, then the aggregates are polished in the Micro-Deval apparatus. After the abrasion process is complete and the aggregates are washed and dried, they are once again tested in the AIMS device. The degree of change in angularity and texture before and after the Micro-Deval is an indication of the polishing potential (due to production, construction, and traffic) of the aggregate source.

4.1 Angularity Testing of Coarse Aggregate Sources

4.1.1 Fracture Face Count (ASTM D5821)

The Fractured Face Count of the aggregate sources was determined according to ASTM D5821. According to the literature, ASTM D5821 is susceptible to variability in the test results due to its reliance on the "User" determining whether or not the aggregate contains fractured faces. Therefore, to eliminate the potential for a discrepancy between what NYSDOT currently reports for Fractured Faces and what the Consultant would record, it was determined that the NYSDOT would measure the Fractured Face count of the gravel sources and provide that information to the Consultant. The test results are shown below:

1. G2
   CAA: 1's: 100/99.5; 1A's: 100/99.9
2. G3
   CAA: 1's: 100/99.2; 1A's: 100/99.4
3. G1
   CAA: 1's: 100/92.9; 1A's: 100/98.6
4. G4
   CAA: 1's: 100/99.2; 1A's: 100/99.4

The Fractured Face measurements of the gravel sources indicate that the G2 gravel had the highest level of angularity, while the G1 gravel had the lowest level of angularity.

The average (1 and 1A's) two-face fractured counts for the aggregates used in the study are shown in Figure 4.1. All of the gravels selected had a one-face fractured count of 100%. The figure also includes the crushed stone sources that were included as "baseline" samples for comparison.

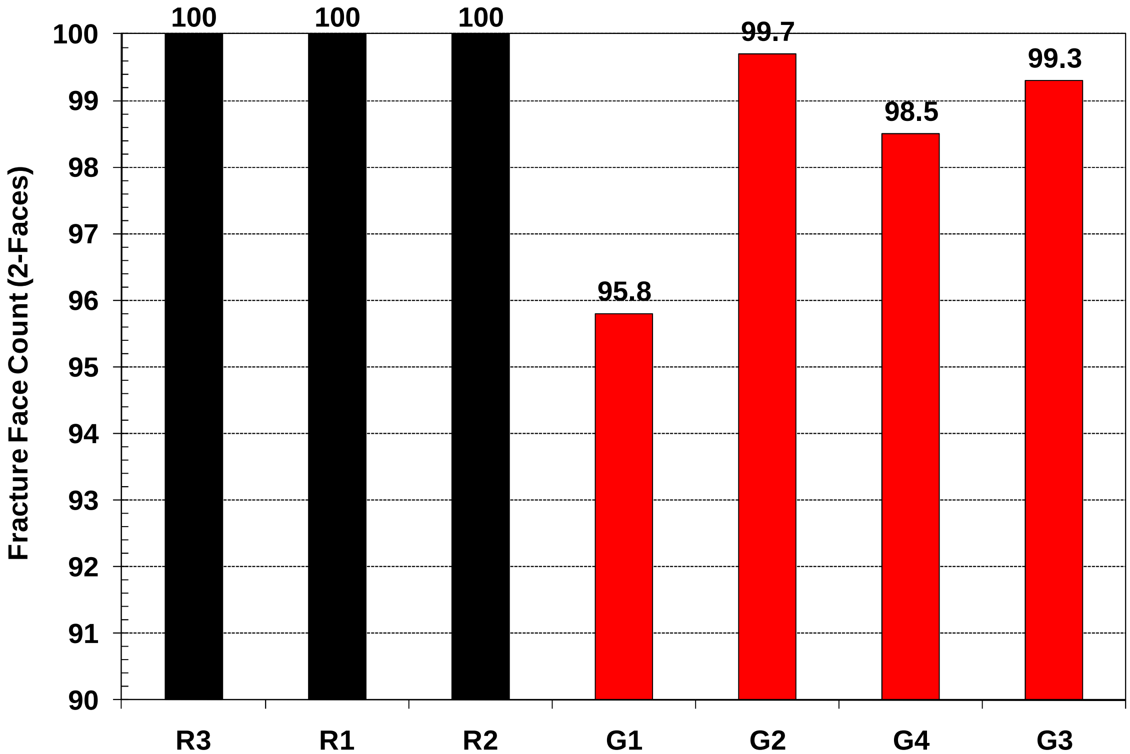


Figure 4.1 – Two Face Fractured Counts for Aggregates in Study

4.1.2 Coarse Aggregate Angularity (AASHTO T326 - Uncompacted Void Content of Coarse Aggregate)

AASHTO T325, *Uncompacted Void Content of Coarse Aggregates*, describes the determination of the loose uncompacted void content of a sample of coarse aggregate. When measured on any aggregate of a known grading, void content provides an indication of the aggregate's angularity, sphericity, and surface texture compared with other coarse aggregates tested in the same grading. The general test procedure and apparatus is similar to that of AASHTO T304, *Uncompacted Void Content of Fine Aggregates*, with respect to concept, test procedure, and the determination of the uncompacted void content. A picture of the test procedure is shown as Figure 4.2.

The test results for the aggregate sources used in the study are shown in Figure 4.3. The test results clearly indicate that the crushed stone (quarried process) aggregates have a higher degree of uncompacted voids, which indicates greater levels of angularity and texture. The test results also indicates that the Coarse Aggregate Angularity, as determined using AASHTO T326, does not correspond to the Coarse Aggregate

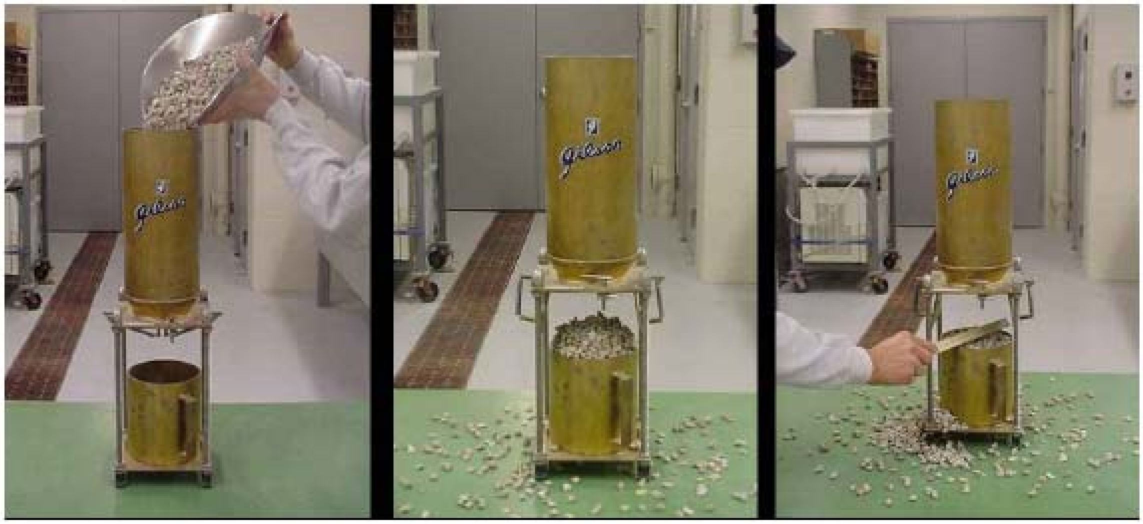


Figure 4.2 – Uncompacted Void Content of Coarse Aggregate Test Equipment

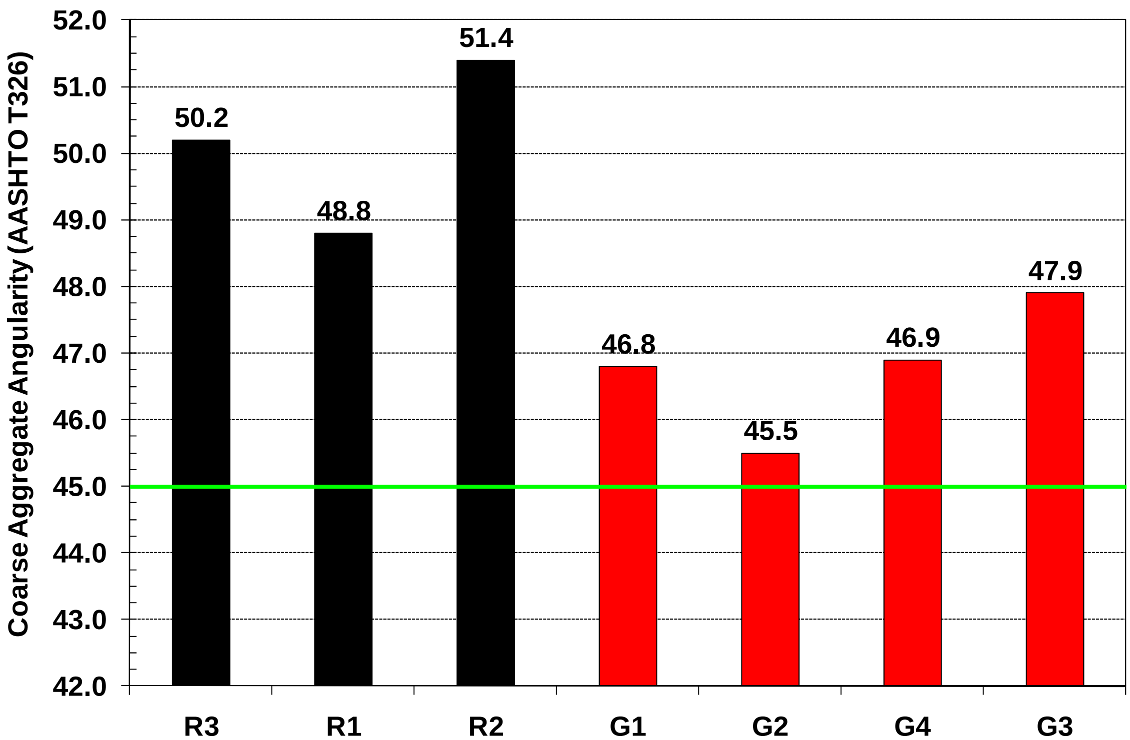


Figure 4.3 – Uncompacted Void Content Test Results for Aggregates in Study

Angularity as determined using the Fractured Face Count, ASTM D5821. Using AASHTO T326, the G2 gravel achieved the lowest level of angularity/texture, while the G2 aggregate source had the highest level of angularity when measured using the Fractured Face Count, ASTM D5821. Figure 4.3 also includes a "Pass/Fail" line at 45% uncompacted voids. This level was determined as an appropriate "Pass/Fail" designation based on the test results of NCHRP Project 4-19 and NCHRP Project 4-19(b). The aggregates evaluated in this study indicates that the G2 sample falls slightly above the 45% uncompacted voids.

4.1.3 Aggregate Imaging System (AIMS) Testing

Aggregate samples were sent to the Texas Transportation Institute (TTI) for angularity and texture assessment using the Aggregate Imaging System (AIMS). The AIMS device determines the shape characteristics of aggregate through image processing and analysis techniques. AIMS equipment consists of a computer automated unit which includes an aggregate measurement tray with marked grid points at specified distances along an x and y axis. The coarse aggregate is placed on the specified grid points for measurement. The system is also equipped with top lighting, used to evaluate texture, and back lighting, used to evaluate angularity. After pictures are taken with a camera unit, the aggregate texture is quantified using wavelet analysis method (called Texture index), while the aggregate angularity is described by measuring the irregularity of a particle surface using the gradient and radius methods (called Angularity Index). A picture of the AIMS measuring device is shown in Figure 4.4.

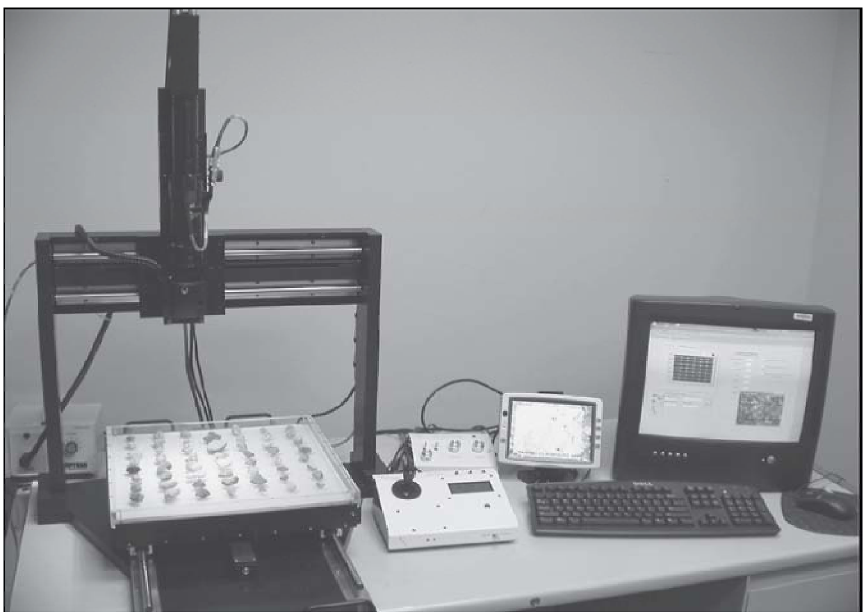


Figure 4.4 – Photo of the AIMS Measurement System

The results for the AIMS testing for the fractioned aggregate sizes are shown in Table 4.1. Examples of the general picture output from the AIMS device can be found in Figures 4.5 to 4.8. When reviewing the table, the reader should understand that the higher the Texture and Angularity value, the more surface texture and aggregate angularity the AIMS device is measuring.

In order to accurately compare the Texture and Angularity Index values, the results were normalized to their respective gradation percentages. For example, the Blades aggregate blend used in the mixture design (to be discussed in Chapter 5) only contained 3.3% of the ¾” to ½” sample size by total weight of the coarse aggregate fraction of the JMF.

Therefore, this sample size was weighted accordingly to better represent the JMF used. The sample calculation used is shown as Equation 4.1. This equation was used to determine the normalized Texture and Angularity Index for each aggregate source.

Table 4.1 – AIMS Texture and Angularity Indexes for Source Aggregates in Study

Sample Name	Sample Size	Texture Index	Angularity Index
G1	3/4 - 1/2	244.29	2178.92
	1/2 - 3/8	219.08	2150.25
	1/4 - #4	208.19	2356.47
G2	3/4 - ½ (only 36 particles)	278.86	2400.06
	1/2 - 3/8	200.09	2312.84
	1/4 - #4	188.25	2437.79
G4	3/4 - 1/2	351.74	2245.82
	1/2 - 3/8	308.38	2245.58
	1/4 - #4	251.19	2304.60
G3	3/4 - 1/2	279.28	2391.54
	1/2 - 3/8	224.22	2425.33
	1/4 - #4	219.55	2515.61
R1	3/4 - 1/2	446.54	2780.79
	1/2 - 3/8	410.38	2933.36
	1/4 - #4	311.35	3106.19
R3	3/4 - 1/2	488.5	2915.50
	1/2 - 3/8	448	2806.00
	1/4 - #4	365.31	3131.25
R2	3/4 - 1/2	463.98	2914.29
	1/2 - 3/8	442.94	2977.09
	1/4 - #4	420.51	3058.53

$$\text{Normalized Index} = \left(\frac{1.2}{37.9}\right)\left({}^{3}\!/_{4}\text{ to }{}^{1}\!/_{2}"\right) + \left(\frac{9.0}{37.9}\right)\left({}^{1}\!/_{2}\text{ to }{}^{3}\!/_{8}"\right) + \left(\frac{27.7}{37.9}\right)\left({}^{1}\!/_{4}"\text{ to No.4}\right) \quad (4.1)$$

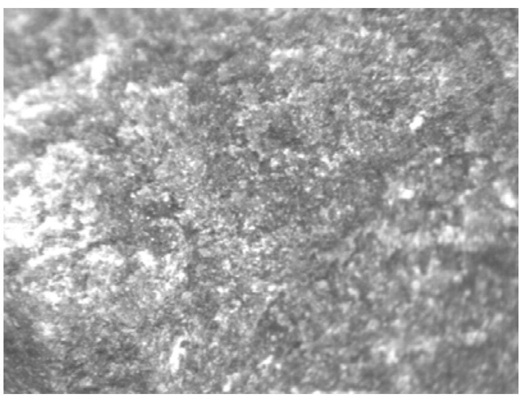
where,

¾ to ½” = AIMS Index Value for Sample Size of ¾ to ½”
½ to 3/8” = AIMS Index Value for Sample Size of ½ to 3/8”
¼” to No. 4 = AIMS Index Value for Sample Size of ¼” to No.4
1.2, 9.0, 27.7% = % of aggregate retained on respective sample size for G1 JMF
37.9% = % of aggregate coarser than the No. 4 Sieve for the G1 JMF

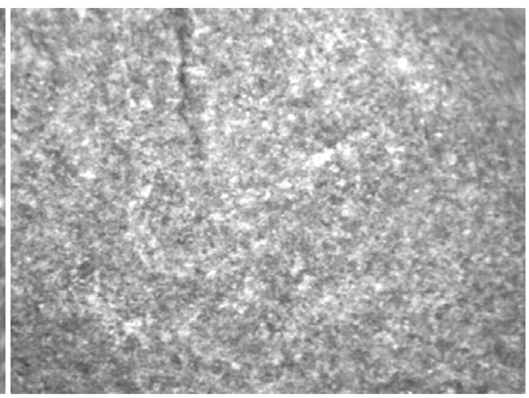
The final normalized Texture and Angularity Index values for the different aggregate sources are shown in Table 4.2 and Figures 4.9 and 4.10, respectively. The AIMS results

G1 - ¾” - ½” Angularity = 1175.84

G1 - ¾” - ½” Angularity = 2897.35



G1 - ¾” - ½” Texture = 253.5

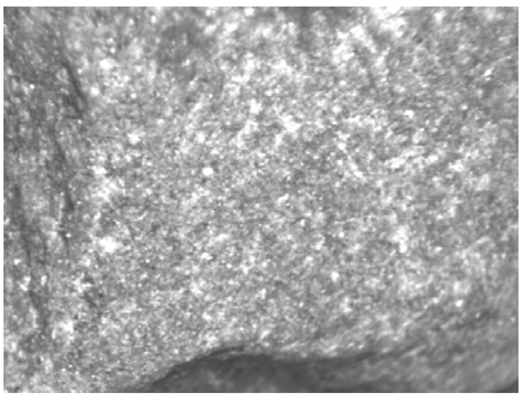


G1 - ¾” - ½” Texture = 217

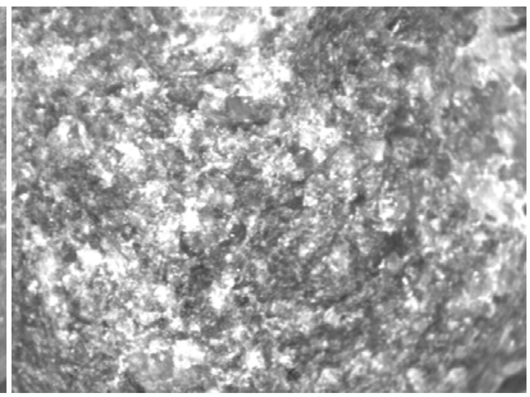
Figure 4.5 – AIMS Angularity and Texture Index for G1 Gravel

G2 - ¾” - ½” Angularity = 2512.50

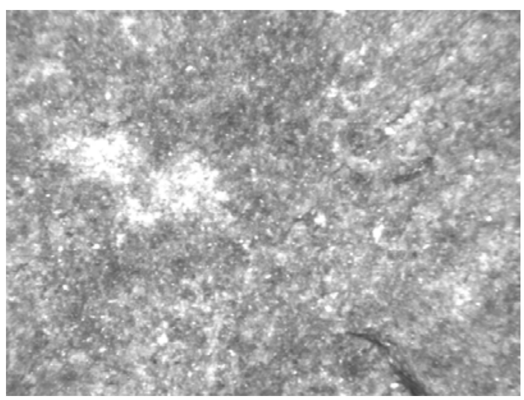
G2 - ¾” - ½” Angularity = 3433.08



G2 - ¾” - ½” Texture = 244



G2 - ¾” - ½” Texture = 488

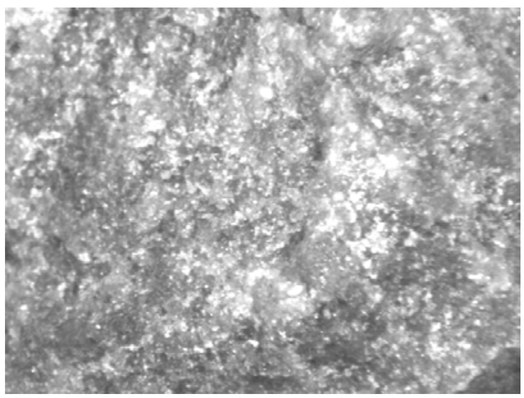


G2 - ¾” - ½” Texture = 208.5

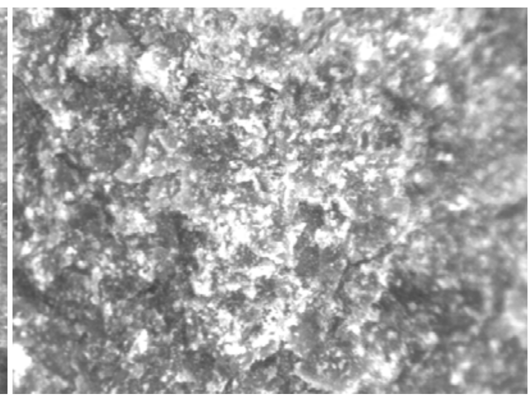
Figure 4.6 – AIMS Angularity and Texture Index for G2 Gravel

G3 - ¾” - ½” Angularity = 3128.09

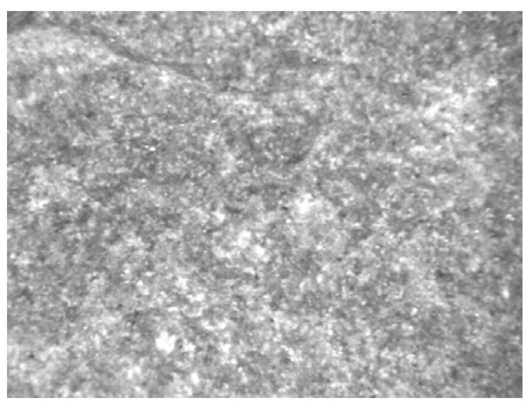
G3 - ¾” - ½” Angularity = 2259.07



G3 - ¾” - ½” Texture = 298



G3 - ¾” - ½” Texture = 464

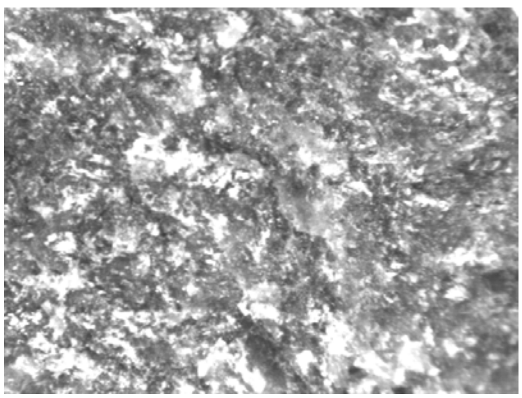


G3 - ¾” - ½” Texture = 217.5

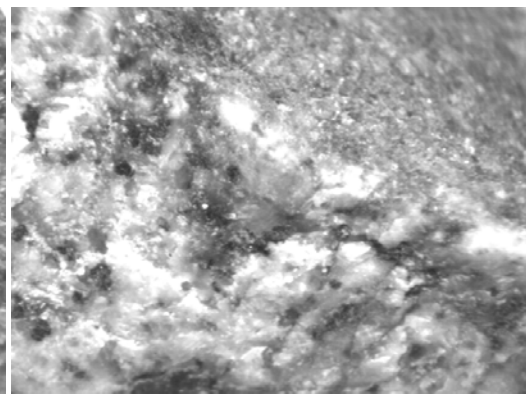
Figure 4.7– AIMS Angularity and Texture Index for G3 Gravel

R1 - ¾” - ½” Angularity = 3447.6

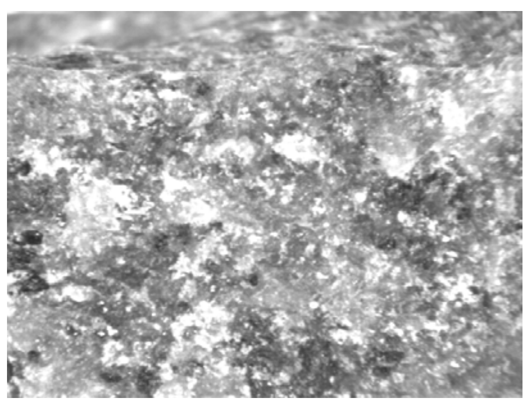
R1 - ¾” - ½” Angularity = 2676.83



R1 - ¾” - ½” Texture = 658.5



R1 - ¾” - ½” Texture = 351



R1 - ¾” - ½” Texture = 457.5

Figure 4.8– AIMS Angularity and Texture Index for R1 Aggregate

Table 4.2 – Normalized AIMS Results

Aggregate Source	Normalized Texture Index	Normalized Angularity Index
R3	401.9	3000.9
R1	351.1	3031.7
R2	444.3	2975.9
G1	212.5	2292.1
G2	193.6	2403.1
G4	242.1	2604.5
G3	223.4	2484.3

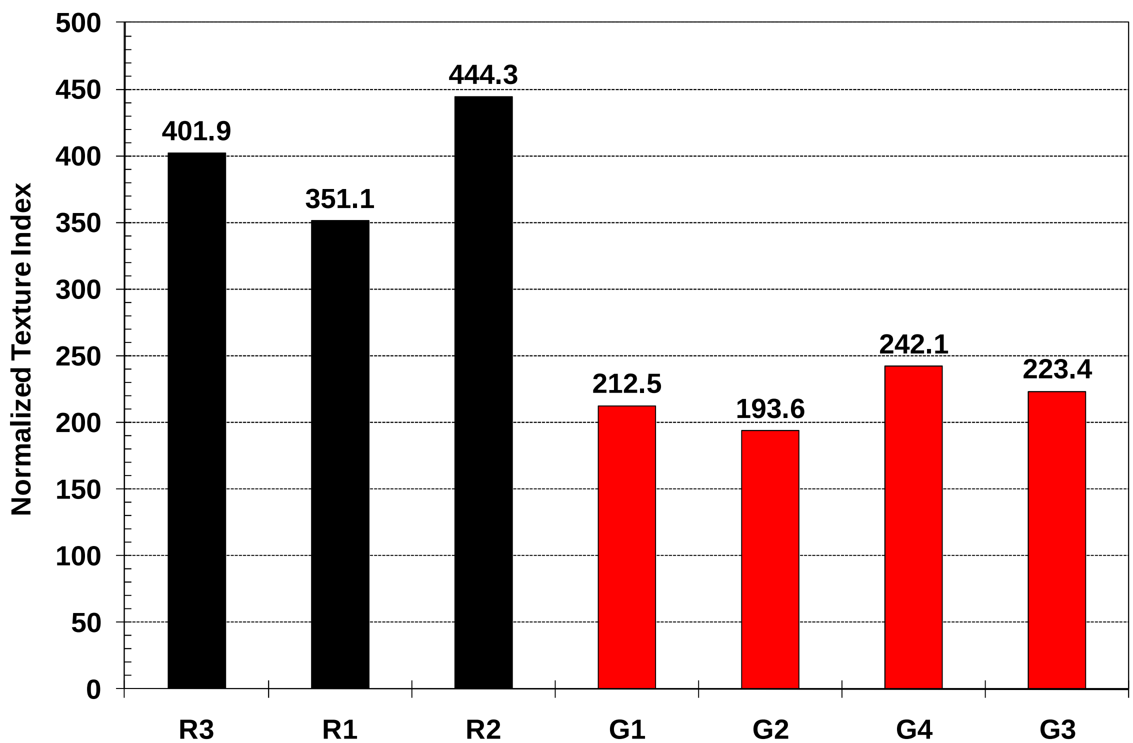


Figure 4.9 – Normalized AIMS Texture Index Results

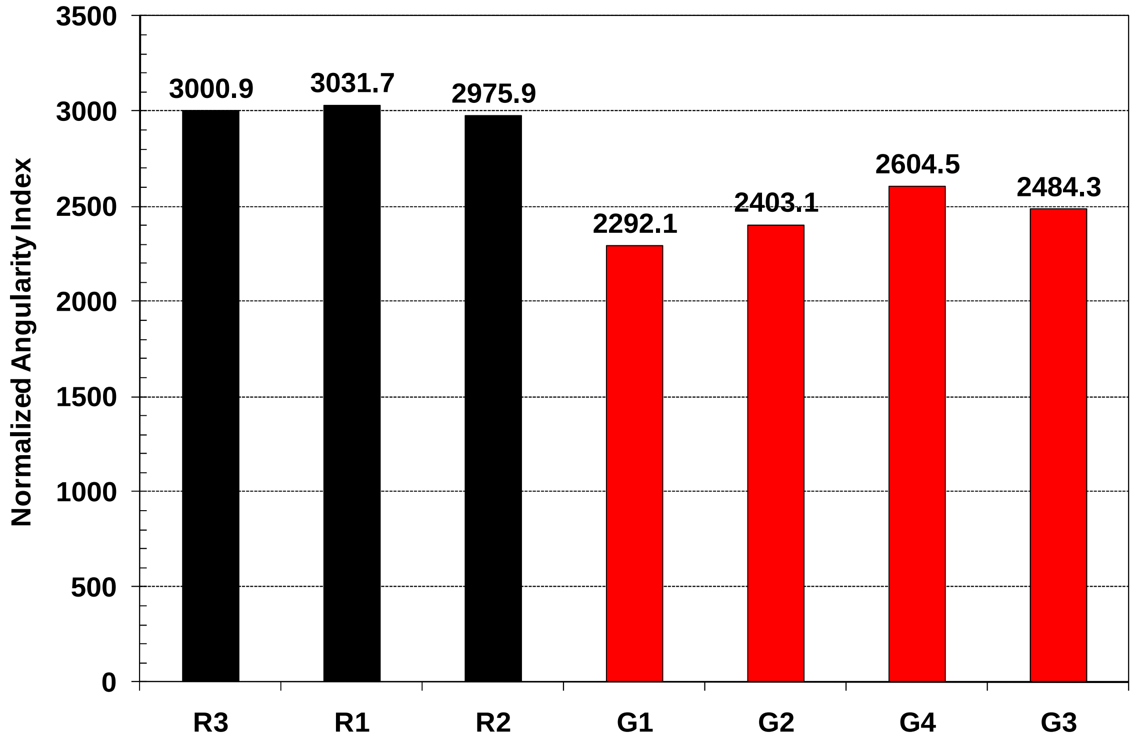


Figure 4.10 – Normalized AIMS Angularity Index Results

clearly indicates that the texture of the gravels were far inferior to that of the crushed stone sources (approximately half of that of the crushed stone sources). Meanwhile, the Angularity Index of the gravel sources were much closer to that of the crushed stone sources. This indicates that the general crushing procedures utilized by the gravel suppliers adequately created angularity. However, since not all of the gravel surface area is crushed, the weathered and polished surface of the gravel significantly reduced the AIMS Texture Index.

4.2 – Relationship Between Angularity Measurements

The inclusion of the AIMS testing provided a good means of evaluating the sensitivity and "accuracy" of ASTM D5821 and AASHTO T326 with respect to adequately indexing the angularity/texture of the different aggregate sources. Since the AIMS test quantitatively measures angularity and texture independently, without the potential bias of user error or judgment, it provides a good comparison to evaluate how well ASTM D5821 and AASHTO T326 quantifies angularity and texture of crushed aggregates.

Figure 4.11 shows the relationships between the AIMS Texture Index and the results of ASTM D5821 and AASHTO T326. The results show a relatively poor correlation between the ASTM D5821 and AIMS Texture Index. Meanwhile, a good correlation was found between the uncompacted voids content of AASHTO T326 and the AIMS Texture Index. This indicates that the uncompacted voids content is highly related to the surface texture of the aggregates evaluated.

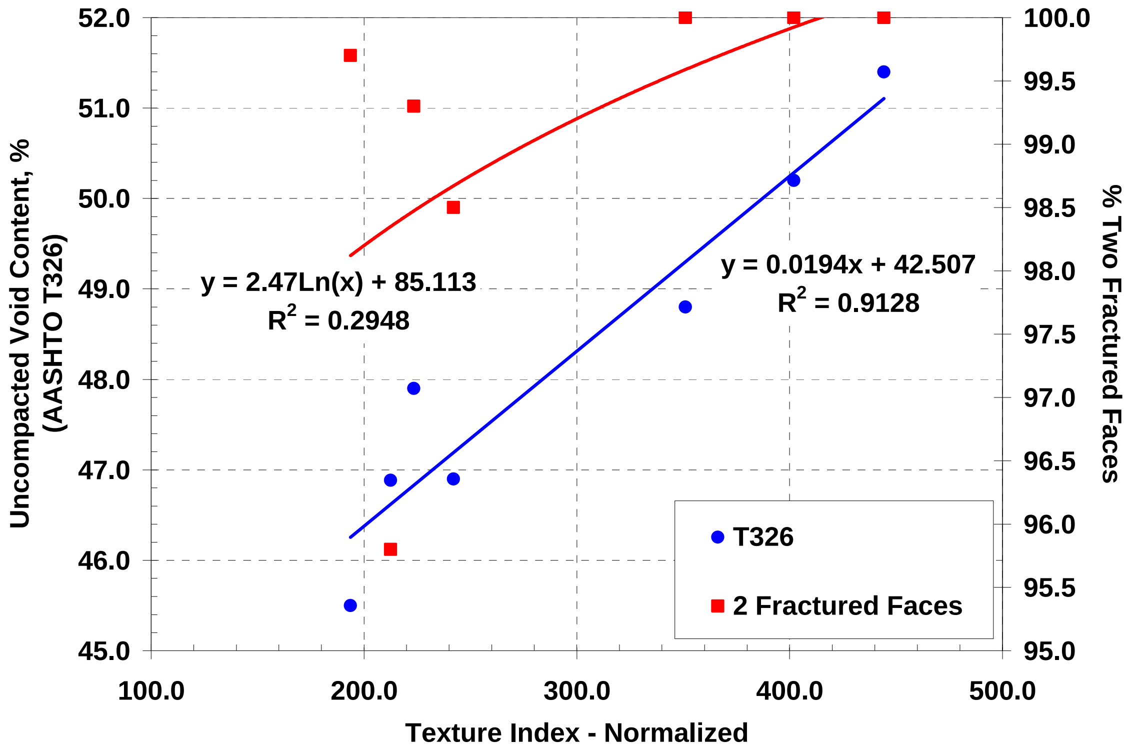


Figure 4.11 – Relationships Between AIMS Texture Index and Coarse Aggregate Angularity Measured in ASTM D5821 and AASHTO T326

Figure 4.12 shows the relationship between the AIMS Angularity Index and the coarse aggregate angularity as measured by ASTM D5821 and AASHTO T326. The figure shows a better, but still poor, relationship between ASTM D5821 and the AIMS Angularity Index. A better correlation was found between AASHTO T326 and the AIMS Angularity Index, although not as strong as what was previously shown for the Texture Index.

The comparisons between the AIMS testing and ASTM D5821 and AASHTO T326 indicate the following:

1. The coarse aggregate angularity, as determined using ASTM D5821, does not seem to represent the physical angularity and texture of the coarse aggregates. Based on the comparisons between the AIMS testing (Angularity Index and Texture Index), poor comparisons were found when comparing the Fractured Face Count results of ASTM D5821.
2. The uncompacted voids content of AASHTO T326 was strongly correlated to the Texture Index of the AIMS device. This would indicate that aggregate surface texture plays a significant role in the uncompacted voids measurement.
3. The uncompacted voids content of AASHTO T326 was also correlated to the Angularity Index of the AIMS device. However, the correlation was not as strong as the Texture Index, perhaps indicating that angularity plays a secondary role to surface texture with respect to the uncompacted voids measurement.

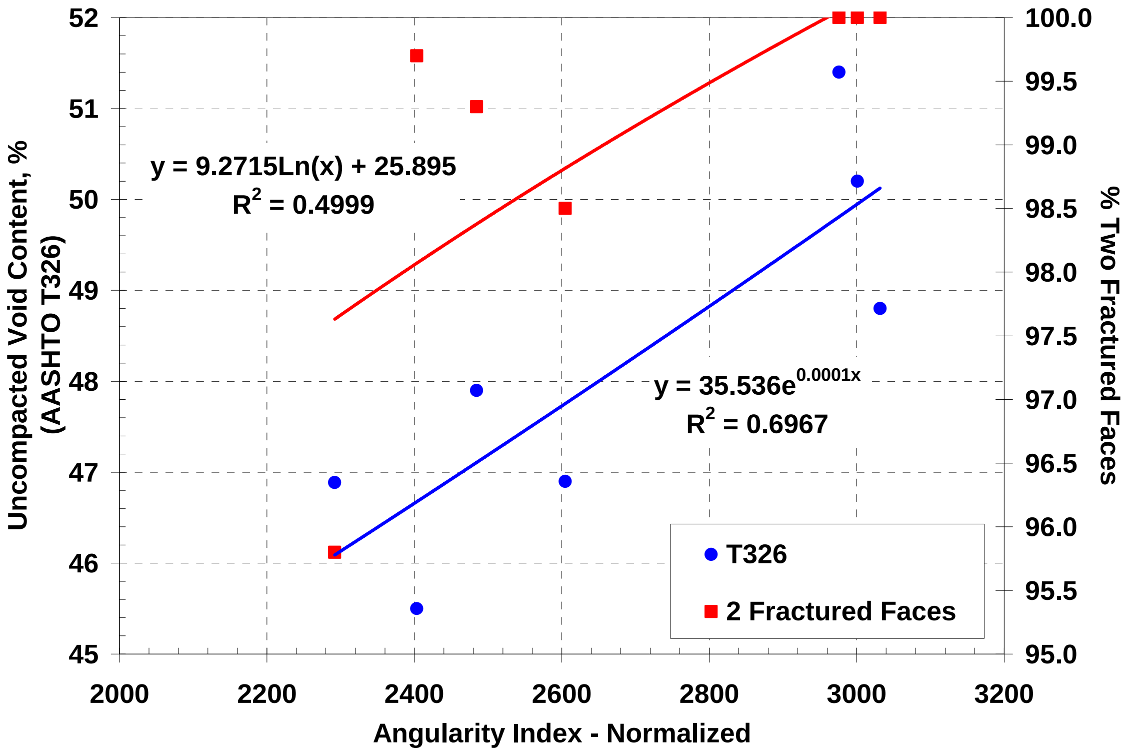


Figure 4.12 – Relationships Between AIMS Angularity Index and Coarse Aggregate Angularity Measured in ASTM D5821 and AASHTO T326

4.3 – Micro-Deval Testing

The Superpave mixture design method did not specify a test method to evaluate the abrasion of aggregates during production, construction and/or under traffic, although the sulfate soundness test evaluates disintegration of aggregates cause by environmental exposure. Several studies have evaluated the Micro-Deval test for inclusion as a durability test for aggregates. In the Micro-Deval test, the aggregate is loaded in a steel jar with water and a charge of steel shot and then rotated at 100 RPM for two hours. The test is not an impact test; however, as the aggregate breaks down, abrasive slurry is created in addition to the steel shot. Figure 4.13 show a picture of the Micro-Deval test apparatus.

The Micro-Deval test was conducted in accordance with ASTM D6928 to determine the abrasion properties of the aggregates. The final results are shown as Table 4.3. On average, the test results indicate that the gravel sources had a Micro-Deval abrasion mass loss (13.1%) almost three times higher than the crushed stone sources (5.8%). However, both types of aggregates met the preliminary recommendation of 18% maximum mass loss (Kandhal and Parker, 1998).

Figure 4.13 – Picture of the Micro-Deval Test Apparatus

Table 4.3 – Micro-Deval Test Results for Aggregate Sources in Study

Aggregate Source	Micro-Deval Percent Loss (%)
G1	13.3
G2	11.6
G3	13
G4	14.5
R1	5.2
R2	6.4
R3	5.7

4.4 – Polishing Potential Using AIMS-Based Procedure

The Texas Transportation Institute (TTI) developed a methodology to quantify the potential for aggregate polishing. The procedure utilizes a combination of the Micro-Deval test and the Aggregate Imaging System (AIMS). The methodology is as follows:

1. Conduct the AIMS testing on the aggregate source;
2. Using the same sample source, conduct the Micro-Deval test in accordance with ASTM D6928;
3. Conduct the AIMS testing again on the aggregate sample recently tested in the Micro-Deval; and
4. Determine the percent difference of the Angularity Index of the Before and After Micro-Deval test.

The test results of Polishing Potential are shown in Table 4.4. The test results do show that the use of the Micro-Deval provides an abrading of the aggregate samples that is clearly distinguishable in the AIMS device. When comparing the gravel sources to the crushed stone sources, the average percent reduction in the angularity measurements were similar, with the gravel and crushed stone sources resulting in a 27.7% and 24.2% reduction in the Angularity Index, respectively. Meanwhile, there was a significant difference in the percent reduction of the Texture Index. The average percent reduction of the Texture Index for the gravel sources was 12.7% while the crushed stone sources had a 26.6% reduction in the Texture Index. However, upon further review of the data in Table 4.4, even with the 26.6% reduction in the Texture Index, the crush stone sources still achieved higher levels in the Texture Index post-Micro-Deval than the gravel sources did pre-Micro-Deval.

Table 4.4 – Polishing Potential Results of Aggregate Sources

Aggregate Source	Sample Size	Texture Index			Angularity Index		
		Before Micro-Deval	After Micro-Deval	Percent Reduction	Before Micro-Deval	After Micro-Deval	Percent Reduction
G1	1/2" to 3/4"	219.1	205.1	6.4%	2150.3	1665.8	22.5%
	1/4" to No.4	208.2	159.7	23.3%	2356.5	1860.6	21.0%
G2	1/2" to 3/4"	200.1	200.8	-0.3%	2312.8	1670.4	27.8%
	1/4" to No.4	188.3	160.9	14.6%	2437.8	1781.2	26.9%
G3	1/2" to 3/4"	224.2	218.8	2.4%	2425.3	1822.6	24.9%
	1/4" to No.4	219.6	166.2	24.3%	2515.6	1774	29.5%
G4	1/2" to 3/4"	227.9	205.3	9.9%	2568.7	1822.3	29.1%
	1/4" to No.4	247.8	196.3	20.8%	2626.3	1574.6	40.0%
R1	1/2" to 3/4"	410.4	347.5	15.3%	2933.4	2621.9	10.6%
	1/4" to No.4	311.3	245.5	21.1%	3106.2	2641.7	15.0%
R2	1/2" to 3/4"	442.9	351.8	20.6%	2914.3	2057.2	29.4%
	1/4" to No.4	420.5	360.6	14.2%	2977.1	2005.5	32.6%
R3	1/2" to 3/4"	448	272.4	39.2%	2806	2120.6	24.4%
	1/4" to No.4	365.3	185.9	49.1%	3131.3	2098.4	33.0%

In the development of the Polishing Potential methodology, the Texas Transportation Institute also developed a chart to help classify the abrasion and breakage potential of aggregates. The chart is currently being evaluated by the Texas Department of Transportation for selection of aggregates to be used in surface course mixtures. The chart utilizes both the Micro-Deval mass loss results and the percent reduction in the AIMS aggregate angularity. The chart was reproduced in this study with the data from the aggregate sources included (Figure 4.14). Figure 4.14 indicates that most of the aggregate sources tested would be in the optimal area of “Low Abrasion/Breakage”. However, two of the aggregate sources, one gravel source (G3) and one crushed stone source (R2), did indicate that there could be a potential for high abrasion loss. This would indicate that angularity could be lost over time resulting in a potential reduction in surface friction of the pavement surface.

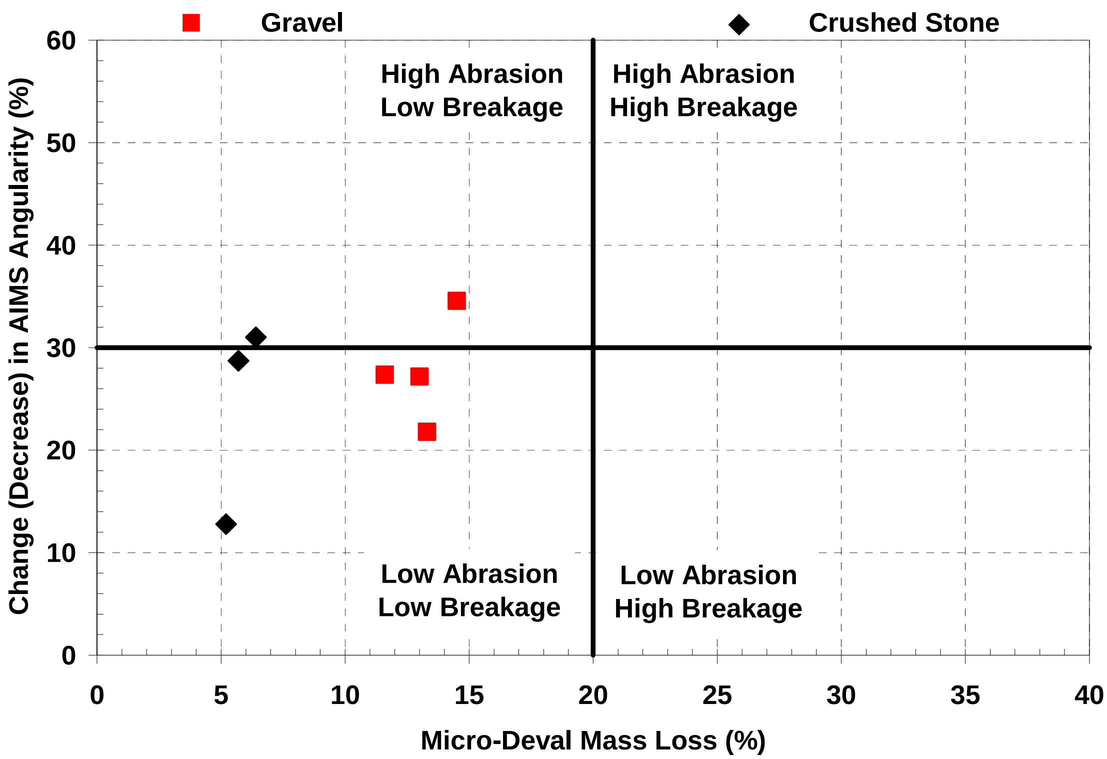


Figure 4.14 – TTI Polishing Potential of Aggregate Sources in Study

4.5 – Angularity Properties of Gravel Blends

The original idea of the study was to sample different gravel sources of various fractured face counts to determine how the different fractured face counts influenced the rutting properties of the asphalt mixtures. Unfortunately, while investigating different gravel sources, it was evident that a majority of the gravel sources had very similar fractured face counts, as determined using ASTM D5821. It was then decided to "artificially" change the fractured face count by blending in "rounded", or uncrushed, gravel to change the fractured face count properties. The final matrix of fractured face counts are shown in Table 4.5. The table is assuming that the fractured face counts should all change equally in accordance with the original fractured face counts provided by the NYSDOT. It was agreed upon from the NYSDOT Technical Working Group that the addition of the uncrushed gravel would be at three percent increments, with equal blending occurring from the 1 and 1A sources (i.e. – 1.5% uncrushed 1's and 1.5% uncrushed 1A's). These blends, along with the 100% crushed gravels, would then be used in the rutting evaluation, discussed later in Chapter 5, to assess the rutting potential that results in changing the fracture face count of the coarse aggregates.

In conjunction with the assumed fractured face counts shown in Table 4.5, the uncompacted void content of the different blends, as well as the AIMS Texture and Angularity Index, were also evaluated.

Table 4.5 – Proposed Coarse Aggregate Angularity Blends

ASTM D5821 Fractured Face Counts (Based on Theoretical Blending)			
G1	G2	G3	G4
100/96	100/99	100/99	100/99
97/93	97/96	97/96	97/96
94/90	94/93	94/93	94/93
91/87	91/90	91/90	91/90
88/84	88/87	88/87	88/87

4.5.1 – Uncompacted Void Content (AASHTO T326) of Gravel Blends

Each of the aggregate blends (crushed and uncrushed aggregates) was tested in accordance to AASHTO T326, *Uncompacted Voids Content of Coarse Aggregates*. The test results, compared to the fracture face counts, are shown in Table 4.6. The results of the uncompacted voids content shows that as the two faced fractured count decreases, so does the uncompacted voids content. This would be expected since the addition of the uncrushed gravel would certainly decrease the texture and angularity of the gravel blend. Attempts were made to evaluate the correlation between the two-face fractured counts and the uncompacted voids content. However, even an average correlation was not able to be developed when pooling all of the data points (Figure 4.15). It is apparent from Figure 4.15 that even though a correlation does not exist when pooling the data, there certainly exists a relationship among each gravel source (Figure 4.16). This may indicate that factors such as the size of the fracture face, texture and angularity of the fracture face, and shape characteristic of the gravel (i.e. – flat, elongated, etc.) may also play a role in the measurement of the uncompacted voids content.

Table 4.6 – Uncompacted Voids Content of Gravels of Varying Crushed Counts

Gravel Source	Percent One-Faced Crushed Count	Percent Two-Faced Crushed Count	Uncompacted Voids Content (%)
G1	88	83.8	46.5
	91	86.8	46.8
	94	89.8	46.6
	97	92.8	46.7
	100	95.8	46.8
G2	88	87.7	44.7
	91	90.7	44.7
	94	93.7	45.3
	97	96.7	45.5
	100	99.7	45.5
G3	88	87.3	47.4
	91	90.3	47.4
	94	93.3	47.3
	97	96.3	47.5
	100	99.3	47.9
G4	88	87.3	45.9
	91	90.3	46.3
	94	93.3	46.6
	97	96.3	46.7
	100	99.3	46.9
R3	100	100	50.2
R1	100	100	48.8
R2	100	100	51.4

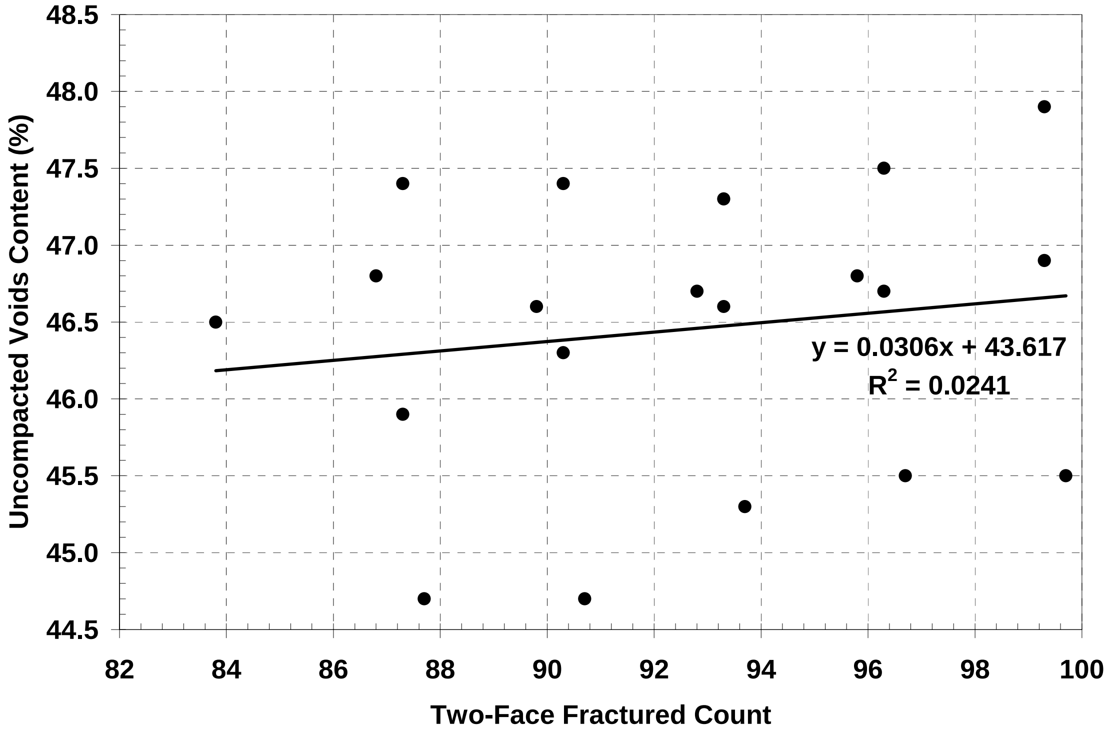


Figure 4.15 – Uncompacted Voids Content vs Two-Face Fractured Count for All Gravels

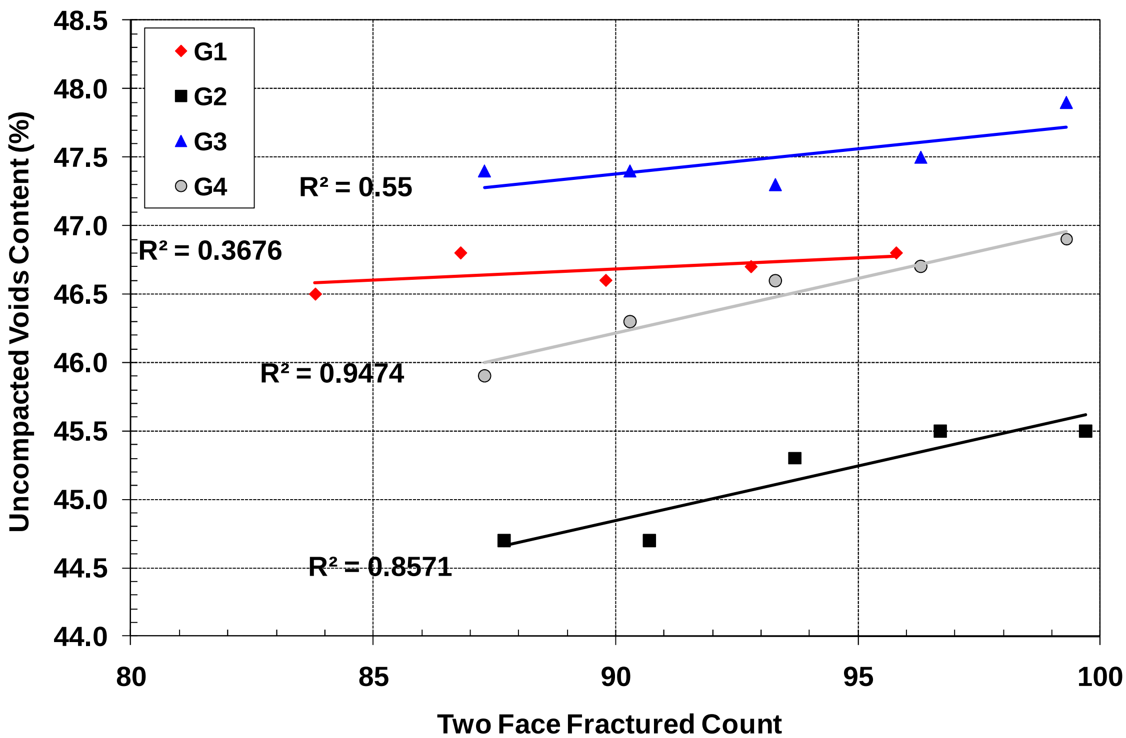


Figure 4.16 – Gravel Source Relationship Between Uncompacted Voids Content and Two-Face Fractured Count

4.5.2 – AIMS Texture and Angularity Index of Gravel Blends

Similar to the uncompacted voids content, the AIMS device was used to evaluate the texture and angularity of the different gravel blends. The resultant AIMS Texture and Angularity Index, along with the two face fractured counts, are shown in Table 4.7. Pictures from the AIMS testing can be found in Appendix A of the report. In general, it is observed that as the two face fractured count decreases, so does the AIMS Texture and Angularity Index. However, as shown earlier in the uncompacted voids comparison, there does not seem to exist a strong relationship between the two face fractured count and the AIMS Texture and Angularity Indexes (Figure 4.17 and 4.18).

The relationship between the two face fractured count and the AIMS Texture and Angularity Index was also evaluated for each gravel source separately to determine if better correlations existed within each gravel source as opposed to pooling all of the data. This result of this is shown in Figure 4.19 and 4.20. The figures show a much better relationship between AIMS Texture Index and the two face fractured counts when evaluating each gravel source separately (Figure 4.19). This is consistent with what was shown earlier with respect to the uncompacted voids and two face fractured counts for each gravel source (Figure 4.16). However, when evaluating the AIMS Angularity

Table 4.7 – AIMS Texture and Angularity Index of Gravel Blends

Gravel Source	Percent One-Faced	Average		Gradation Weighted	
		Texture	Angularity	Texture	Angularity
G1	88	266.9	2315.4	243.6	2341.0
	91	258.3	2205.9	237.5	2213.9
	94	268.1	2234.4	248.8	2203.2
	97	267.9	2323.5	247.5	2348.1
	100	267.2	2228.6	247.3	2292.1
G2	88	293.3	2271.0	262.0	2328.2
	91	281.4	2169.7	274.4	2156.1
	94	287.3	2298.2	271.0	2365.3
	97	276.4	2244.9	265.2	2357.0
	100	222.4	2383.5	193.6	2403.1
G3	88	281.7	2318.1	253.6	2339.9
	91	272.7	2342.2	246.3	2213.6
	94	297.8	2439.7	269.7	2204.1
	97	288.9	2422.1	259.3	2346.9
	100	311.0	2447.5	293.4	2487.2
G4	88	298.0	2315.4	275.7	2346.9
	91	302.7	2205.9	272.3	2216.1
	94	301.9	2234.4	276.0	2202.4
	97	304.1	2323.5	274.7	2357.1
	100	303.8	2265.3	266.5	2290.3

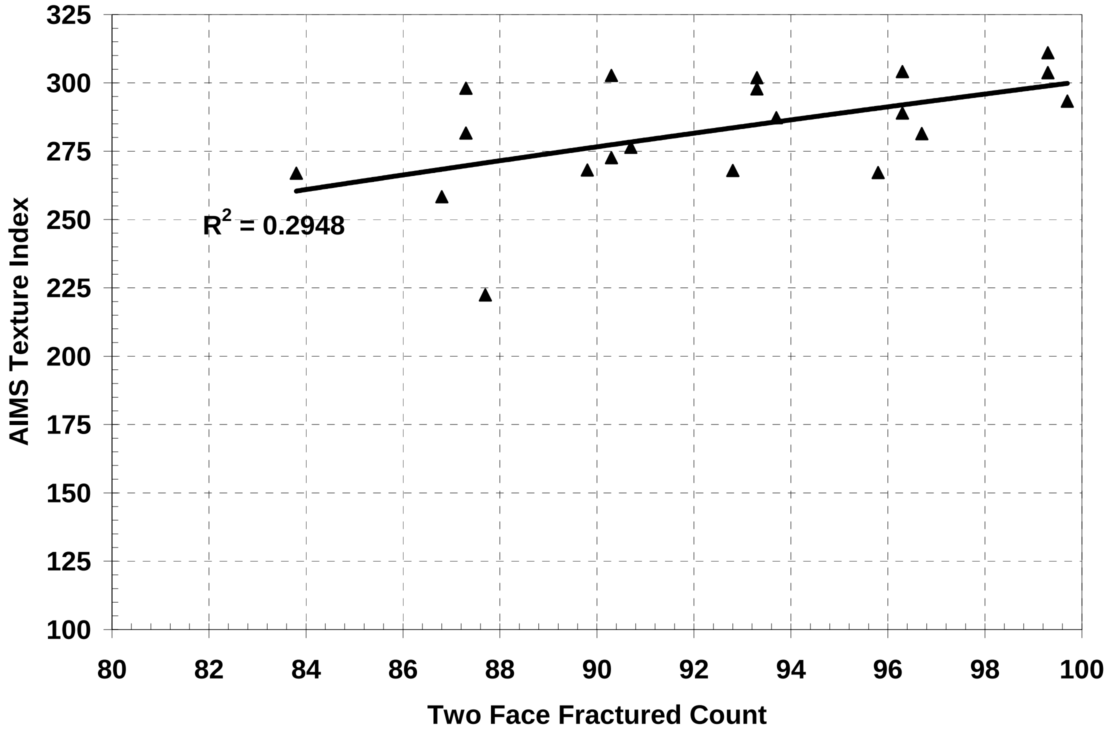


Figure 4.17 – Relationship Between Two Face Fracture Count and the AIMS Texture Index

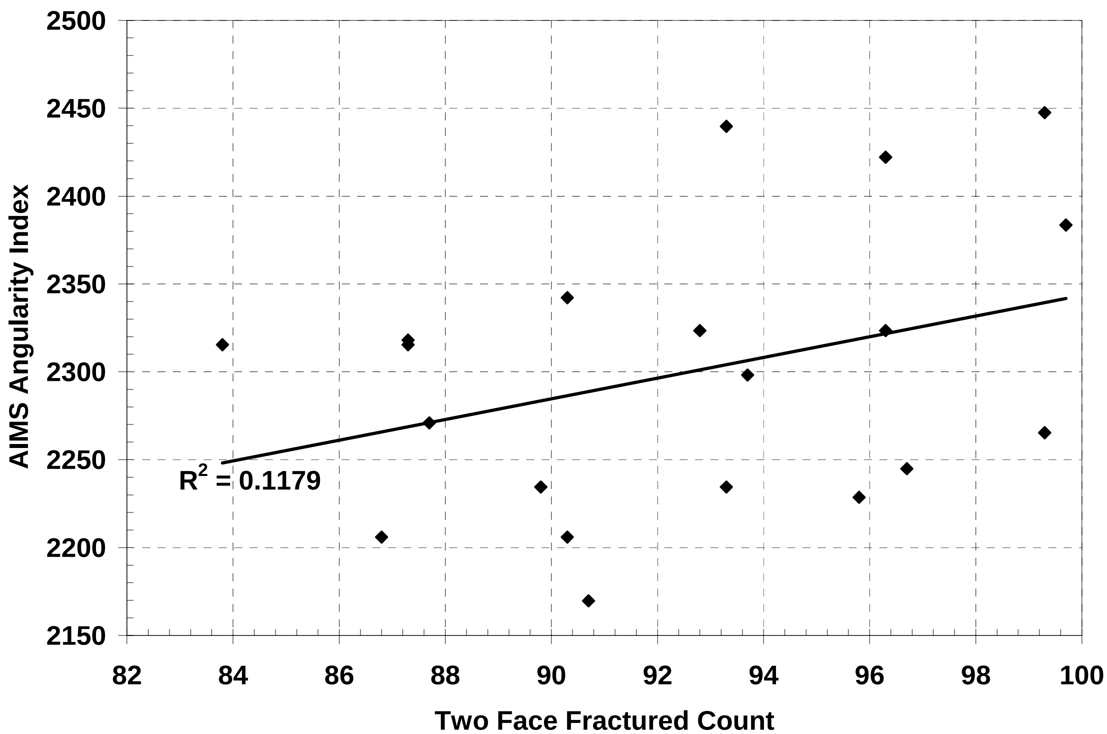


Figure 4.18 – Relationship Between Two Face Fracture Count and the AIMS Angularity Index

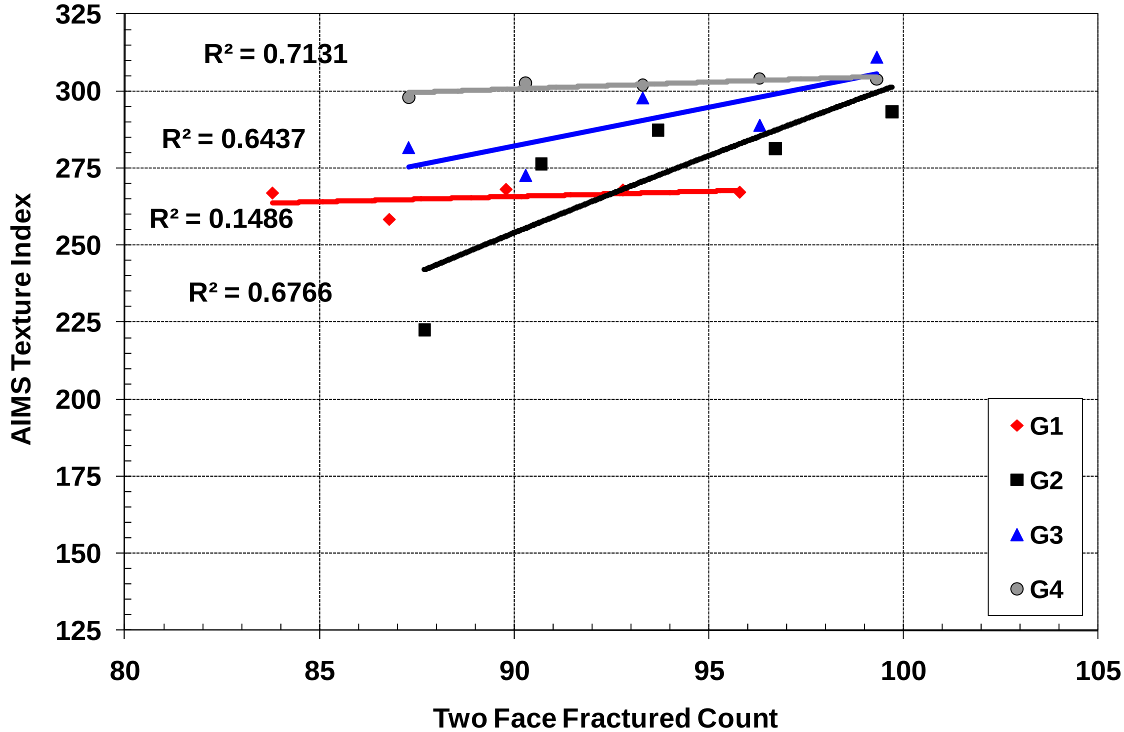


Figure 4.19 – Gravel Source Relationship Between AIMS Texture Index and Two Face Fractured Count

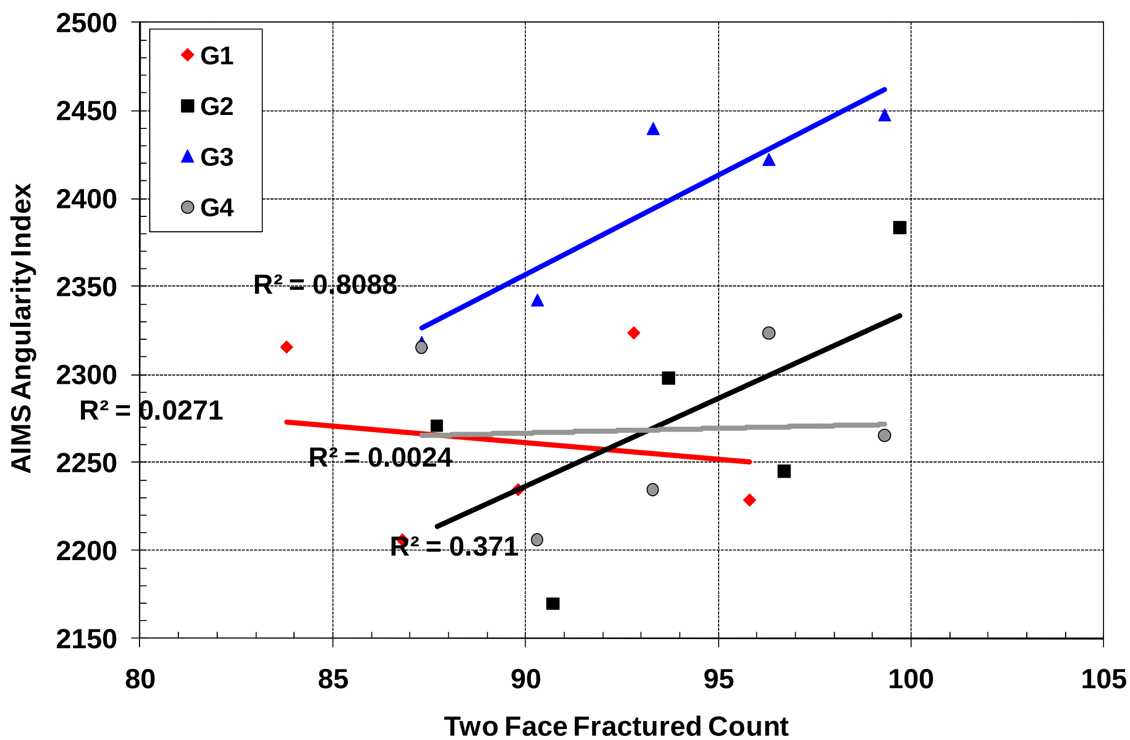


Figure 4.20 – Gravel Source Relationship Between AIMS Angularity Index and Two Face Fractured Count

Index and two face fracture count for each aggregate source, the relationships were much poorer (Figure 4.20).

4.5.3 – Relationship Between AIMS Texture and Angularity Testing to Uncompacted Voids Content

The last set of comparisons that were evaluated in the study was if any relationship existed between the AIMS Texture and Angularity Index and the Uncompacted Voids Content (AASHTO T326) for the different gravel blends. Figures 4.21 and 4.22 show the relationships developed with the datasets. For both the AIMS Texture Index and Angularity Index, it is apparent that a relationship does not exist. This was somewhat of a surprise considering the initial test results for the gravel and crushed stone sources clearly indicated a strong relationship between the AIMS Texture Index and Uncompacted Voids Content (AASHTO T326).

The dataset was then separated by gravel source and re-evaluated to determine if a "source-dependent" relationship existed. Figure 4.23 shows that a moderately strong relationship exists between the AIMS Texture Index and the Uncompacted Voids Content. However, Figure 4.24 indicates that a poor relationship exists between the AIMS Angularity Index and the Uncompacted Voids Content. These results compare favorably with the relationships previously shown in Section 4.2.

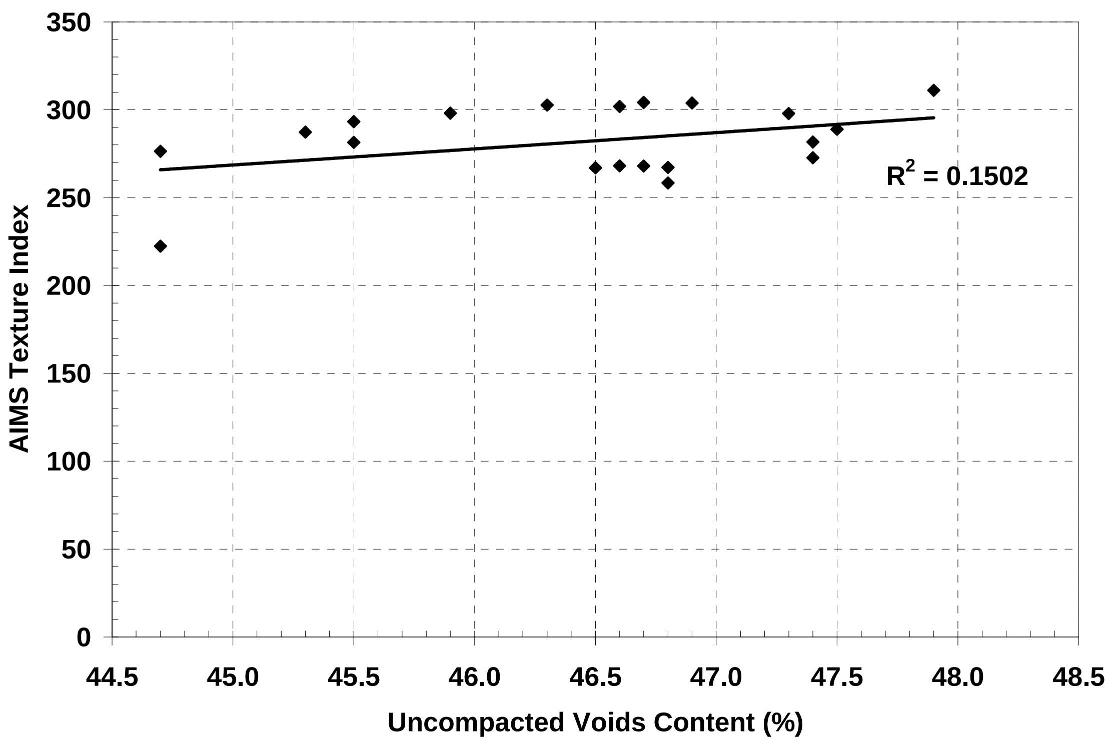


Figure 4.21 – Relationship Between AIMS Texture Index and the Uncompacted Voids Content of the Gravel Blends

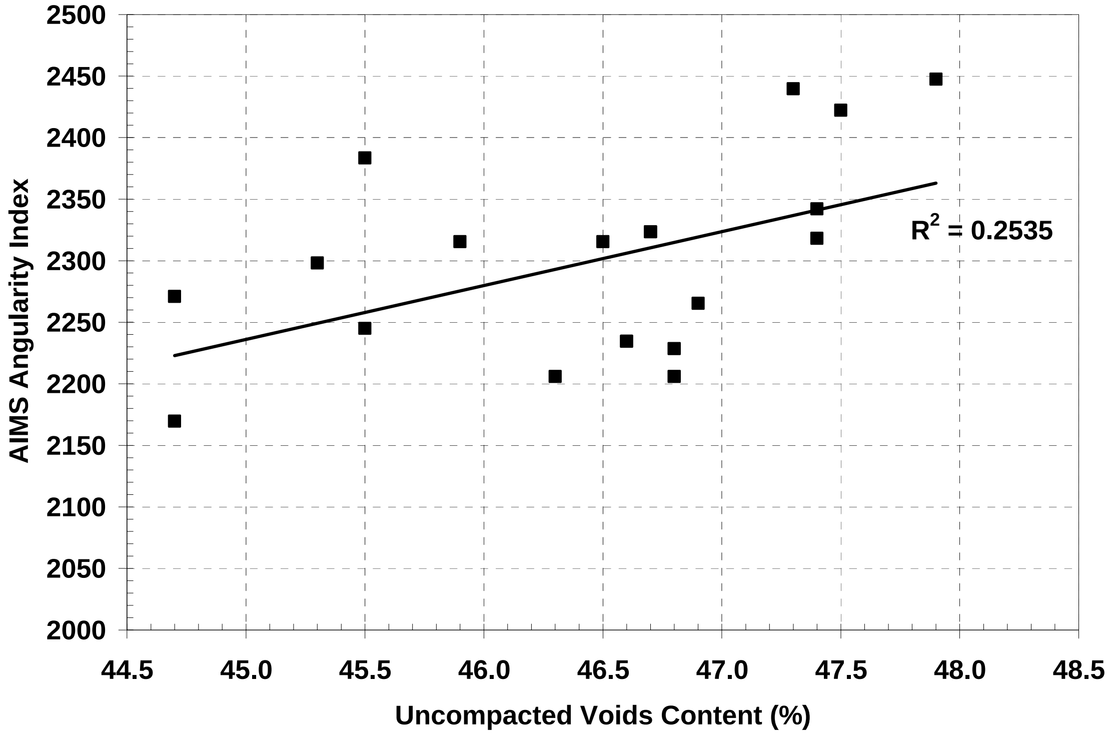


Figure 4.22 – Relationship Between AIMS Angularity Index and the Uncompacted Voids Content of the Gravel Blends

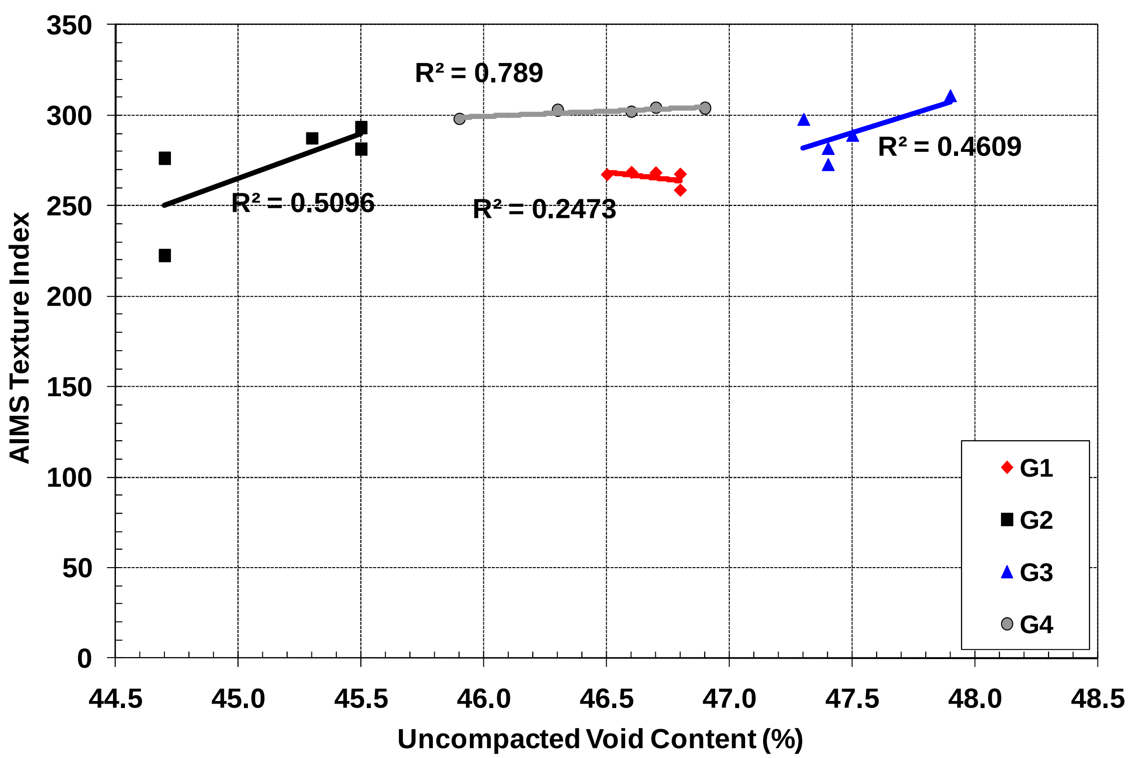


Figure 4.23 – Relationship Between AIMS Texture Index and Uncompacted Voids Content for Each Independent Gravel Source

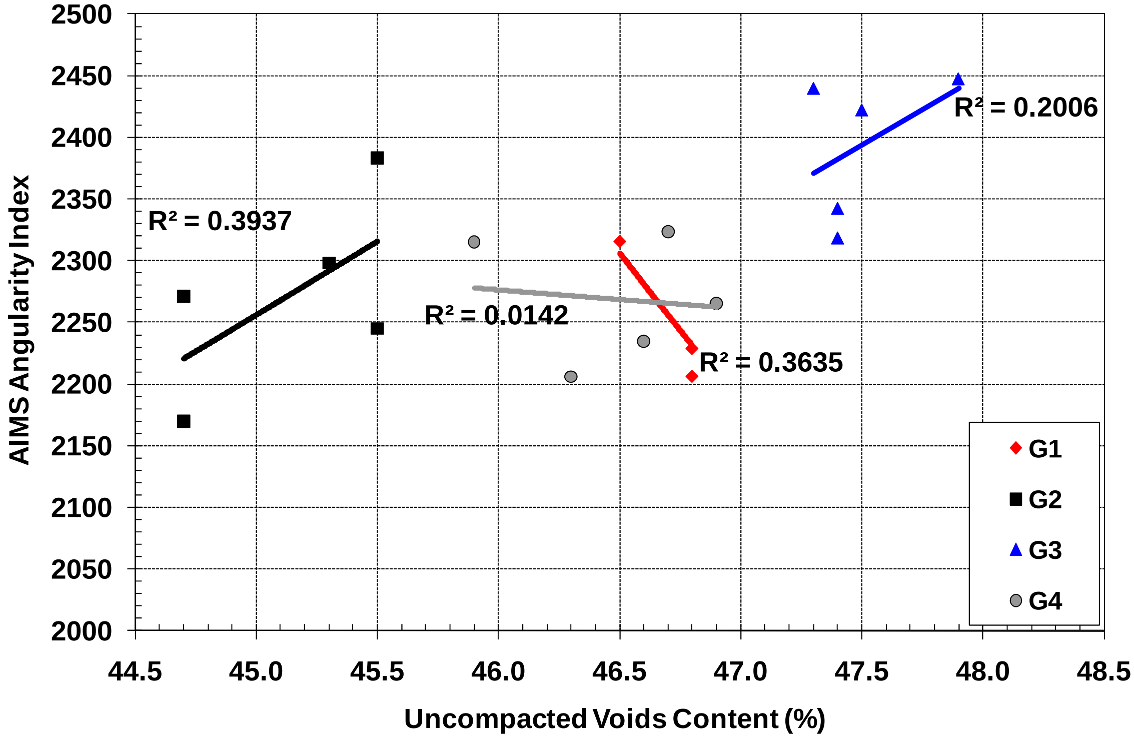


Figure 4.24 – Relationship Between AIMS Angularity Index and Uncompacted Voids Content for Each Independent Gravel Source

4.6 – Conclusions from Aggregate Testing

A variety of aggregate angularity tests were conducted to evaluate the angularity and texture properties of collected gravel and crushed stone aggregates utilized for hot mix asphalt. Currently, the New York State Department of Transportation (NYSDOT) utilizes ASTM D5821, *Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate* to characterize and accept coarse aggregates for hot mix asphalt production. Based on the aggregate testing conducted in the study, the following conclusions were drawn:

- The results of ASTM D5821 were found to have minimal correlation to aggregate texture and angularity measurements as determined with the Aggregate Imaging System (AIMS). The AIMS device provides an unbiased, image-based indexing of aggregate surface texture and angularity measurements using a combination of imaging and advanced analytical procedures.
- The results of ASTM D5821 were found to have minimal correlation to uncompacted voids content as determined using AASHTO T326. The uncompacted voids content of coarse aggregates had been identified under NCHRP Projects 4-19 and 4-19b as being strongly correlated to asphalt mixture rutting performance (i.e. – as the uncompacted voids content increased, mixture rutting decreased). The comparisons between the gravel sources actually

conflicted one another, where the gravel source with the highest content of two-face fractured faces achieved the lowest uncompacted voids content.

- Aggregate texture, as determined with the AIMS test, was found to be strongly correlated to the uncompacted voids content measurements – this was especially true when evaluating the source crushed stone and gravel samples. Correlations were still found with the blended gravel samples, however, this was only when comparing each gravel source separately.
- Aggregate angularity was found to have a lesser significant correlation, when comparing the AIMS Angularity Index and the uncompacted voids content of AASHTO T326.
- The evaluation of the polishing potential of the aggregates, as determined by Texas Transportation Institute (TTI) procedure shown in Section 4.4, identified the majority of the crushed stone and gravel sources in the study as low potential for polishing and breaking. However, the test procedure did indicate that some of the aggregates may be prone to polishing under significant traffic applications.

One potential reason for the lack of better correlation between the gravel blends and the AIMS Texture and Angularity Index is the 2-D manner in which the AIMS device measures these properties. If a crushed gravel has two crushed faces, exhibiting relatively good texture and angularity, the AIMS device may not recognize this if the aggregate is not placed properly on the AIMS imaging surface. This introduces a minor user bias within the test procedure that may have an effect overall results and comparisons to other test procedures, like the Uncompacted Voids Content test (AASHTO T326).

CHAPTER 5

HMA Mixture Design

HMA mixture designs were conducted in accordance with NYSDOT Materials Method: MM 5.16, *Superpave Hot Mix Asphalt Mixture Design and Mixture Verification Procedures*. Each of the coarse aggregates sources was blended with a single fine aggregate (5-33GFM) to construct the aggregate gradation. The 5-33GFM fine aggregate has a fine aggregate angularity (FAA) value of 46%, adhering to the requirements of NYS 5.16 for pavement with greater than 30 million ESAL's. It was decided to use a sole fine aggregate to help and eliminate confounding parameters that could influence permanent deformation properties of the asphalt mixtures. The addition of the fine aggregate was held between 37 and 42% in an attempt to keep aggregate blend gradation consistent for each mixture constructed. The final aggregate blend gradations are shown in Table 5.1. The third crushed stone source, R3, was not able to be utilized for the mixture design due to the irregular nature of the gradation of aggregates sampled. Several attempts were made to construct an aggregate blend using the natural gradations of the R3 aggregate stockpiles (#57 stone, #8 stone, and #10), however, significant differences were found on the 3/8 inch and the No. 4 sieve that could not be corrected to within an acceptable range (Table 5.2).

Table 5.1 – Aggregate Blend Gradations for Mixture Evaluated in the Study

Sieve Size (mm)	Specification		Percent Passing					
	Max	Min	G1	G2	G3	G4	R1	R2
50			100.0	100.0	100.0	100.0	100.0	100.0
37.5			100.0	100.0	100.0	100.0	100.0	100.0
25			100.0	100.0	100.0	100.0	100.0	100.0
19		100	100.0	100.0	100.0	100.0	100.0	100.0
12.5	100	90	98.0	99.7	99.2	98.8	100.0	99.9
9.4	90		87.9	91.4	93.7	89.8	94.8	94.3
4.75			57.2	59.3	63.1	62.1	61.8	48.3
2.36	58	31	32.5	31.7	33.2	31.7	32.9	31.5
1.18			21.0	21.5	21.7	19.8	20.8	20.7
0.60			15.3	16.4	15.9	14.5	15.0	15.1
0.30			11.2	12.4	11.9	10.3	10.4	10.7
0.150			8.2	9.0	8.9	7.0	6.8	7.2
0.075	10	2	7.2	6.9	7.3	6.6	5.3	5.7

For the >30 Million ESAL's traffic design, a design gyration level of 100 gyrations, as specified in NYSDOT Materials Method: MM 5.16, was used to determine the optimum asphalt content of the mixtures. Asphalt contents of the designed mixtures were also compared with typical asphalt mixture designs, containing gravels used in this study, from New York State Department of Transportation for comparison to ensure mixture volumetrics obtained were representative. Final volumetric properties for the mixtures are shown in Table 5.3.

Table 5.2 – R3 Aggregate Blend and Average JMF of Mixes in Study

Sieve Size (mm)	Specification		Percent Passing		
	Max	Min	R3	Ave JMF	Differences
50			100.0	100.0	0.0
37.5			100.0	100.0	0.0
25			100.0	100.0	0.0
19		100	100.0	100.0	0.0
12.5	100	90	97.0	98.9	-2.0
9.4	90		75.2	90.7	-15.5
4.75			40.6	60.4	-19.8
2.36	58	31	32.4	32.3	0.2
1.18			19.3	21.0	-1.7
0.60			11.9	15.5	-3.6
0.30			7.7	11.5	-3.7
0.150			5.2	8.3	-3.1
0.075	10	2	3.3	7.0	-3.7

Table 5.3 – Final Volumetrics for Mixture Designs Used in Study (Compacted to 3.5% Air Voids)

Aggregate Source	Asphalt Content (%)	Gmm (g/cm^3)	Gmb (g/cm^3)	Gsb (g/cm^3)	VMA	VFA
G1	6.4	2.424	2.339	2.616	16.3	78.5
G2	6.8	2.401	2.317	2.596	16.8	79.2
G3	6.9	2.387	2.303	2.561	16.3	78.5
G4	6.3	2.42	2.335	2.608	16.1	78.3
R1	7.5	2.482	2.395	2.585	14.3	75.5
R2	7.5	2.582	2.492	2.786	17.3	79.7
NYSDOT Spec	*> 5.2*	*N.A.*	*N.A.*	*N.A.*	*> 14*	*65 to 80*

All mixture designs were conducted using the unmodified (Neat) PG64-22 asphalt binder. Verification designs using the two polymer-modified asphalt binders were not conducted to expedite the project. Also, due to time constraints, moisture sensitivity testing (in accordance to AASHTO T283) was not conducted.

All performance samples were compacted to densities ranging between 94 and 93% of maximum specific gravity, G_{mm} (i.e. – 6 to 7% air voids). This range in density was chosen to represent typical in-place compacted densities in the pavement.

5.1 Asphalt Binder Properties

Three different asphalt binders were used during the mixture performance testing. Prior to conducting the mixture testing work, the asphalt binders were continuous PG graded (AASHTO M320 and R29) and tested using AASHTO TP70, *Multiple Stress Creep*

Recovery Test of Asphalt Binder Using a Dynamic Shear Rheometer. Table 5.4 contains the results of the asphalt binder testing. The results show that the high temperature PG grade, which would influence the rutting performance of the asphalt mixtures, increases from the PG64-22 to the PG64-22 (ER) to the PG76-22. The non-recoverable creep compliance (J_{nr}), measured at 64^{o}C, decreases in the same manner. The lower the Jnr value, the more rut resistant the asphalt binder.

Table 5.4 – Asphalt Binder and Respective Properties Used During Study

PG Binder Name	Continuous PG Grade			MSCR	
	High Temp, ^{o}C	Intermediate Temp, ^{o}C	Low Temp, ^{o}C	Jnr @ 3.2 kPa & 64C	% Recovery @ 3.2 kPa & 64C
64-22	69.3	20.4	-26.5	2.42	1.91
64ER	75.1	22.2	-26.8	0.80	20.75
76-22	77.4	21.6	-26.6	0.53	28.36

CHAPTER 6

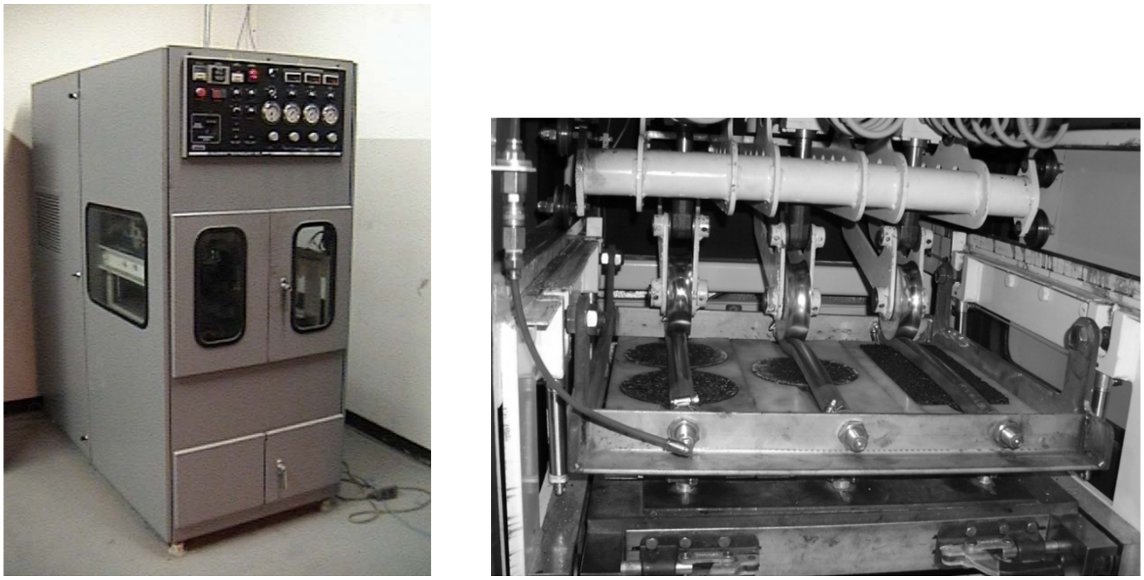
Permanent Deformation Testing

The permanent deformation (rutting) properties of the different mixtures were evaluated under two different tests; 1) Asphalt Pavement Analyzer and 2) Asphalt Mixture Performance Tester (AMPT) Flow Number. Along with testing the baseline asphalt mixtures discussed earlier in Chapter 5, variations of the designed asphalt mixtures were also tested with two additional asphalt binders. As discussed earlier in Chapter 5, the asphalt mixture designs were conducted using the current base asphalt binder for New York State, a PG64-22. However, it is common knowledge that the use of polymer modified asphalt binders has shown to improve the permanent deformation properties of asphalt mixtures. Therefore, it was decided to include two additional asphalt binders with each mix. This resulted in a total of three (3) different asphalt binders used for each asphalt mixture; 1) PG64-22, 2) PG64-22 meeting the NYSDOT Elastic Recovery specification, and 3) PG76-22. The purpose of including the additional two asphalt binders was to determine if perhaps a better asphalt binder could be used in-lieu of an aggregate blend that had lesser angularity properties. In total, 396 APA samples were compacted and tested. A total of 198 repeated load (Flow Number) samples were compacted and tested.

6.1 - Asphalt Pavement Analyzer (AASHTO TP63)

The Asphalt Pavement Analyzer (APA) was conducted in accordance with AASHTO TP63, *Determining Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer (APA).* A hose pressure of 120 psi and a wheel load of 120 lb were used in the testing. A test temperature of 58^{o}C was selected for testing to correspond with previous test temperatures used in earlier NYSDOT studies. Testing was continued until 8,000 loading cycles and APA rutting deformation was recorded at each cycle. The APA device used for testing at Rutgers University is shown in Figure 6.1.

Prior to testing, each sample was heated for 6 hours (+/- 15 minutes) at the testing temperature to ensure temperature equilibrium within the test specimen was achieved. Testing started with 25 cycles used as a seating load to eliminate any sample movement during testing. After the 25 seating cycles completed, the data acquisition began sampling test information until a final 8,000 loading cycles was reached. A typical test output is shown in Figure 6.2.



(a) (b)

Figure 6.1 – a) Asphalt Pavement Analyzer (APA) at Rutgers University; b) Inside the Asphalt Pavement Analyzer Device

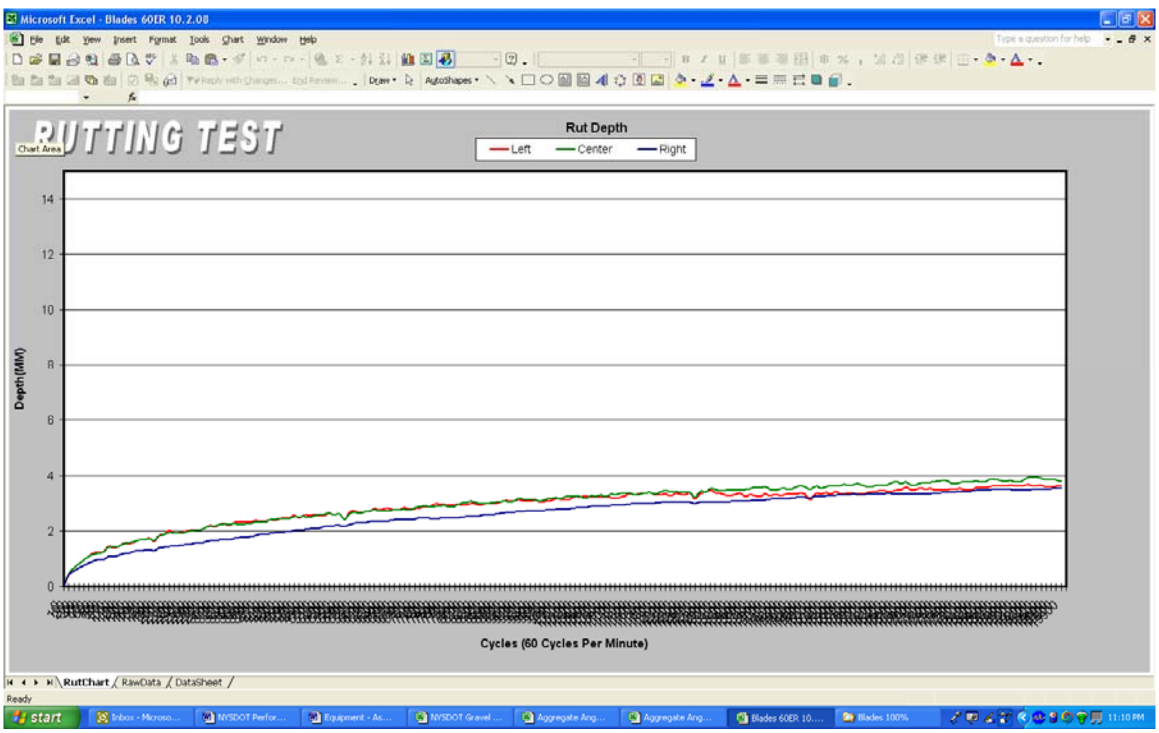
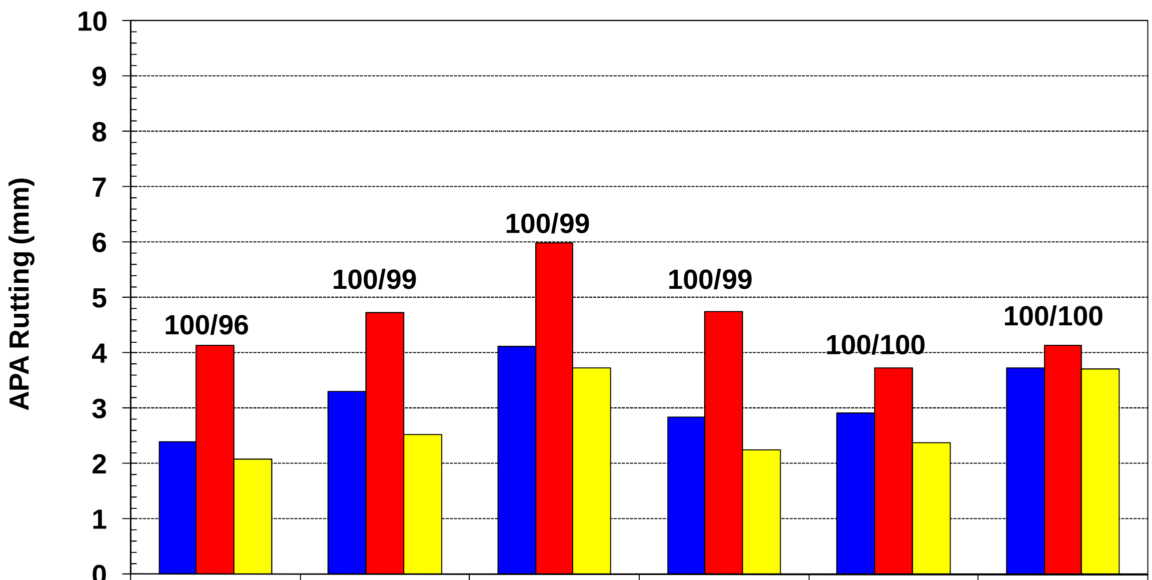


Figure 6.2 – Typical Graphical Output from the Asphalt Pavement Analyzer

6.1.1 – Asphalt Pavement Analyzer Results for Baseline Mixes

Each of the six baseline mixes, without gravel blending, were evaluated to determine the APA rutting performance. The test results for the mixes are shown in Figure 6.3 along with the aggregate's Coarse Aggregate Angularity (CAA) results as determined by ASTM D5821. The average test results clearly indicate that the crushed gravels provide comparable APA rutting resistance to the crushed stone (R1 and R2) mixtures. However, the CAA of the aggregate did not necessary match the respective APA rutting results, especially when the polymer modified binders were used. For example;

- For the unmodified PG64-22 asphalt binder, the 100/99 G2 mix and the 100/96 G1 mix achieved almost the identical APA rutting, 4.74 mm and 4.76 mm, respectively. Meanwhile, the 100/99 G3 mix resulted in a higher degree of APA rutting, 5.99 mm, than the 100/96 G1 mix.
- For the polymer-modified PG76-22 asphalt binder, the 100/96 G1 mix achieved lower APA rutting levels than the 100/100 R2 mix, 2.26 mm and 3.71 mm, respectively.



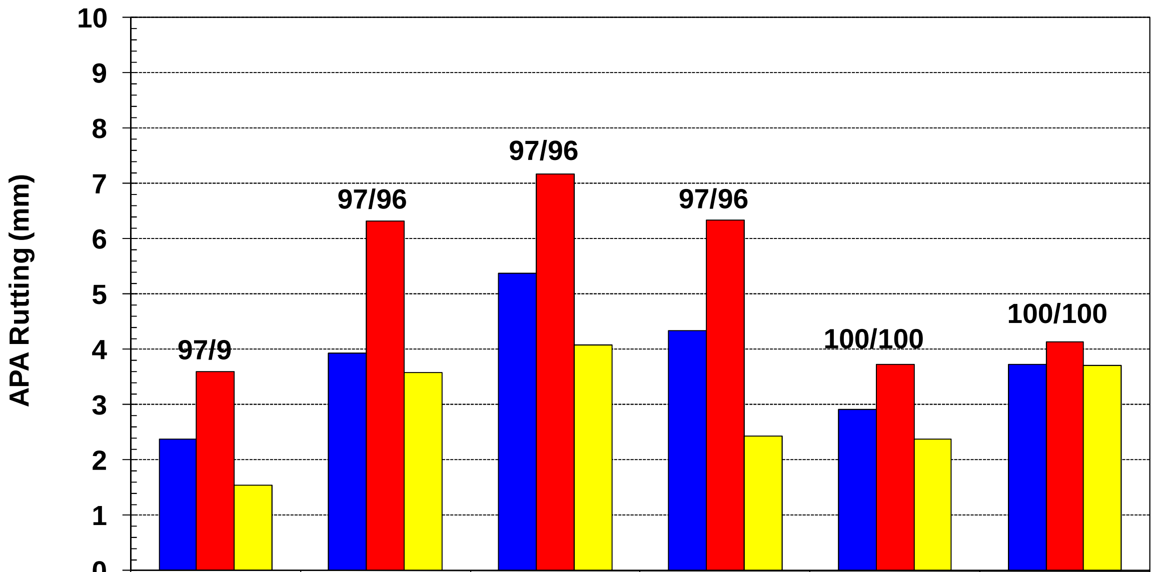
	G1	G2	G3	G4	R1	R2
64-ER	2.40	3.31	4.11	2.84	2.91	3.73
64-22	4.14	4.74	5.99	4.76	3.72	4.14
76-22	2.08	2.52	3.73	2.26	2.37	3.71

Figure 6.3 – Asphalt Pavement Analyzer Results of Crushed Gravel and Crushed Stone Sources

It is very clear that the asphalt binder stiffness has an impact on the rutting behavior as well. The addition of the polymer modified binders, PG64-22 with the elastic recovery specification (64-22ER) and the PG76-22, clearly improved the APA rutting performance, although not equally. It clearly appeared from Figure 6.3 that the gravel mixes benefited from the polymer modified binders more than the crushed stone sources.

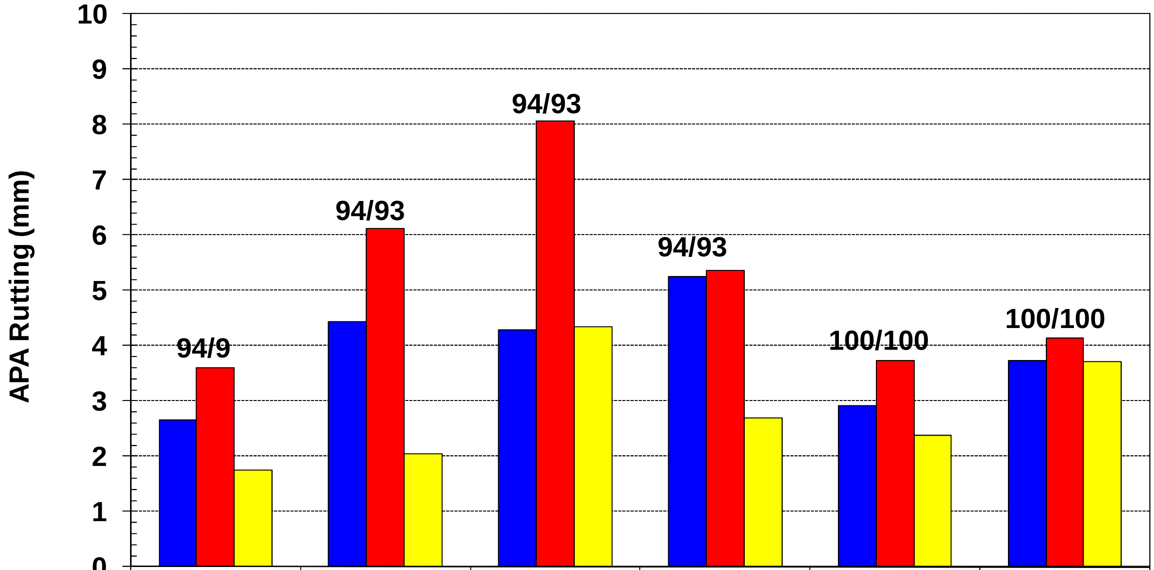
By “bumping” the asphalt binder grade from the unmodified PG64-22 to the modified PG64-22 ER, the APA rutting reduced for the gravel mixes and crushed stone mixes by 35% and 15%, respectively. By “bumping” the asphalt binder grade from the PG64-22 to the polymer modified PG76-22, the APA rutting reduced 46% and 23% for the gravel mixes and crushed stone mixes, respectively.

The Asphalt Pavement Analyzer results for the blended gravel mixes, along with their associated Fractured Face Counts (ASTM D5821) are shown in Figure 6.4 through 6.7. The test results again show inconsistencies with the general assumption that as the Fractured Face Count decreases, so does the resistance to permanent deformation. In a number of cases, the average APA rutting depth of the gravel mixes were equivalent to that of the crushed stone sources (CAA = 100/100).



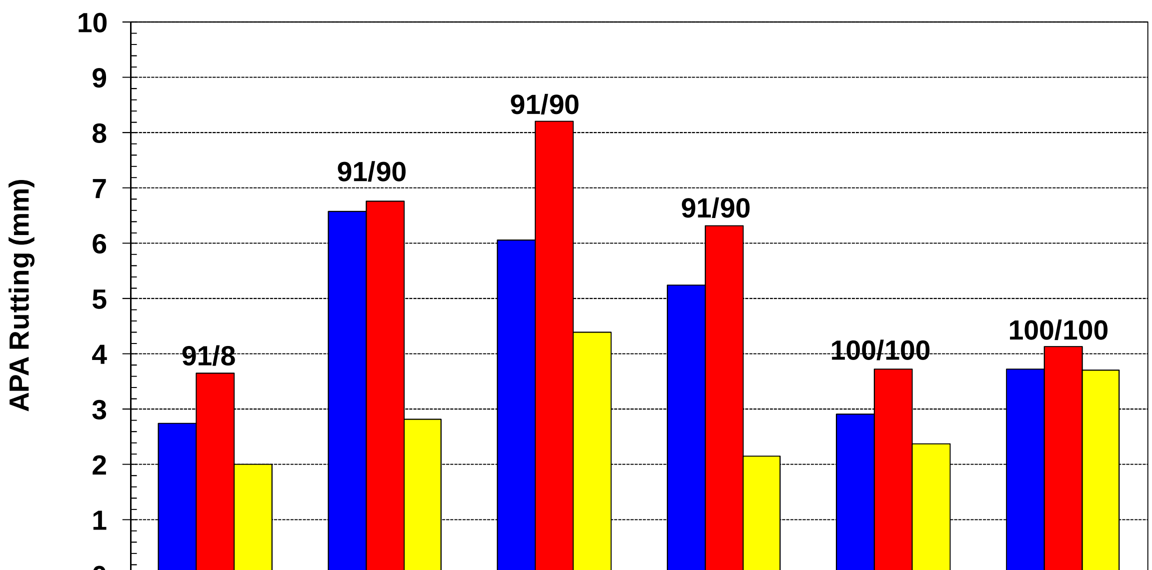
	G1	G2	G3	G4	R1	R2
64-ER	2.38	3.93	5.38	4.35	2.91	3.73
64-22	3.60	6.32	7.17	6.34	3.72	4.14
76-22	1.55	3.58	4.08	2.44	2.37	3.71

Figure 6.4 – Asphalt Pavement Analyzer Results of 97% One Face Fractured Count



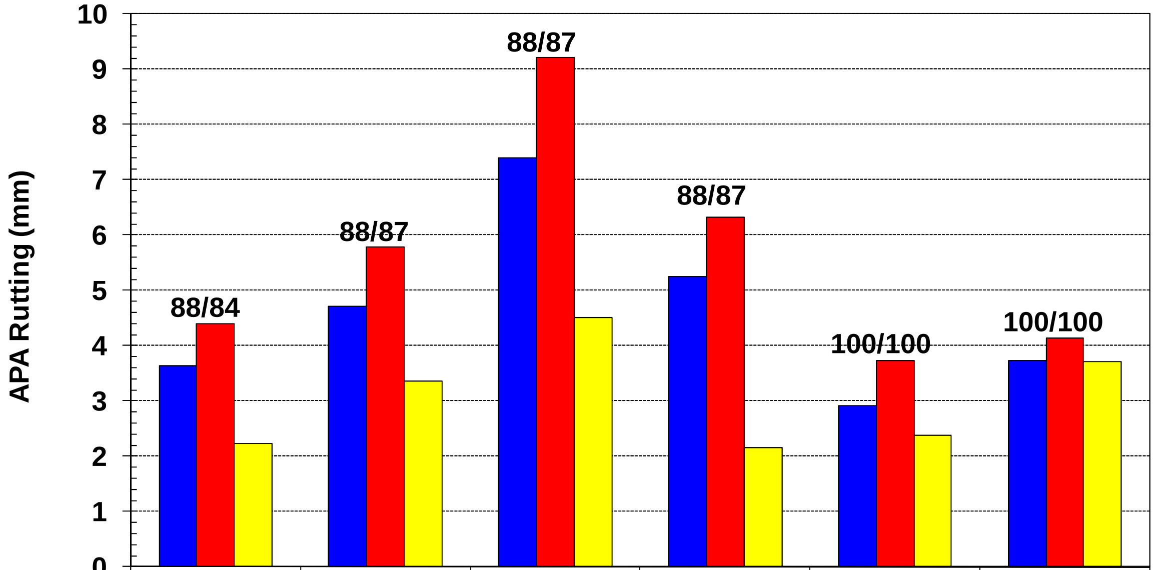
	G1	G2	G3	G4	R1	R2
64-ER	2.65	4.44	4.29	5.25	2.91	3.73
64-22	3.60	6.12	8.07	5.37	3.72	4.14
76-22	1.75	2.05	4.35	2.69	2.37	3.71

Figure 6.5 – Asphalt Pavement Analyzer Results of 94% One Face Fractured Count



	G1	G2	G3	G4	R1	R2
64-ER	2.74	6.58	6.07	5.25	2.91	3.73
64-22	3.65	6.77	8.21	6.32	3.72	4.14
76-22	2.00	2.83	4.39	2.15	2.37	3.71

Figure 6.6 – Asphalt Pavement Analyzer Results for 91% One Face Fractured Count



	G1	G2	G3	G4	R1	R2
64-ER	3.64	4.71	7.39	5.25	2.91	3.73
64-22	4.40	5.78	9.21	6.32	3.72	4.14
76-22	2.23	3.37	4.50	2.15	2.37	3.71

Figure 6.7 – Asphalt Pavement Analyzer Results for 88% One Face Fractured Count

The preliminary Asphalt Pavement Analyzer rutting results did not appear to correlate to the Coarse Aggregate Angularity (CAA) measurements as determined by ASTM D5821. The rutting results showed that the gravel mix, having a CAA of 100/96, had identical APA rutting results as a 100/99 gravel mix, while achieving comparable APA rutting results to a 100/100 crushed stone source. Meanwhile, increasing the asphalt binder stiffness properties by "grade bumping" clearly impacted the APA rutting performance for all mixes, although greater improvements were found in the gravel mixes. In fact, in reviewing Figure 6.7, one can see that by changing the asphalt binder grade to a PG76-22 asphalt binder in the 88/87 and 88/84 gravel mixes, the APA rutting properties were similar to that of the two crushed stone mixes (100/100) when the crushed stone mixes used an unmodified PG64-22 asphalt binder.

One and Two Face Fractured Counts from the pooled dataset were compared with the Asphalt Pavement Analyzer rutting to determine if Fractured Face Counts correlated to general permanent deformation resistance. Results for the PG64-22 asphalt binder are shown in Figures 6.8 and 6.9. The results clearly show that a poor correlation exists between the coarse aggregate Fractured Face Count and the measured APA rutting. Although the general trend is what one would expect (i.e. – decrease in APA rutting as the CAA increases), the correlation between rutting and the CAA is poor. The correlations did not improve, and in most cases were worse, when comparing the PG64-22(ER) and PG76-22 asphalt binders. However, the results are not shown for brevity.

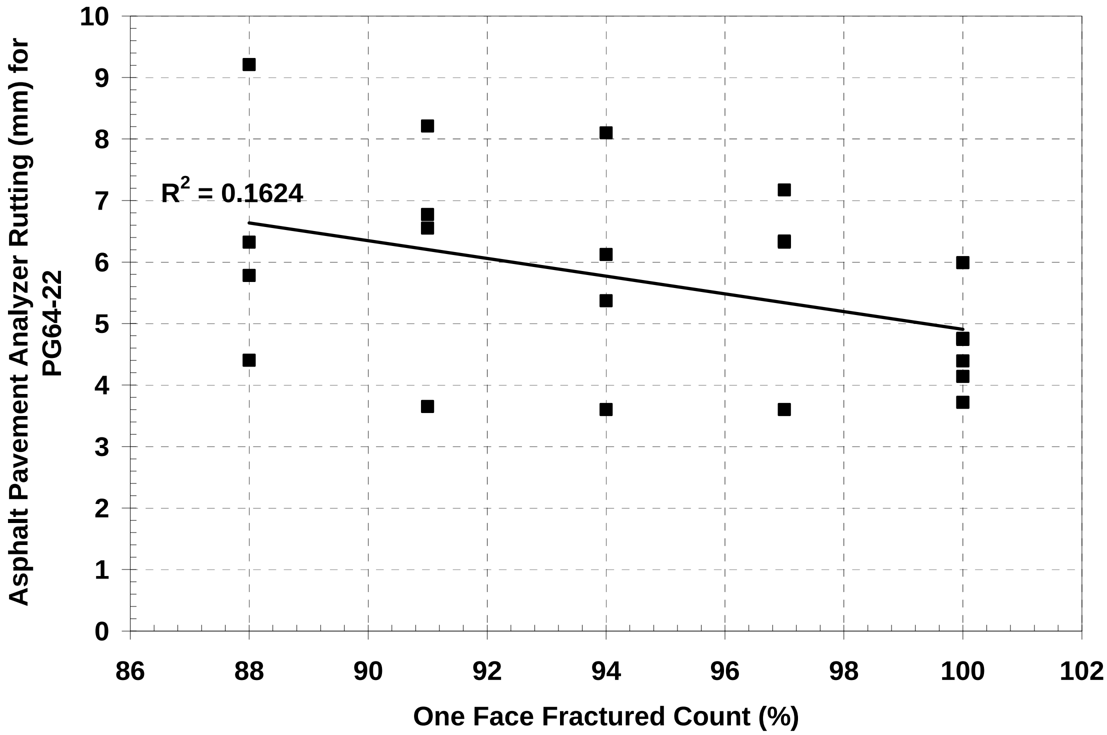


Figure 6.8 – One Face Fractured Count vs Asphalt Pavement Analyzer Rutting for the PG64-22 Asphalt Binder

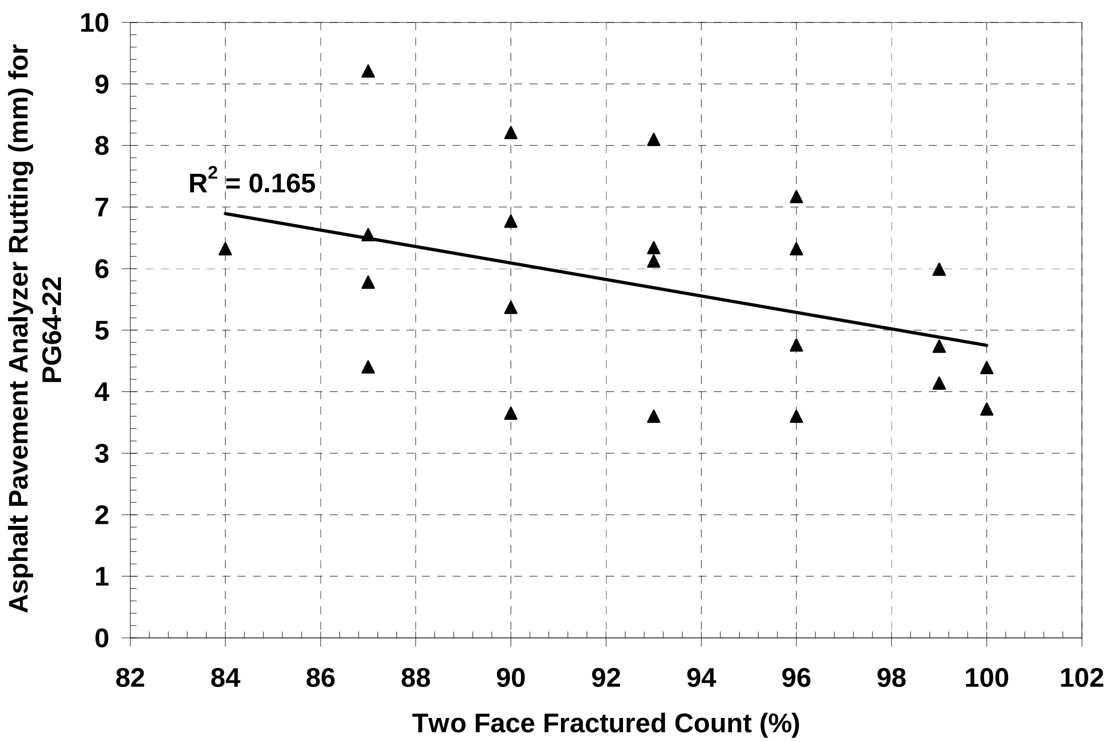


Figure 6.9 – Two Face Fractured Count vs Asphalt Pavement Analyzer Rutting for the PG64-22 Asphalt Binder

6.1.2 – Aggregate Angularity Measurements vs APA Rutting for Baseline Aggregate Sources

The baseline aggregate sources (mixtures) were compared to the aggregate angularity tests previously found to reasonable rank and measure the texture and angularity properties of the aggregates (AIMS device and Uncompacted Voids Content). Figure 6.10 shows the comparison between the Asphalt Pavement Analyzer rutting and the measured Uncompacted Voids Content of the different aggregate sources. The figure clearly indicates that poor correlations are found between the coarse aggregate angularity and the APA results for all three asphalt binders. Although the APA is capable of ranking the results of different asphalt binders, it appears the APA is not sensitive enough to differentiate this narrow range of Uncompacted Voids Content.

The baseline aggregates were also compared to the AIMS Angularity and Texture Index to the Asphalt Pavement Analyzer (APA) rutting. These results are shown in Figures 6.11 and 6.12. Once again, the correlations indicate that the APA is more sensitive the asphalt binder stiffness as opposed to the range of AIMS Angularity and Texture found in the baseline aggregate sources.

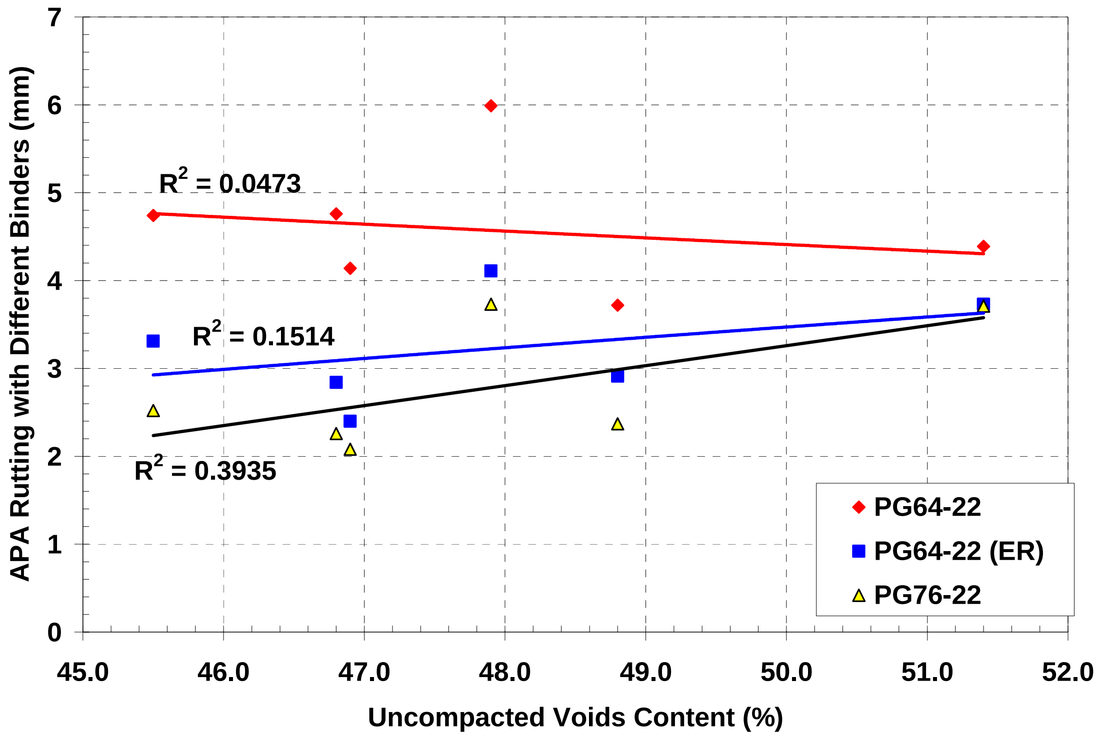


Figure 6.10 – APA Rutting vs Uncompacted Voids Content for All Three Binders

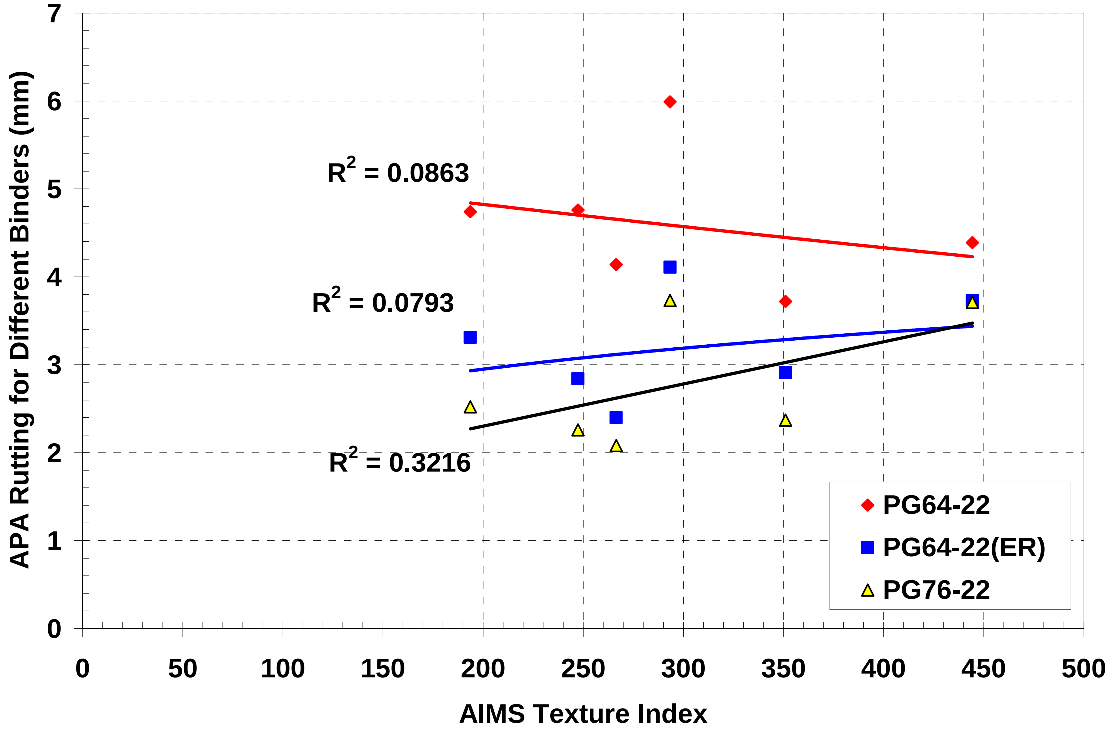


Figure 6.11 – APA Rutting vs AIMS Texture Index for All Three Binders

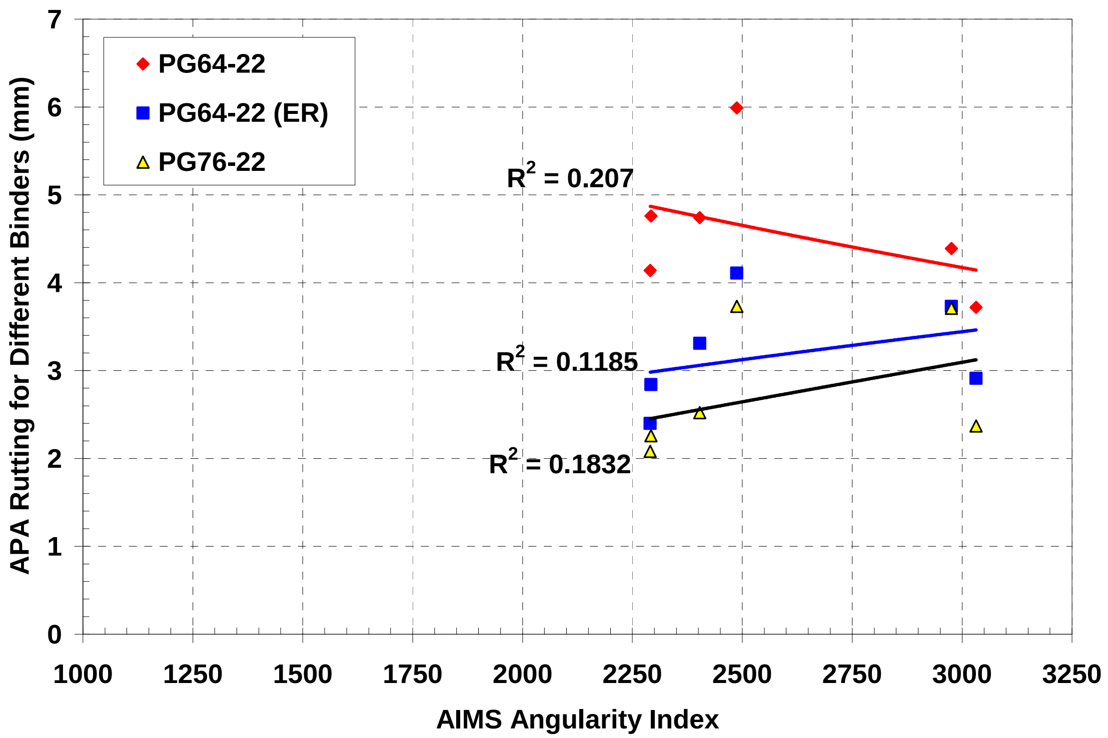


Figure 6.12 – APA Rutting vs AIMS Angularity Index for All Three Binders

6.1.3 – Asphalt Pavement Analyzer vs Angularity Measurements for Gravel Blends

The Asphalt Pavement Analyzer (APA) was again used to measure the rutting performance of the different gravel mixes when uncrushed gravels were blended in the specified percentages shown earlier in Table 4.5. Each of the blended gravel blends were also evaluated using the three different asphalt binders; PG64-22, PG64-22(ER), and PG76-22. Figure 6.13 through 6.15 shows the APA rutting results for the three different asphalt binders evaluated in the study. In each graph, although the test data was pooled to determine the regression correlation, the individual aggregate sources are shown for further discussion. Again, similar to the baseline data, a poor correlation exists between the APA test data and the Uncompacted Voids Content. Again, from the test data shown in Figures 6.13 through 6.15, the Asphalt Pavement Analyzer may not be sensitive enough to differentiate between the narrow changes in coarse aggregate angularity of the aggregate blends used.

The Asphalt Pavement Analyzer (APA) was also compared to the AIMS Texture and Angularity Indexes for the different gravel blends and three different asphalt binders. The test results for the PG64-22 asphalt binder are shown in Figures 6.16 and 6.17. The PG64-22(ER) and the PG76-22 asphalt binders are not shown because the correlations and trends were the same or worse than the PG64-22 data. The test results shown in Figure 6.16 and 6.17, as also indicated in the APA figures, indicate a poor relationship between the AIMS Indexes and Asphalt Pavement Analyzer rutting results.

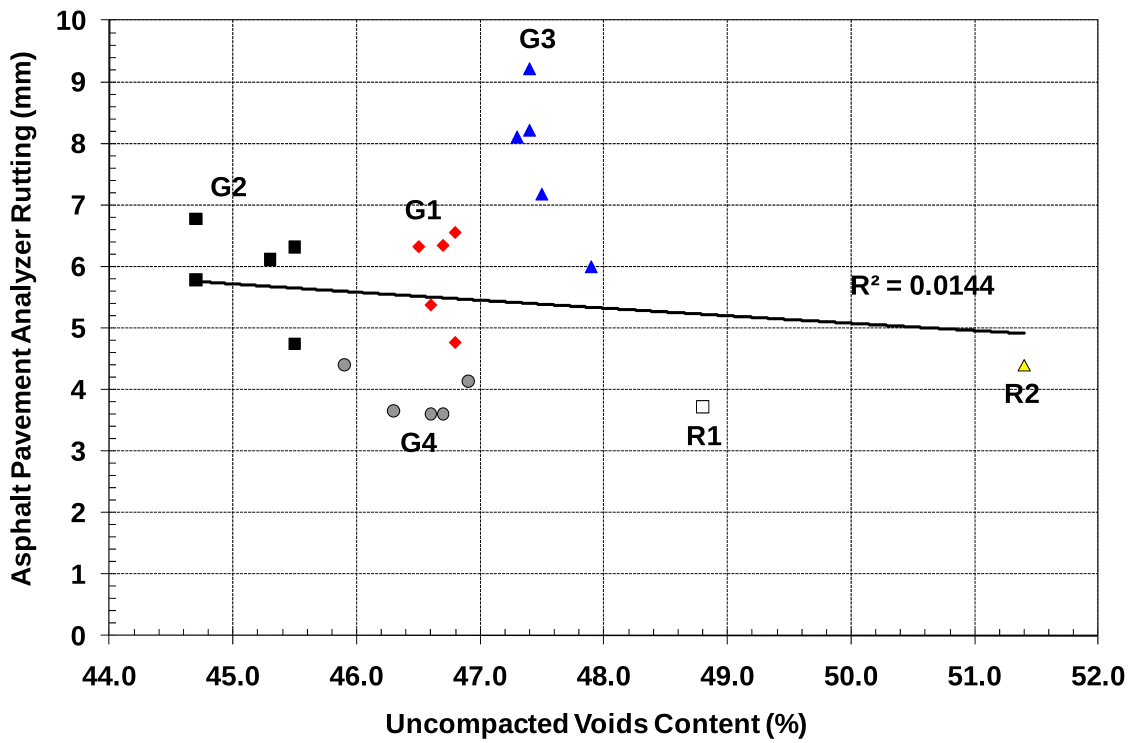


Figure 6.13 – Asphalt Pavement Analyzer Rutting for All Mixtures vs Uncompacted Voids Content – PG64-22 Asphalt Binder

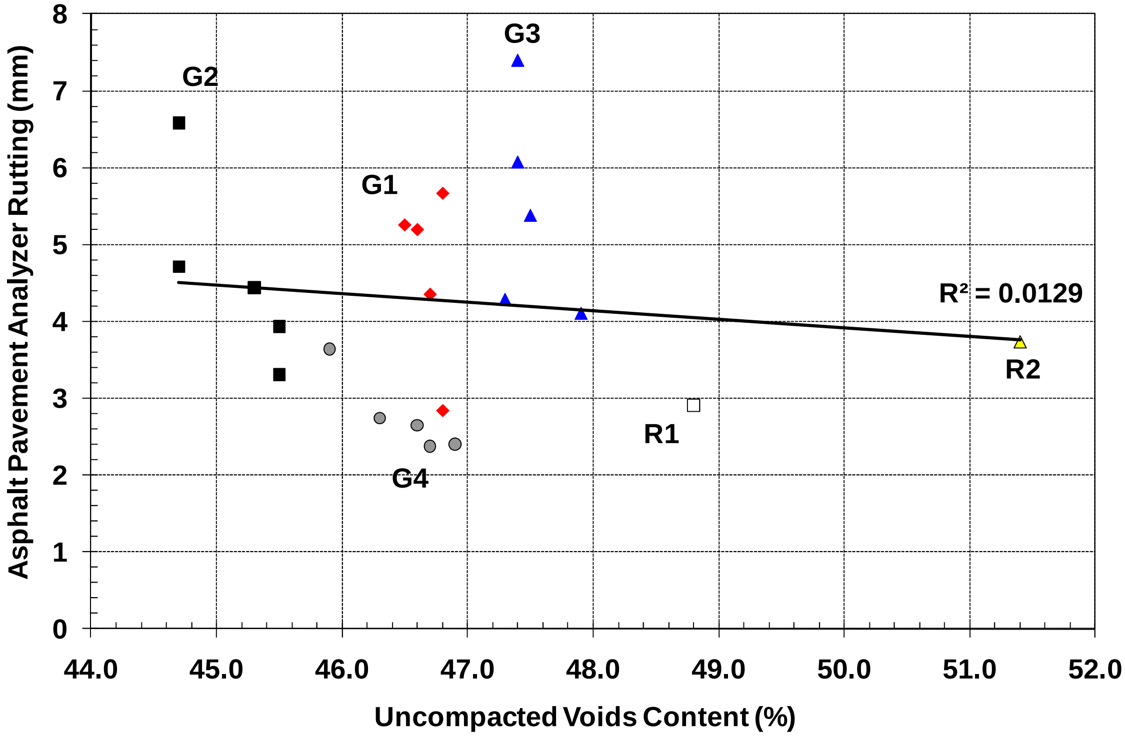


Figure 6.14 – Asphalt Pavement Analyzer Rutting for All Mixtures vs Uncompacted Voids Content – PG64-22 (ER)

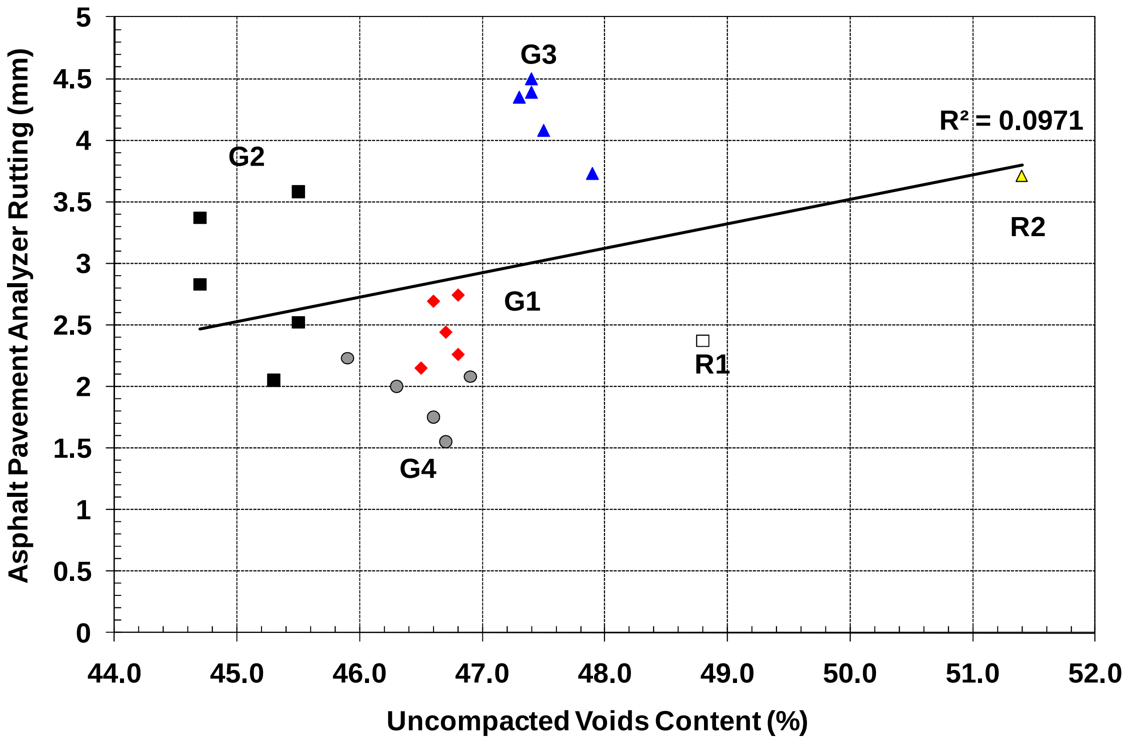


Figure 6.15 – Asphalt Pavement Analyzer Rutting for All Mixtures vs Uncompacted Voids Content – PG76-22 Asphalt Binder

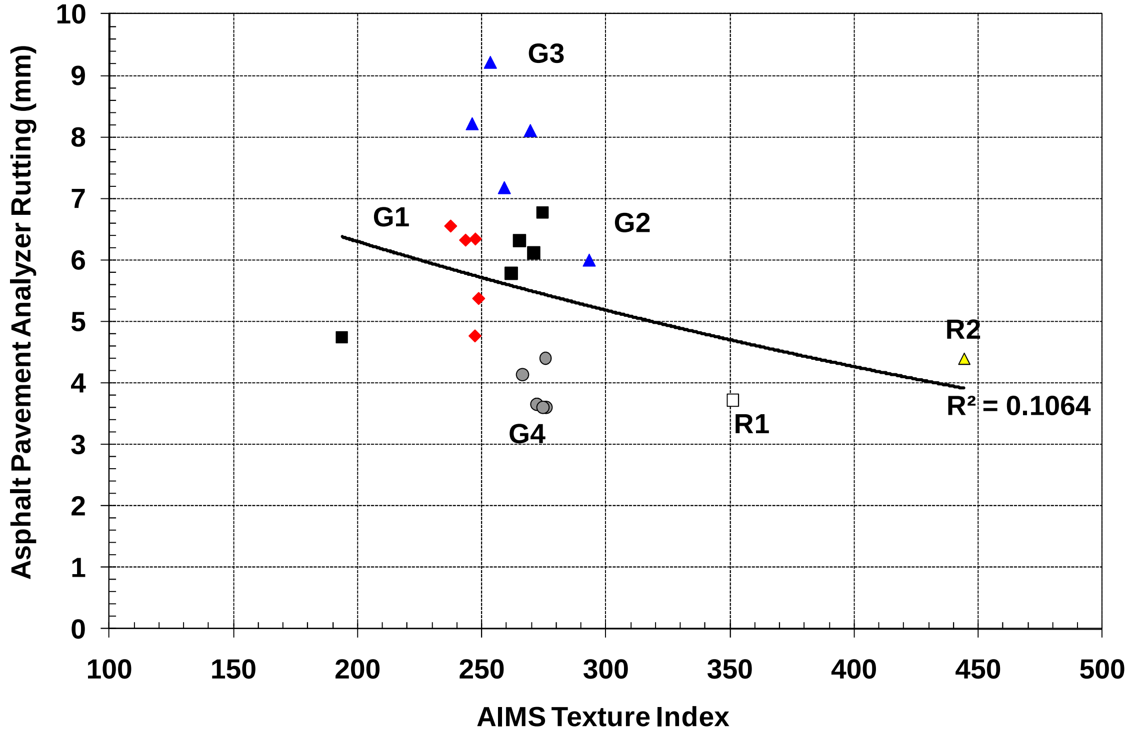


Figure 6.16 – Asphalt Pavement Analyzer Rutting for All Mixtures vs AIMS Texture Index – PG64-22 Asphalt Binder

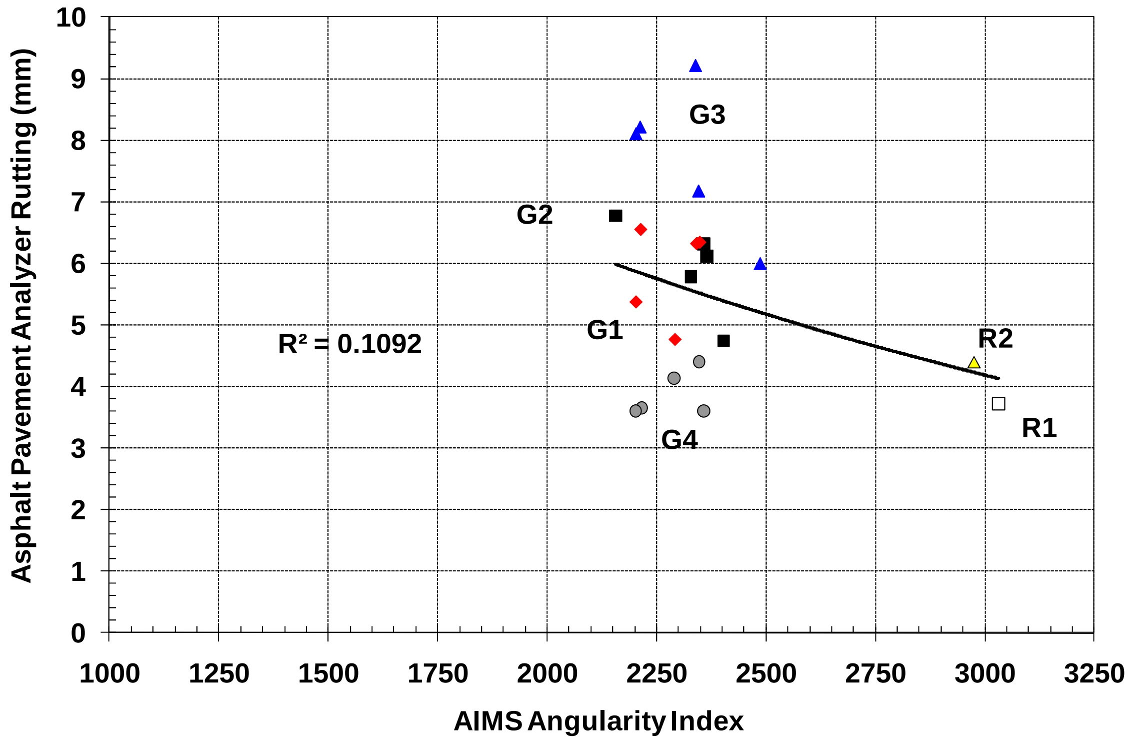


Figure 6.17 – Asphalt Pavement Analyzer Rutting for All Mixture vs AIMS Angularity Index – PG64-22 Asphalt Binder

6.1.4 – Summary of Asphalt Pavement Analyzer Results

In total, 396 mixture samples were tested using the Asphalt Pavement Analyzer to determine the mixture performance relative to varying the coarse aggregate angularity properties. Based on the average results previously shown, the following conclusions can be drawn:

- The rutting measured in the Asphalt Pavement Analyzer appeared to be sensitive to type of asphalt binder used in the study. As shown throughout the figures, the APA was always able to differentiate between the asphalt binders used when using the same aggregate blend and angularity.
- The rutting measured in the Asphalt Pavement Analyzer appeared to lack the sensitivity required to distinguish asphalt mixtures of different levels of aggregate angularities and textures. As shown in the figures above, neither the AIMS Texture Index, AIMS Angularity Index, and Uncompacted Voids Content correlated to the Asphalt Pavement Analyzer rutting results. The test data did indicate that when solely looking at the APA rutting of the individual mixtures (i.e. – not pooled), a slightly better correlation was able to be generated.
- One of the issues that may have caused the lack of sensitivity is the selected test temperature used in the study (i.e. – 58^{o}C). In hind sight, selecting a higher test temperature would have created a greater potential for mobilization (or permanent deformation) within the mixture. Greater levels of mobilization may have activated the internal shear strength of the asphalt mixture, which in turn, should have emphasized the aggregate blends with higher levels of angularity and texture.

6.2 – Repeated Load Permanent Deformation Testing (Flow Number) with Confining Pressure

The Asphalt Mixture Performance Tester (AMPT) was used to evaluate the permanent deformation properties of the different aggregate sources and gravel blends. The AMPT was used in the Repeated Load Permanent Deformation (RLPD), where the temperature conditioned asphalt specimen is subjected to a cyclic stress (or deviatoric stress, σ_d). A photo of the AMPT used in this study is shown in Figure 6.18.

Figure 6.18 – Asphalt Mixture Performance Tester (AMPT) Used in Study

The testing conditions used for the study were as follows:

- Test Temperature = 58^{o}C
- Applied Deviatoric Stress, σ_d = 100 psi
- Applied Confining Stress, σ_3 = 20 psi

The test temperature and stress conditions were selected based on information previously presented in Chapters 2 and 3.

During testing, the applied deviatoric stress (σ_d), specimen deformation, and test temperature are recorded at the end of each load cycle. Each load cycle consists of a 0.1 second load pulse with a 0.9 second rest period. The recorded permanent deformation vs load cycle is then applied to determine the Flow Number, which represents the point of tertiary flow or flow failure of the asphalt mixture; therefore, the higher the Flow Number, the greater resistance to permanent deformation. For this study, the Flow Number was determined using the Francken model, as described by Dongré et al. (2009). This new method of determining the Flow Number is not as sensitive to machine noise as what was previously used.

Although not commonly used, confining pressure was also applied to the test specimens during loading. The main purpose of the confining pressure is to provide greater influence of the internal friction properties of the asphalt mixture, which is predominantly controlled by the angularity and texture of the aggregates, on the permanent deformation results. This phenomena is most often represented using the Mohr-Columb Failure Envelope theory (Figure 6.19). In Figure 6.19, the shear strength of the material is dependent on the cohesion (C) and internal friction properties (ϕ) of the specimen. If confining pressure (σ_3) is not applied to the specimen during loading ($\sigma_1 = \sigma_d + \sigma_3$), then the shear strength of the asphalt mixture is purely dependent on the cohesive (C) properties of the asphalt mixture as the shear strength envelope becomes flat. With the cohesive properties being dominated by the asphalt binder properties, test results without confining pressure are more influenced by the asphalt binder high temperature stiffness, as opposed to both the combined effect of the asphalt binder stiffness and aggregate angularity and texture properties.

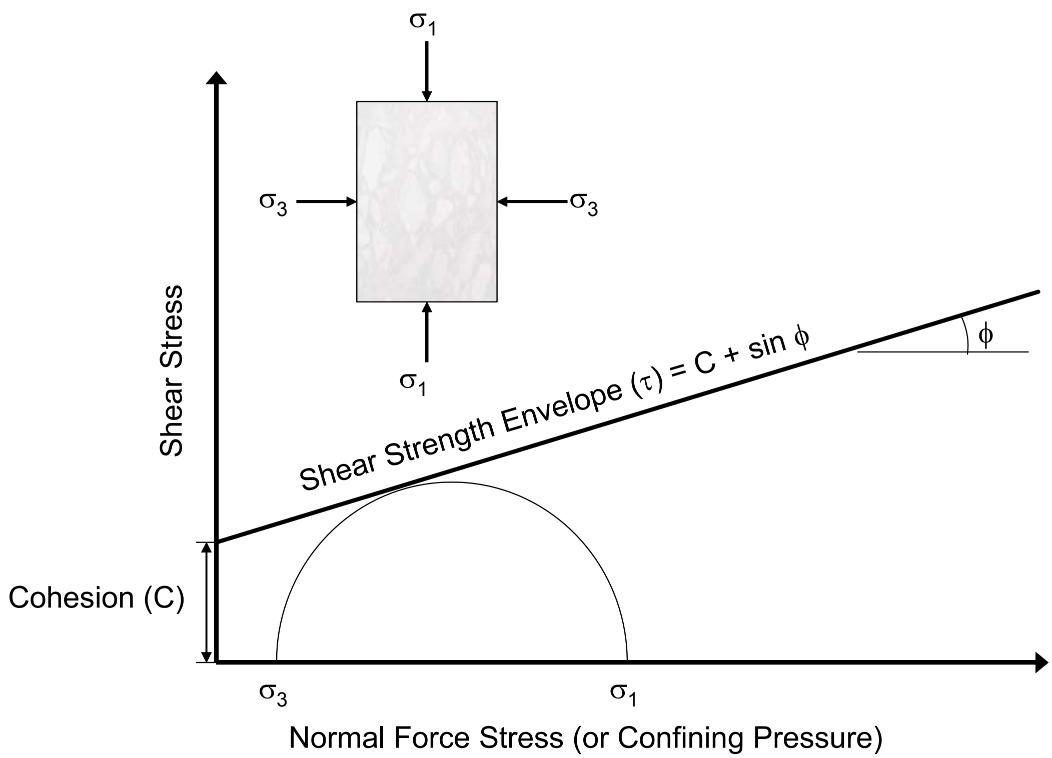
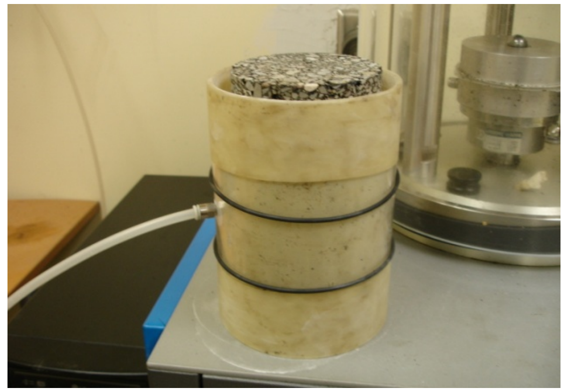


Figure 6.19 – Schematic of Mohr Columb Failure Envelope

To apply the confining pressure, a latex membrane is placed in an expander (Figure 6.20a), placed over top of the specimen (Figure 6.20b), and then released over the sample. Air pressure is then applied within the AMPT chamber to achieve the desired confining pressure (Figure 6.20c).

(a)



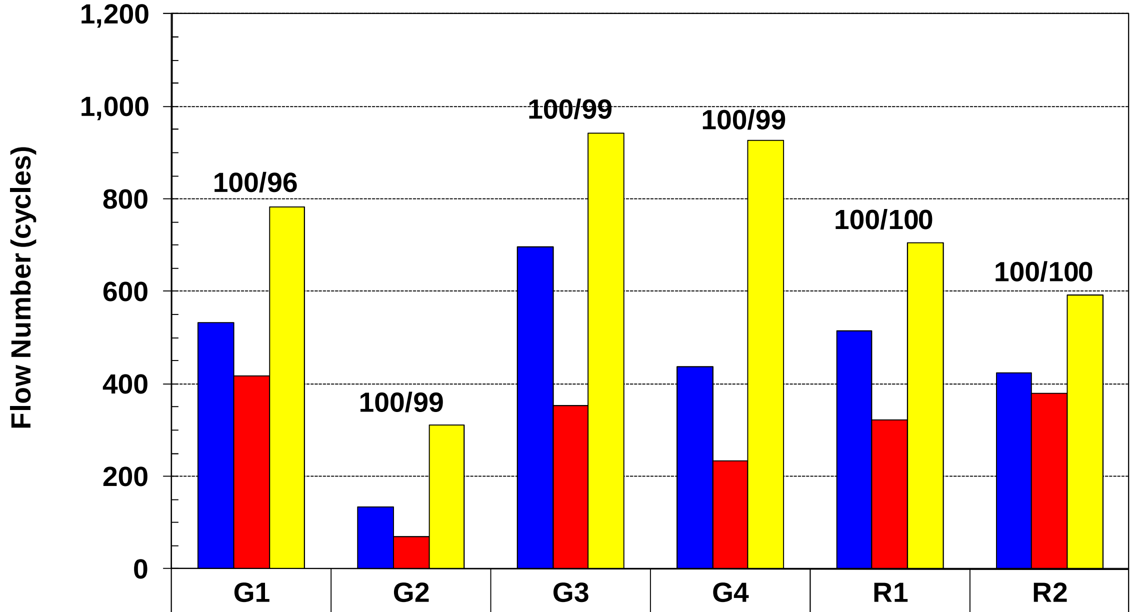
(b)

(c)

Figure 6.20 – (a) Membrane Expander, (b) Placing Expander Over Specimen, (c) Applying Confining Pressure to Specimen

6.2.1 – Flow Number Results for Baseline Aggregates

The average results for the Flow Number testing are shown in Figure 6.21. The test results indicate that the gravel mixes, using crushed gravel (Single Face Crushed Count = 100%) performed as well and better than the crushed stone mixes (i.e. – R1 and R2). This is again good evidence indicating that crushed gravel at CAA levels of 100/99 to 100/96 perform as well as 100/100 crushed stone sources with respect to



	G1	G2	G3	G4	R1	R2
64-ER	532	134	697	437	514	425
64-22	417	70	354	234	323	381
76-22	782	311	943	926	705	593

Figure 6.21 – Average Flow Number Results for Baseline Aggregates

permanent deformation. However, it should be noted that one of the 100/99 crushed gravel mixes did not perform as well as the two other 100/99, or even as well as the 100/96 CAA mix. This is consistent with the test results shown earlier in the Asphalt Pavement Analyzer rutting results. These results again show that the relative ranking of CAA, as measured using ASTM D5821, does not provide a good indication of rutting performance of the asphalt mixture.

Once again, the asphalt binder grade had a significant impact on the permanent deformation resistance, as measured by the Flow Number in the repeated load test. Significant increases in the measured Flow Number occurred when “bumping” the asphalt binder grade, especially with the gravel mixes. When “bumping” the asphalt binder grade from the unmodified PG64-22 to the polymer modified PG64-22(ER), a 76% and 36% improvement in the Flow Number results were measured for the gravel mixes and crushed stone mixes, respectively. When “bumping” the asphalt binder grade from the unmodified PG64-22 to the polymer modified PG76-22, a 223% and 87% improvement in the Flow Number results were measured for the gravel mixes and crushed stone mixes, respectively.

Correlation regression analysis was conducted for the Flow Number results of the gravel blend and crushed stone mixes, comparing them to the Fractured Face Count values. The regression analysis is shown in Figures 6.22 to 6.27. The figures indicate that a relatively poor correlation exists between the CAA, as determined by ASTM D5821, and the Flow Number results for all three asphalt binders. The correlation continues to get poorer as

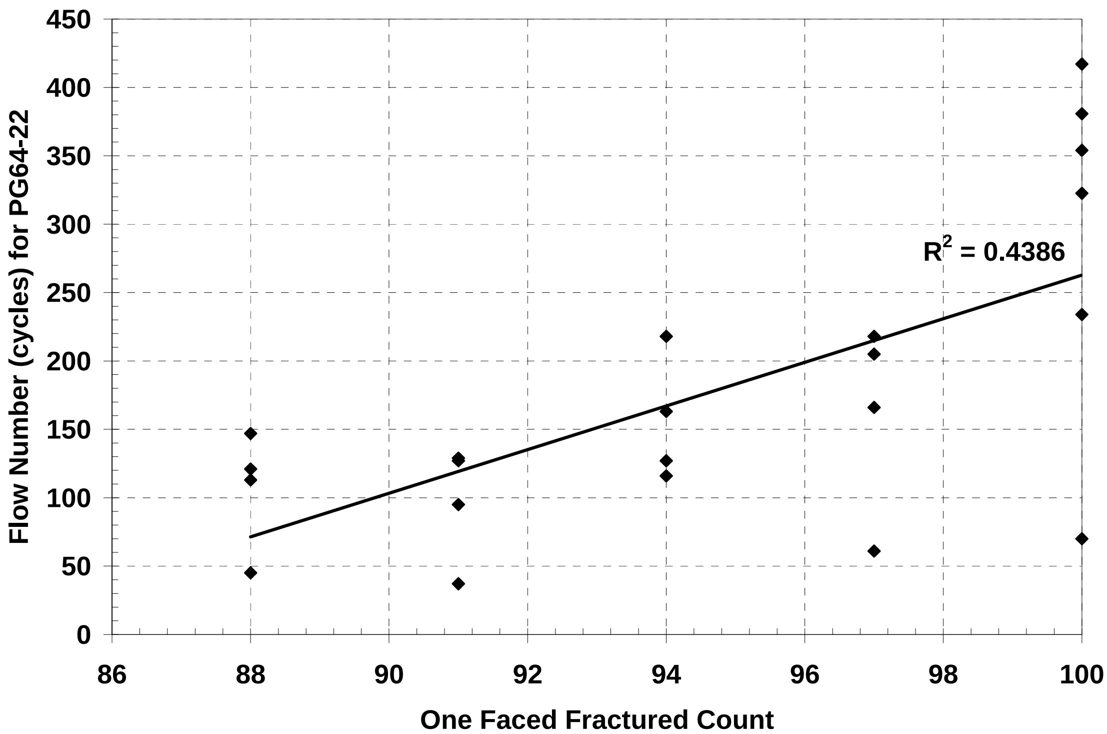


Figure 6.22 – Flow Number Results vs One Face Fractured Count for PG64-22 Asphalt Binder

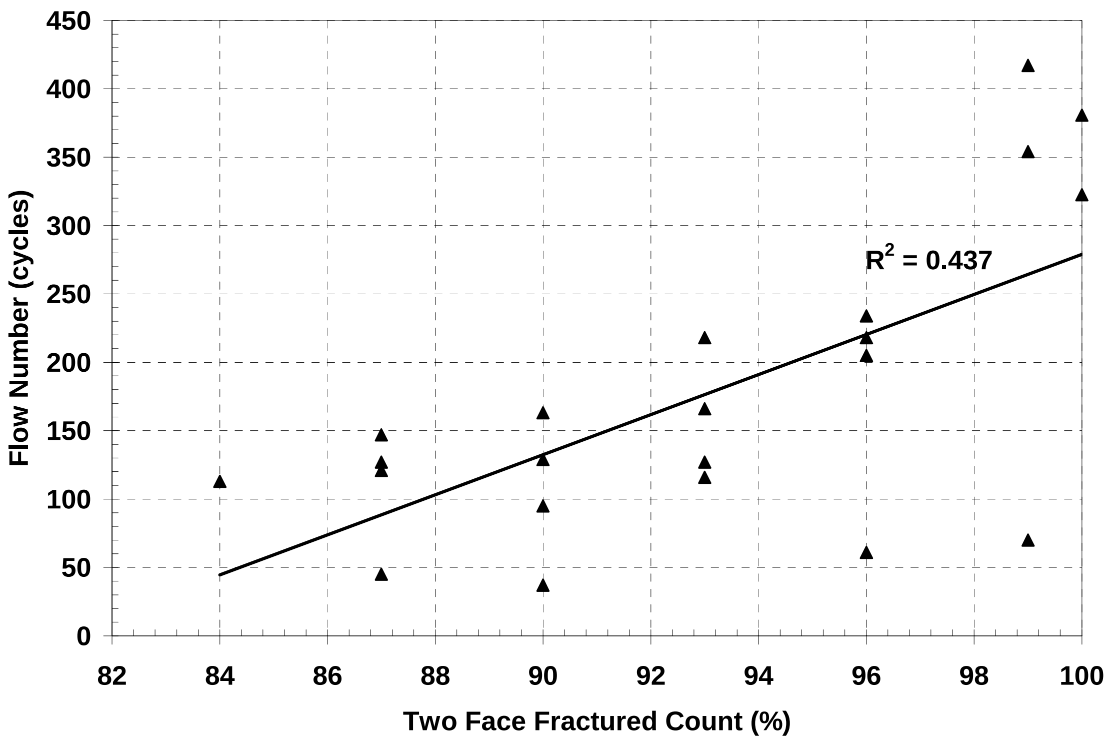


Figure 6.23 – Flow Number Results vs Two Face Fractured Count for PG64-22 Asphalt Binder

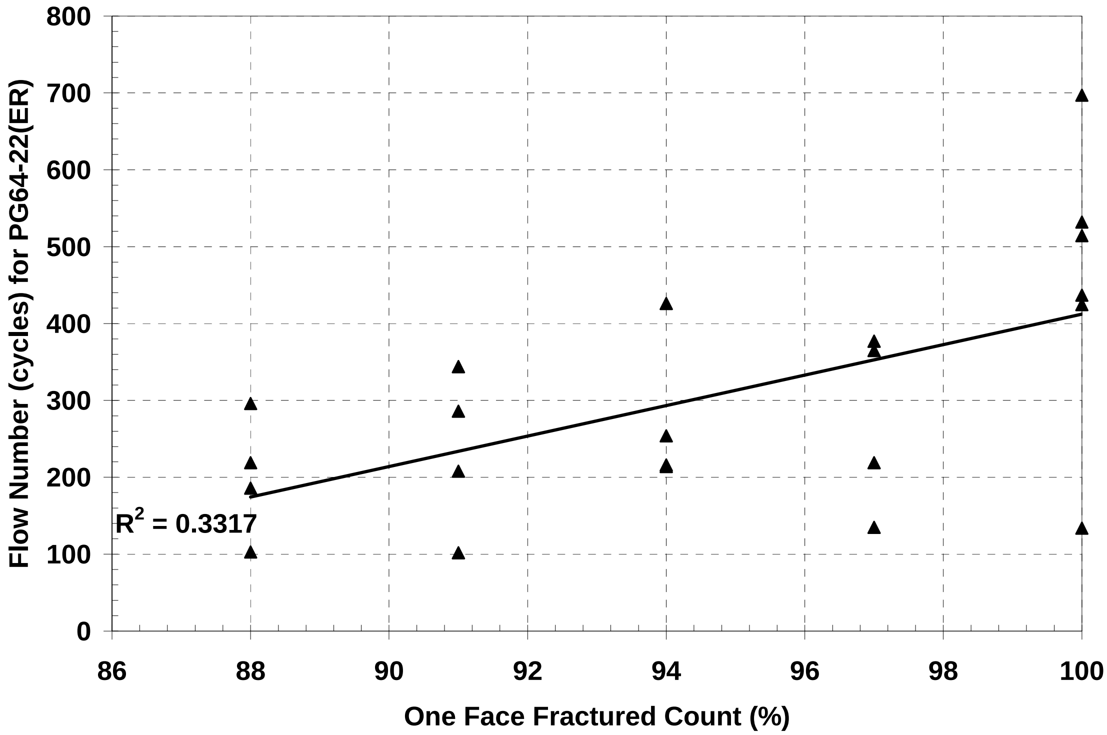


Figure 6.24 – Flow Number Results vs One Face Fractured Count for PG64-22(ER) Asphalt Binder

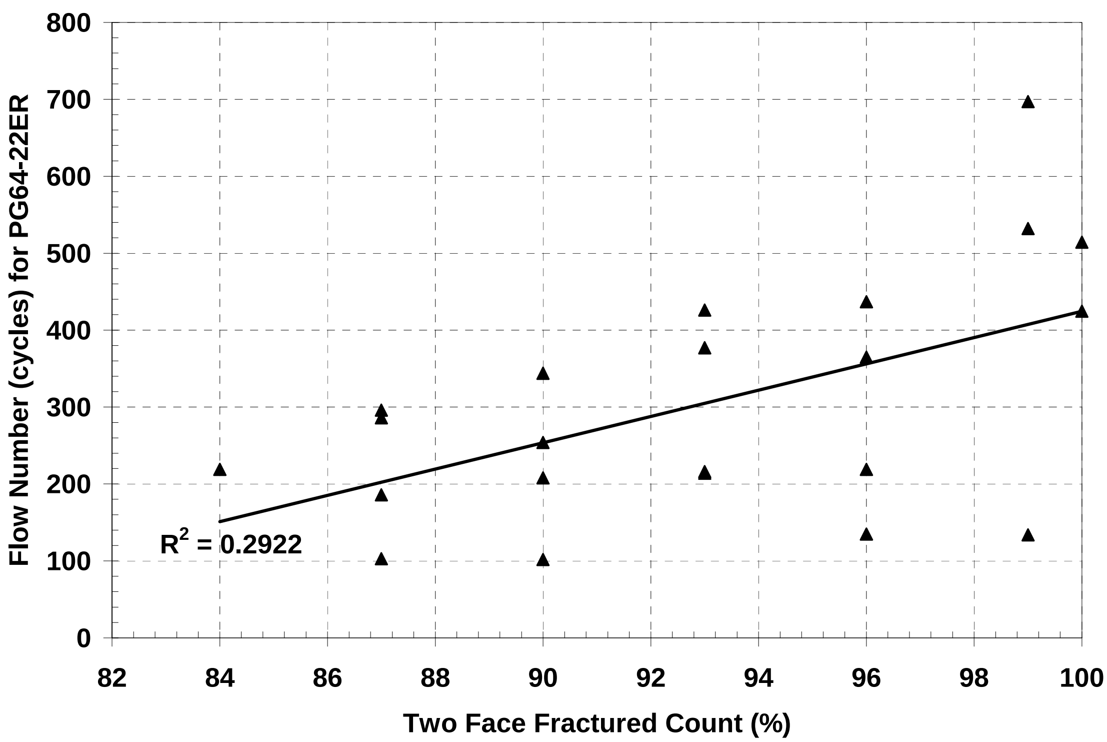


Figure 6.25 – Flow Number Results vs Two Face Fractured Count for PG64-22(ER)

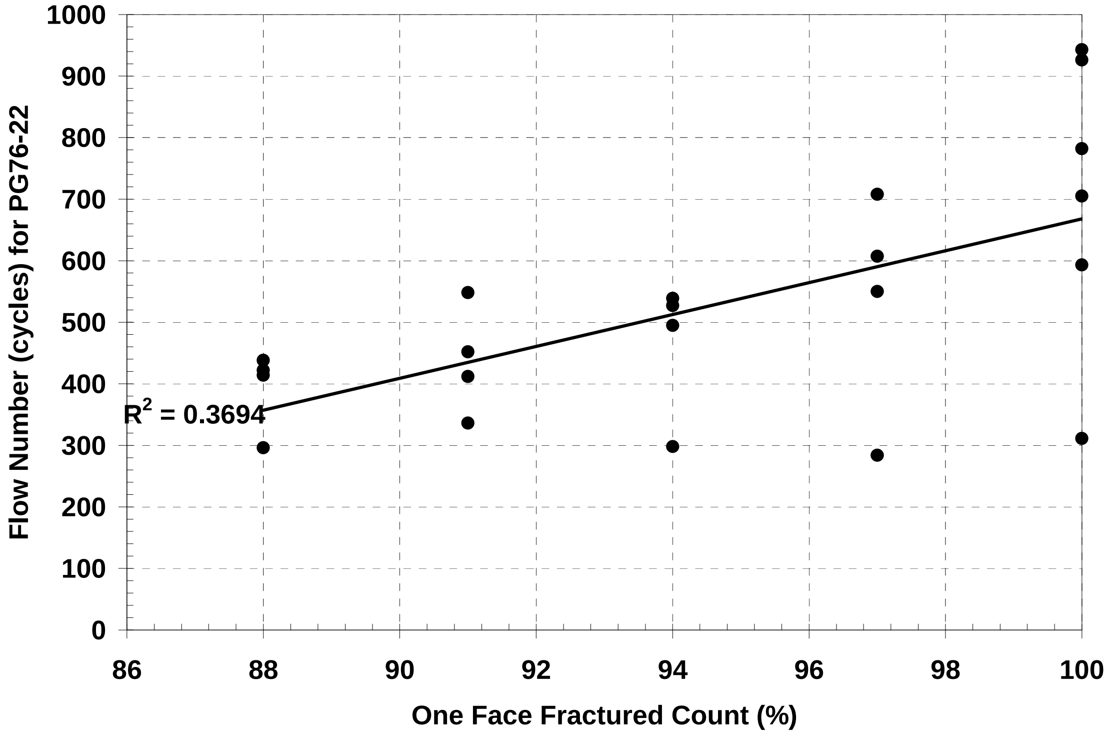


Figure 6.26 – Flow Number Results vs One Face Fractured Count for PG76-22 Asphalt Binder

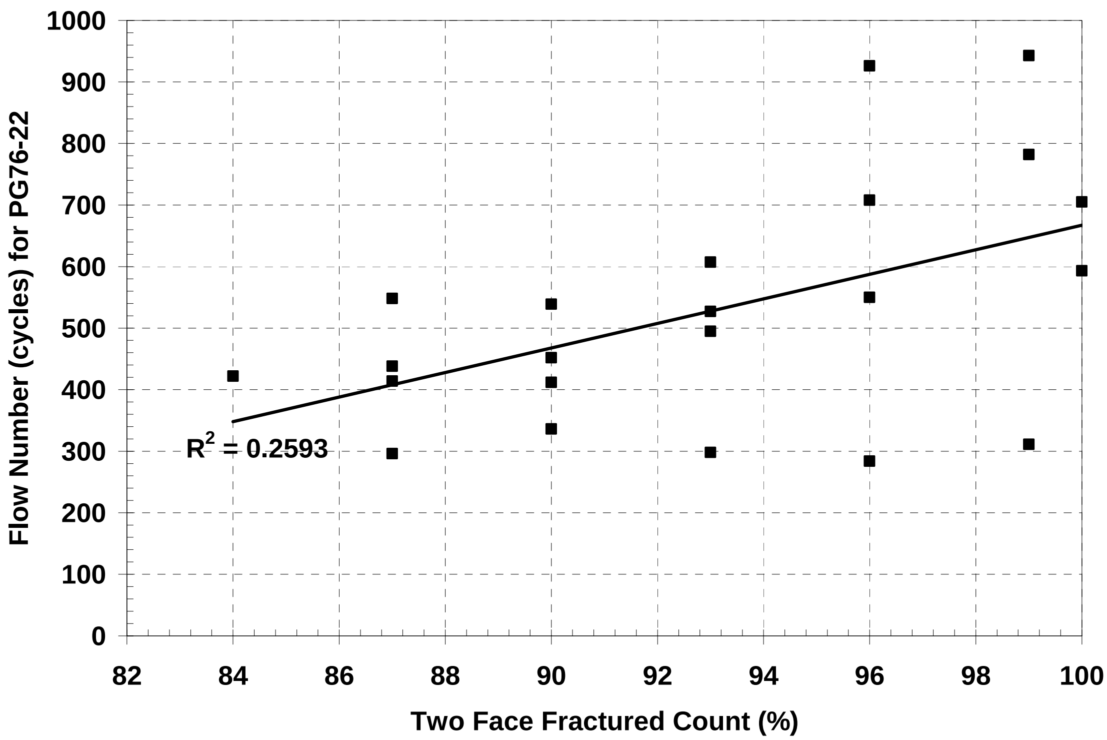


Figure 6.27 – Flow Number Results vs Two Face Fractured Count for PG76-22 Asphalt Binder

the asphalt binder high temperature stiffness increases from the unmodified PG64-22, to the PG64-22 (ER) and the PG76-22 asphalt binders. This is another indication that as the high temperature asphalt binder stiffness increases, the influence of the aggregate angularity and texture properties on the permanent deformation properties decreases.

6.2.2 – Flow Number vs Uncompacted Voids Content

The different gravel sources were blended with uncrushed gravel to achieve varying levels of Coarse Aggregate Angularity (CAA) as measured with ASTM D5821. The Uncompacted Voids Content, determined using AASHTO T326, was determined for each of the blended gravels, as well as the baseline gravel and crushed stones aggregates. The Flow Number was measured for each of the resultant mixtures and then compared to their Uncompacted Voids Content properties. The results, for each of the asphalt binder grades used, of this testing is shown in Figures 6.28 through 6.30. The Uncompacted Voids Content shows a much better correlation to the Flow Number than the Asphalt Pavement Analyzer. This is most likely due to the addition of the applied confining pressure increasing the influence of the aggregate properties on the permanent deformation resistance.

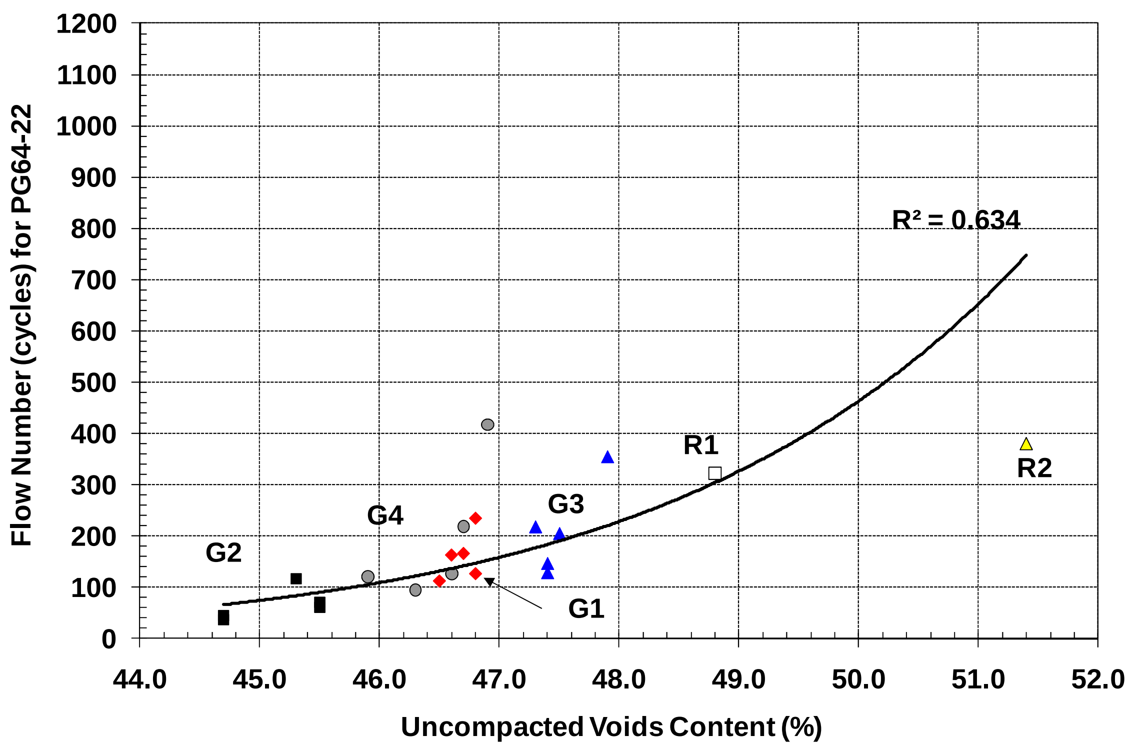


Figure 6.28 – Flow Number vs Uncompacted Voids Content for the PG64-22 Asphalt Binder

What is interesting in Figures 6.28 through 6.30 is as the asphalt binder stiffness increases (i.e. – going from the PG64-22 to PG64-22(ER) to PG76-22), the correlation between the Uncompacted Voids Content and Flow Number decreases. This is most likely due to the influence of the stiffer asphalt over-powering the aggregate texture and angularity properties. Using the Mohr-Columb Failure Envelope as an example, increasing the asphalt binder stiffness basically increases the cohesion within the asphalt mixture, thereby, increasing the overall shear strength of the mixture. However, as shown in the results, the increase is not exactly equal for each mixture, as the test results indicated the gravel mixes had greater levels of improvement than the crushed stone mixes.

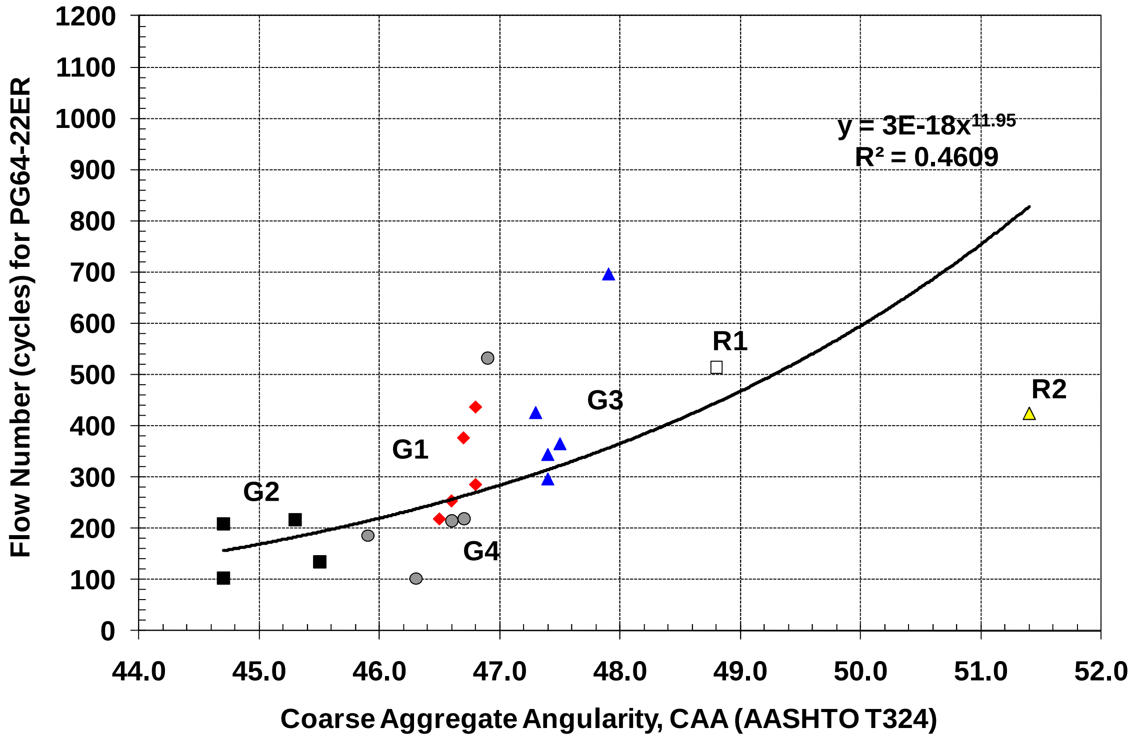


Figure 6.29 – Flow Number vs Uncompacted Voids Content for the PG64-22(ER) Asphalt Binder

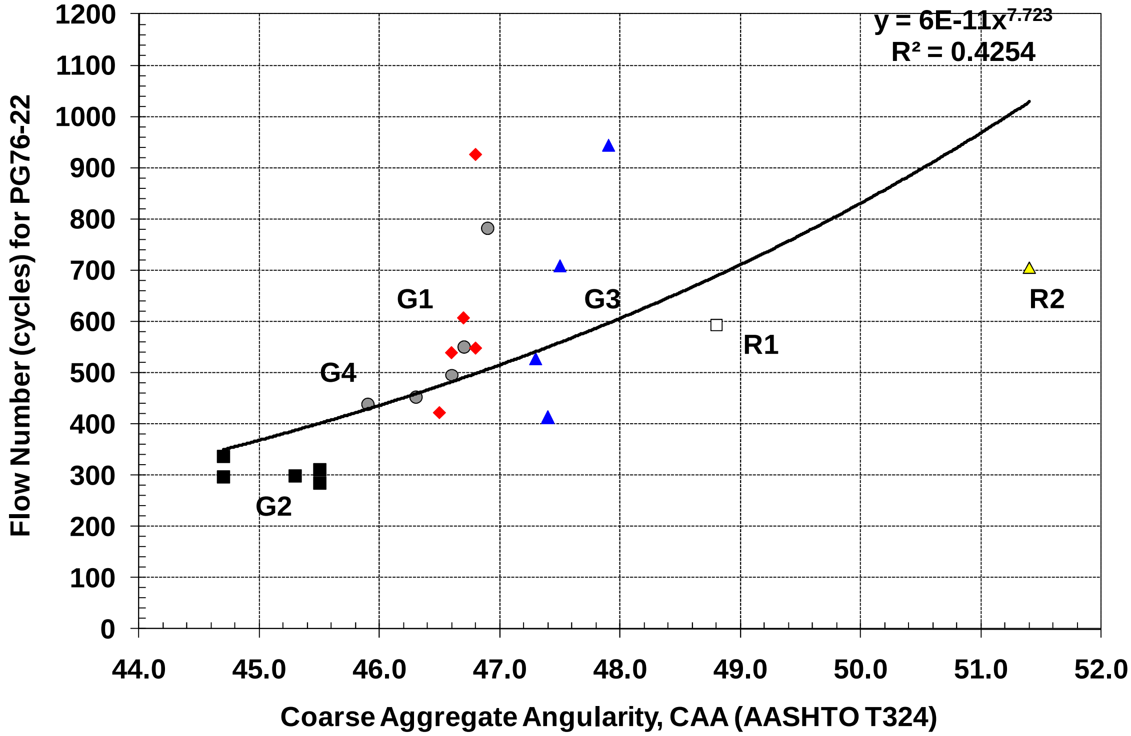


Figure 6.30 – Flow Number vs Uncompacted Voids Content for the PG76-22 Asphalt Binder

6.2.3 – Flow Number vs AIMS Angularity and Texture Index

The Aggregate Imaging System (AIMS) was used to quantify the angularity and texture properties of the different aggregate sources and gravel blends. The potential benefit of using the AIMS device is it is promoted as unbiased assessment of aggregate angularity and texture properties, unlike some tests that rely on the User's perception/opinion (ASTM D5821) and/or manipulated gradations or sample preparation procedures (AASHTO T326).

The AIMS Angularity and Texture Index properties shown with the corresponding Flow Number results are shown in Figures 6.31 through 6.33 for the AIMS Texture and 6.34 through 6.36 for the AIMS Angularity. The test results indicate that a good correlation was not able to be achieved between the AIMS Angularity and Texture Indexes and the Flow Number determined for the various mixtures. The potential reason for this may be explained by two reasons:

1. Although the AIMS device is supposed to eliminate User bias in the testing, there still exists some User input that may result in errors/inconsistencies with the measurements. This comes in the form of how the User places the aggregate particle on the imaging tray. If a crushed gravel particle is placed down in a manner hiding the crushed face, it will appear smoother and less angular. This may be a reason for the lack of sensitivity in the AIMS measurements.

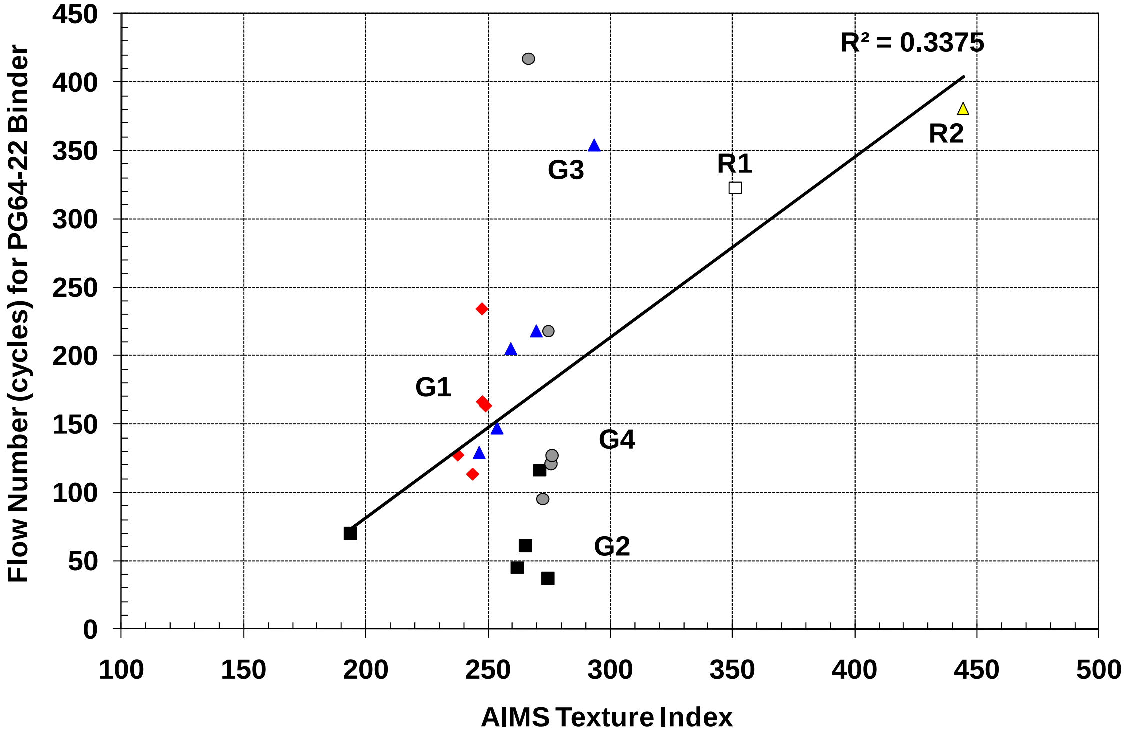


Figure 6.31 – AIMS Texture Index vs Flow Number for the PG64-22 Asphalt Binder

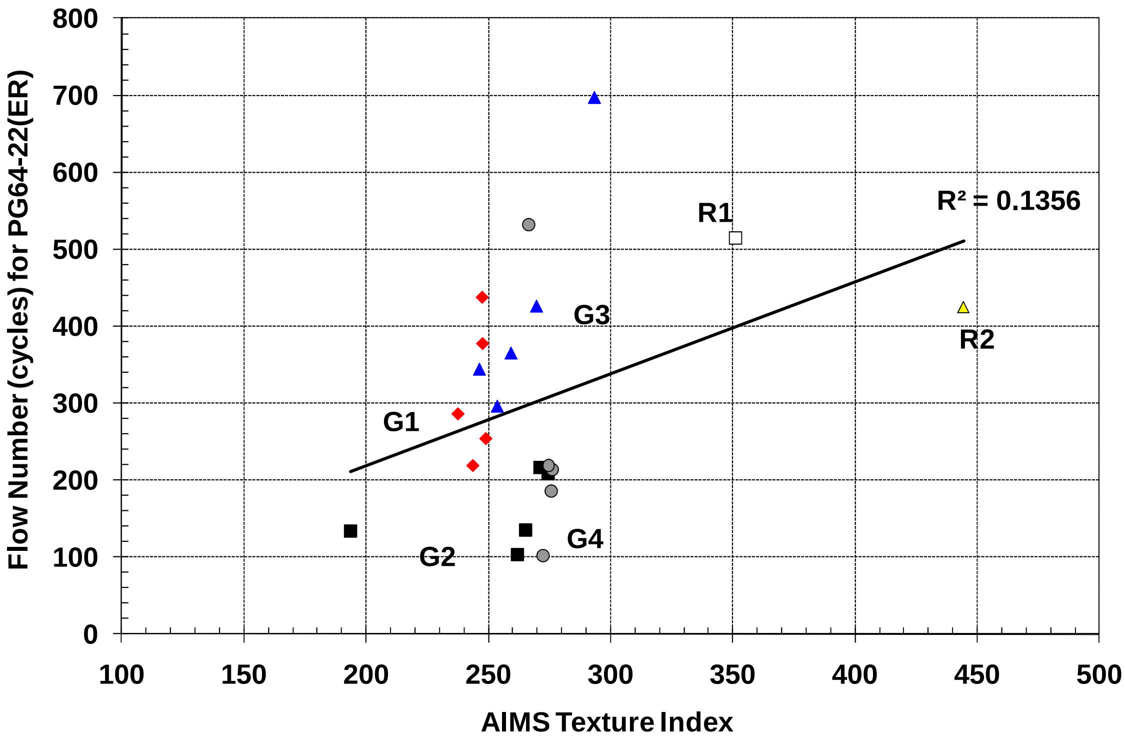


Figure 6.32 – AIMS Texture Index vs Flow Number for the PG64-22(ER) Asphalt Binder

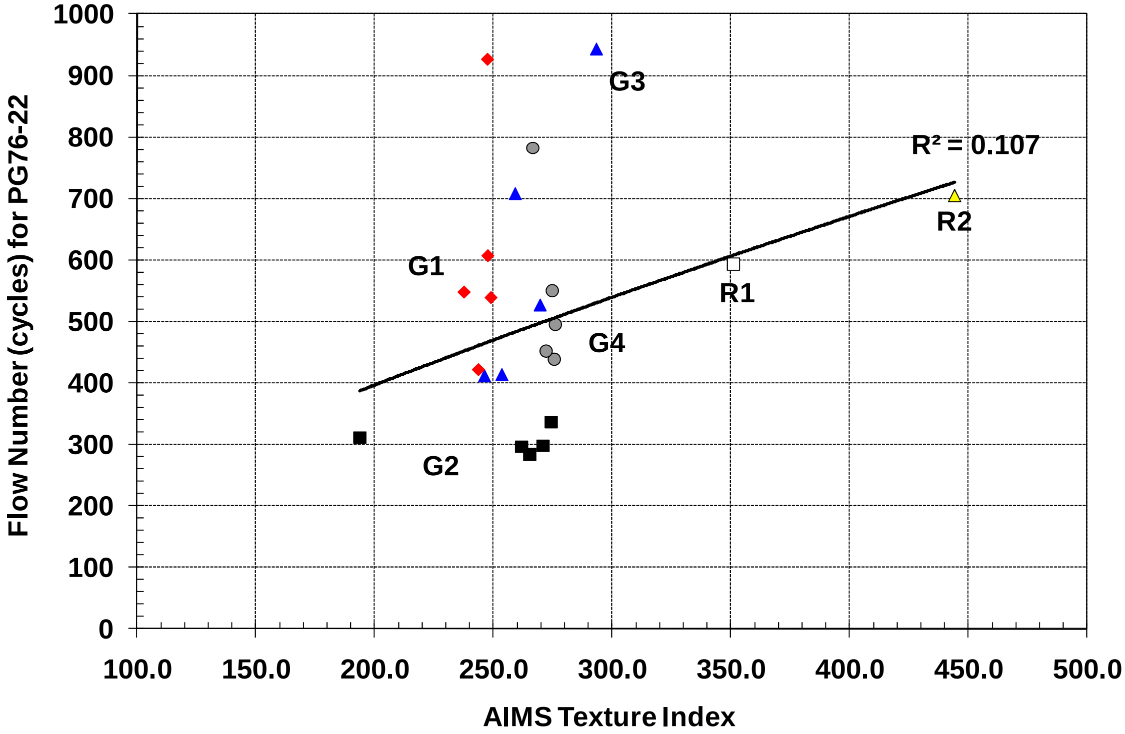


Figure 6.33 – AIMS Texture Index vs Flow Number for the PG76-22 Asphalt Binder

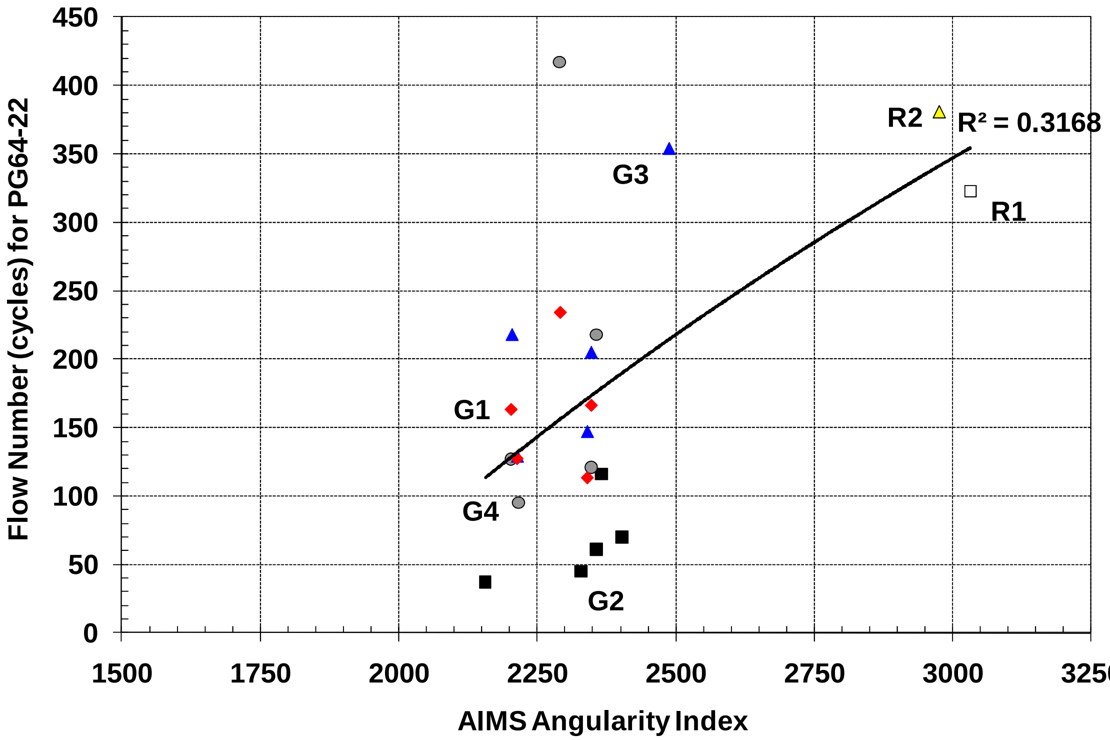


Figure 6.34 – AIMS Angularity Index vs Flow Number for the PG64-22 Asphalt Binder

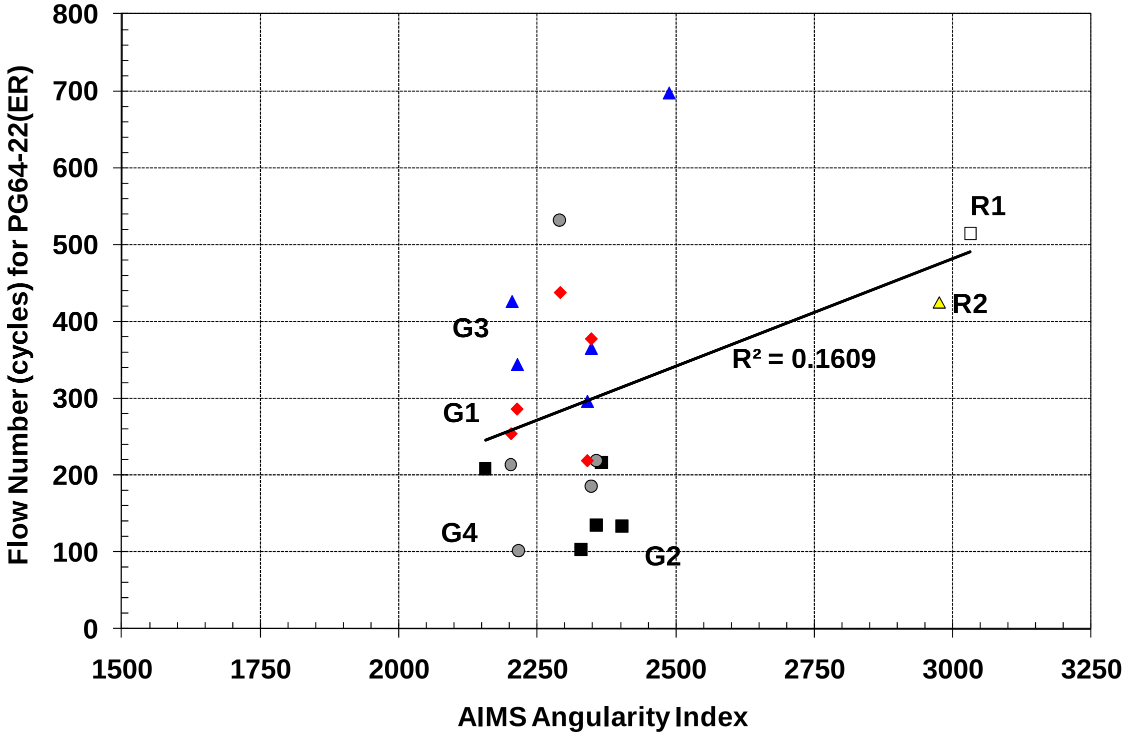


Figure 6.35 – AIMS Angularity Index vs Flow Number for the PG64-22(ER) Asphalt Binder

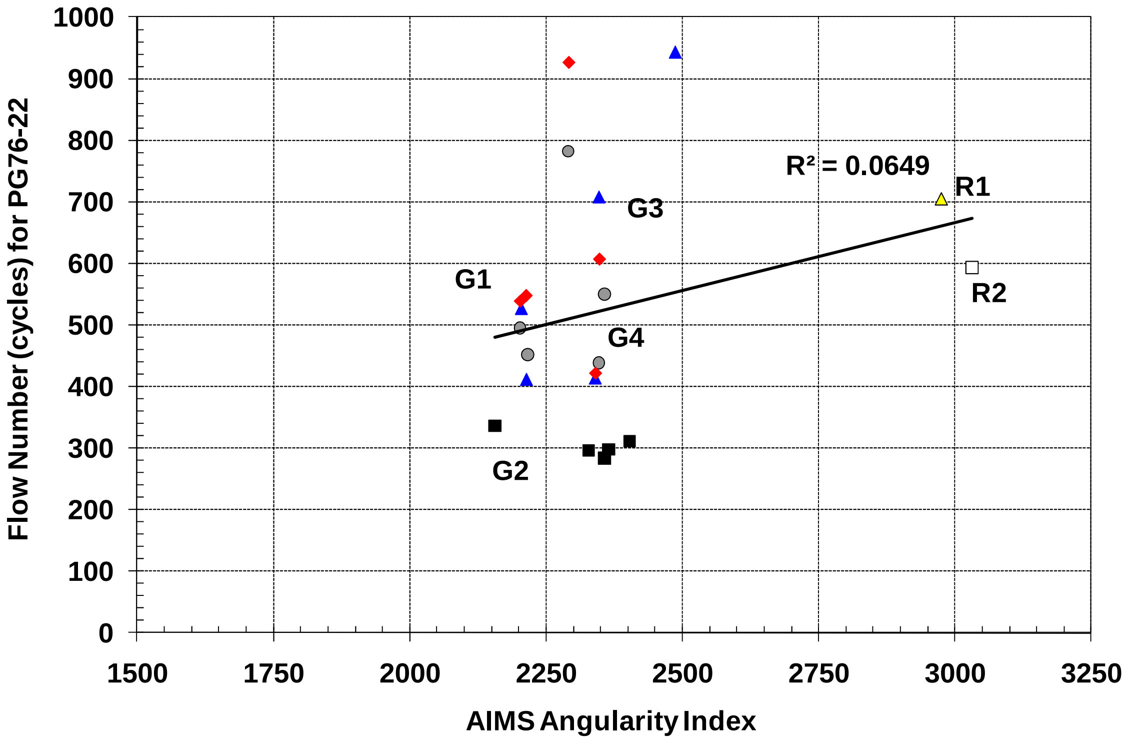


Figure 6.36 – AIMS Angularity Index vs Flow Number for the PG76-22 Asphalt Binder

2. Another possible reason for the lack of correlation is the interaction between the texture and angularity in generating internal shear strength of the asphalt mixture. The Uncompacted Voids Content (AASHTO T326), as shown earlier in Chapter 4 with the comparisons between the AIMS device and the Uncompacted Voids Content, is correlated to both the Texture Index and the Angularity Index. Therefore, attempting to look at how either texture or angularity alone compares to rutting may not be appropriate. It would appear some manner of combining both indexes would be a better means of comparing to the asphalt mixture shear strength (i.e. – permanent deformation resistance). It was beyond the scope of this project to look at a combined texture/angularity index from the AIMS device, although several attempts at looking at non-linear regressions were unsuccessful.

6.3 – Summary of Results from Permanent Deformation Testing

The research project evaluated 396 Asphalt Pavement Analyzer and 198 Flow Number test specimens in an attempt to determining if; 1) Coarse Aggregate Angularity, as determined by ASTM D5821, correlated to the permanent deformation performance, and 2) If additional aggregate angularity and texture measurements, in particular the AIMS device and Uncompacted Voids Content (AASHTO T326), correlated to the laboratory permanent deformation testing. The preliminary evaluation of the test data indicated;

1. The Coarse Aggregate Angularity (CAA) angularity rankings, as determined by ASTM D5821, did not match general rutting performance for either the Asphalt Pavement Analyzer or the Flow Number test. Gravels mixes, that had Coarse Aggregate Angularity (CAA) measurements of 100/99, did not perform as well as gravel mixes with CAA measurements of 100/96. Based on the idea of the CAA measurements, this should have been reversed. Also, some of the 100/99 CAA mixtures performed equal to or better than one of the 100/100 crushed stone sources. Further regression analysis looking at how well the Asphalt Pavement Analyzer rutting and Flow Number permanent deformation correlated to Fractured Face Count showed that a poor relationship exists between these parameters. This provides a strong argument that the NYSDOT should no longer require Coarse Aggregate Angularity specifications using ASTM D5821.
2. Increasing the asphalt binder high temperature stiffness from unmodified PG64-22 to the PG64-22(ER) and the PG76-22 resulted in lower permanent deformation values. It was also found that the correlations between the permanent deformation properties and the angularity parameters became poorer as the asphalt binder stiffness increased. This is a clear indication that the asphalt binder high temperature stiffness begins to dominate the angularity contribution to permanent deformation resistance at a certain point. This lends to the notion that poorer angular aggregates may still be able to be utilized in high traffic pavements as long as the high temperature PG grade of the asphalt binder is increased.
3. The Uncompacted Voids Content, as determined by AASHTO T326, provided a relatively good correlation to the Flow Number parameter, although not for the Asphalt Pavement Analyzer, for the PG64-22 asphalt binder. This is most likely

due to the incorporation of applied confining pressure in the Flow Number test which increased the contribution of the frictional properties of the aggregates in resisting permanent deformation. Although the Asphalt Pavement Analyzer did appear to be sensitive to the asphalt binder high temperature grade (or stiffness), it may not be sensitive enough to the narrow changes in aggregate angularity evaluated in this study.

4. The AIMS Angularity and Texture Index did not correlate well to the permanent deformation results of the Asphalt Pavement Analyzer and Flow Number test. It is hypothesized that this may be due to the interaction of texture and angularity in the development of shear strength in the asphalt mixture. With the Uncompacted Voids Content test correlating well to both the AIMS Texture and Angularity Index, and the Uncompacted Voids Content also correlating well to the permanent deformation testing of the Flow Number, comparing only angularity or texture may not be appropriate. It appears there needs to be a means of combining the AIMS Angularity and Texture measurements into a single index. Although this was beyond the scope of this research, simple non-linear multiple regression techniques were attempted but unsuccessful at generating a regression equation that combines both the AIMS Texture and Angularity measurements relating them to the permanent deformation test results of the Flow Number test.

CHAPTER 7

Statistical Analysis of Data

A statistical analysis was conducted using a Student's t-test analysis (two sample assuming equal or unequal variances). The analysis was utilized to determine if the samples were statistically equal or statistically not equal among the common test results and parameters. A 95 percent confidence interval was chosen for the analysis. A similar type of statistical analysis was conducted by Jones et al. (1998) to evaluate the performance of modified asphalts from mixture testing and therefore was thought to be suitable to be used for this research.

The formula for the independent samples t-test employing a pooled variance is (Dretzke, 2001)

$$t = \frac{\left(\overline{X_1} - \overline{X_2}\right) - \left(\mu_1 - \mu_2\right)}{S_{(\overline{X_1} - \overline{X_2})}} \tag{5}$$

where,

$\left(\overline{X_1} - \overline{X_2}\right)$ - difference between the two sample means
$\left(\mu_1 - \mu_2\right)$ - the hypothesized difference between the population means
$S_{(\overline{X_1} - \overline{X_2})}$ - the standard error of the difference

The standard error is calculated using a pooled variance estimate. The formula for the pooled variance is

$$S^2_{pooled} = \frac{(n_1 - 1)S_1^2 + (n_2 - 1)S_2^2}{(n_1 - 1) + (n_2 - 1)} \tag{6}$$

where,

S_1^2 - the variance in sample 1
S_2^2 - the variance in sample 2
n_1 – the number of observations in sample 1
n_2 – the number of observations in sample 2

The pooled variance estimate is the weighted average of the sample variances where each variance is weighted by its respective degrees of freedom. The formula for the standard error of the difference is given by

$$S_{\left(\overline{\overline{X_1} - \overline{X_2}}\right)} = \sqrt{\left(\frac{S^2_{pooled}}{n_1} + \frac{S^2_{pooled}}{n_2}\right)} \tag{7}$$

The assumptions underlying the independent samples of the t-test are:

1. Observations are randomly sample from population 1 and population 2.
2. The sample of observations from population 1 is independent of the sample observations from population 2.
3. Observations are normally distributed in both population 1 and population 2.
4. The variances of population 1 and population 2 are unknown but are equal.

Prior to using the Student's t-test, an F-test was utilized to first determine whether the variances of the datasets were equal or unequal. After determining whether they were equal or unequal, the appropriate Student's t-test (equal or unequal variances) was then used to determine if the datasets were statistically equal at a 95% Confidence Level.

The main purpose of the statistical analysis was to determine;

1. At what level of angularity in the gravel mixes does the permanent deformation properties not equal the performance of the crushed stone mixes;
2. At what grade of asphalt binder does the permanent deformation performance of the gravel mixes equal that of the crushed stone mixes.

Based on the work conducted in Chapter 6, it is evident that the Asphalt Pavement Analyzer was not sensitive enough to distinguish between the general angularity differences utilized in this research study. Therefore, the statistical analysis reported in Chapter 6 is only conducted using the Flow Number results generated during the work in Chapter 6.

7.1 – Statistical Analysis of Gravel Mixture Angularity to Crushed Stone

The F- and t-Tests were used to determine whether or not the repeated load permanent deformation properties of the gravel mixes, as determined with the Flow Number, were Statistically Equal at a 95% Confidence Level to the crushed stone mixes for the same PG grade asphalt binder used. For presentation purposes, the statistically summary tables are shown using both the Coarse Aggregate Angularity (CAA), as determined using ASTM D5821 and currently specified by NYSDOT, and also the Uncompacted Voids Content, as determined using AASHTO T326, which was found to correlate to the confined, repeated load permanent deformation tests.

Tables 7.1 through 7.4 show the statistical analysis represented with the CAA and Uncompacted Voids Content for the four gravel mixes evaluated in the study. A "Y" indicates that the permanent deformation performance of the gravel mixture at that angularity level was Statistically Equal at a 95% Confidence Level. Meanwhile, a "N" indicates that the permanent deformation performance was Not Statistically Equal. A "N (Y)" indicates that the permanent deformation results were Not Statistically Equal, however, the performance of the gravel mixture was actually better than the crushed stone mixture. This was given a "Y" simply because the study is trying to determine if the gravel mixes perform as good as, or better, than the crushed stone mixes.

Table 7.1 – t-Test Results for G1 Gravel Mixes

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22		G1				
		100/96	97/93	94/90	91/87	88/84
R1	100/100	N	N	N	N	N
R2	100/100	N	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G1				
		100/96	97/93	94/90	91/87	88/84
R1	100/100	Y	Y	N	Y	N
R2	100/100	Y	Y	N	Y	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G1				
		100/96	97/93	94/90	91/87	88/84
R1	100/100	N (Y)	Y	Y	Y	N
R2	100/100	Y	Y	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22		G1				
		46.8	46.7	46.6	46.8	46.5
R1	48.8	N	N	N	N	N
R2	51.4	N	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G1				
		46.8	46.7	46.6	46.8	46.5
R1	48.8	Y	Y	N	Y	N
R2	51.4	Y	Y	N	Y	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G1				
		46.8	46.7	46.6	46.8	46.5
R1	48.8	N (Y)	Y	Y	Y	N
R2	51.4	Y	Y	N	N	N

Table 7.2 – t-Test Results for G2 Gravel Mixes

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22		G2				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	N	N	N	N	N
R2	100/100	N	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G2				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	N	N	N	N	N
R2	100/100	N	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G2				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	N	N	N	N	N
R2	100/100	N	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22		G2				
		45.5	45.5	45.3	44.7	44.7
R1	48.8	N	N	N	N	N
R2	51.4	N	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G2				
		45.5	45.5	45.3	44.7	44.7
R1	48.8	N	N	N	N	N
R2	51.4	N	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G2				
		45.5	45.5	45.3	44.7	44.7
R1	48.8	N	N	N	N	N
R2	51.4	N	N	N	N	N

Table 7.3 – t-Test Results for G3 Gravel Mixes

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22		G3				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	Y	Y	Y	N	N
R2	100/100	Y	Y	Y	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G3				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	Y	Y	Y	N	N
R2	100/100	Y	Y	Y	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G3				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	N (Y)	N (Y)	Y	N	Y
R2	100/100	Y	Y	N	N	Y

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22		G3				
		47.9	47.5	47.3	47.4	47.4
R1	48.8	Y	Y	Y	N	N
R2	51.4	Y	Y	Y	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G3				
		47.9	47.5	47.3	47.4	47.4
R1	48.8	Y	Y	Y	N	N
R2	51.4	Y	Y	Y	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G3				
		47.9	47.5	47.3	47.4	47.4
R1	48.8	N (Y)	N (Y)	Y	N	Y
R2	51.4	Y	Y	N	N	Y

Table 7.4 – t-Test Results for G4 Gravel Mixes

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22		G4				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	Y	N	N	N	N
R2	100/100	Y	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G4				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	N (Y)	Y	N	N	N
R2	100/100	N (Y)	Y	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G4				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	N (Y)	Y	Y	Y	N
R2	100/100	Y	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22		G4				
		46.9	46.7	46.6	46.3	45.9
R1	48.8	Y	N	N	N	N
R2	51.4	Y	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G4				
		46.9	46.7	46.6	46.3	45.9
R1	48.8	N (Y)	Y	N	N	N
R2	51.4	N (Y)	Y	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G4				
		46.9	46.7	46.6	46.3	45.9
R1	48.8	N (Y)	Y	Y	Y	N
R2	51.4	Y	N	N	N	N

The tables indicate that:

- When the asphalt mixtures used a PG64-22 asphalt binder, Statistically Equal permanent deformation properties were found when the Uncompacted Voids Content was 46.9% or greater.
- When the asphalt mixture used the PG64-22(ER) asphalt binder, Statistically Equal permanent deformation properties were found when the Uncompacted Voids Content was 46.7% or greater.
- When the asphalt mixture used the PG76-22 asphalt binder, Statistically Equal permanent deformation properties were found when the Uncompacted Voids Content was 46.3% or greater.

Based on the statistical analysis conducted when comparing the gravel and crushed stone mixes using the identical PG binder grade, it appears that permanent deformation properties were Statistically Equal at a 95% Confidence Level when the coarse aggregate portion of the asphalt mixture was able to achieve an Uncompacted Voids Content of 47% or greater (conservative approach). An Uncompacted Voids Content of 47% did not correlate to a specific CAA and was found to be source dependent, which is most likely a function of the raw stock gravel feed and crushing process of the gravel supplier.

7.2 – Statistical Analysis for the Potential of PG Grade "Bumping"

An additional statistical analysis, using the same methodology as before, was conducted to determine if there was a potential for asphalt suppliers to still utilize gravels of lesser angularity by increasing the high temperature PG graded (called grade "bumping") of the asphalt binder. This was accomplished by statistically comparing the permanent deformation properties of the crushed stone sources using the unmodified PG64-22 asphalt binder with the gravel mixes using the polymer-modified PG64-22(ER) and PG76-22 asphalt binders. The resulting F- and t-test results are shown in Tables 7.5 through 7.8.

Table 7.5 – t-Test Results for G1 Gravel Mixes (Grade “Bumping” Analysis)

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G1				
		100/96	97/93	94/90	91/87	88/84
R1	100/100	N (Y)	N (Y)	N (Y)	N (Y)	Y
R2	100/100	N (Y)	N (Y)	N (Y)	N (Y)	Y

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G1				
		46.8	46.7	46.6	46.8	46.5
R1	48.8	N (Y)	Y	N	Y	N
R2	51.4	N (Y)	Y	N	Y	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G1				
		46.8	46.7	46.6	46.8	46.5
R1	48.8	N (Y)	N (Y)	N (Y)	N (Y)	Y
R2	51.4	N (Y)	N (Y)	N (Y)	N (Y)	Y

Table 7.6 – t-Test Results for G2 Gravel Mixes (Grade "Bumping" Analysis)

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G2				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	Y	Y	Y	Y	Y
R2	100/100	Y	Y	Y	Y	Y

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G2				
		45.5	45.5	45.3	44.7	44.7
R1	48.8	N	N	N	N	N
R2	51.4	N	N	N	N	N

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G2				
		45.5	45.5	45.3	44.7	44.7
R1	48.8	Y	Y	Y	Y	Y
R2	51.4	Y	Y	Y	Y	Y

Table 7.7 – t-Test Results for G3 Gravel Mixes (Grade "Bumping" Analysis)

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G3				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	N (Y)	N (Y)	N (Y)	Y	Y
R2	100/100	N (Y)	N (Y)	N (Y)	Y	Y

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G3				
		47.9	47.5	47.3	47.4	47.4
R1	48.8	N (Y)	Y	N	Y	Y
R2	51.4	N (Y)	Y	N (Y)	Y	Y

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G3				
		47.9	47.5	47.3	47.4	47.4
R1	48.8	N (Y)	N (Y)	N (Y)	Y	Y
R2	51.4	N (Y)	N (Y)	N (Y)	Y	Y

Table 7.8 – t-Test Results for G4 Gravel Mixes (Grade "Bumping" Analysis)

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G4				
		100/99	97/96	94/93	91/90	88/87
R1	100/100	N (Y)	N (Y)	N (Y)	Y	Y
R2	100/100	N (Y)	N (Y)	N (Y)	Y	Y

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG64-22 (ER)		G4				
		46.9	46.7	46.6	46.3	45.9
R1	48.8	N (Y)	N	N	N	Y
R2	51.4	N (Y)	N	N	N	Y

Statistically Equal 95% Confidence Level Flow Number Test Results						
PG76-22		G4				
		46.9	46.7	46.6	46.3	45.9
R1	48.8	N (Y)	N (Y)	N (Y)	Y	Y
R2	51.4	N (Y)	N (Y)	N (Y)	Y	Y

The tables indicate:

- When the asphalt binder grade is "bumped" from an unmodified PG64-22 to a polymer modified PG64-22(ER), the gravel mixtures that had an Uncompacted Voids Content of 45.9 or greater achieved permanent deformation Flow Number values Statistically Equal at a 95% Confidence Level to the crushed stone mixtures that used the unmodified PG64-22 asphalt binder. This would suggest that asphalt suppliers could utilize gravel aggregates of a lesser Uncompacted Voids Content (less than 45.9) than the crushed stone mixtures as long as the asphalt binder used was a polymer modified PG64-22(ER).
- When the asphalt binder grade is "bumped" from an unmodified PG64-22 to a polymer modified PG76-22, the gravel mixtures that had an Uncompacted Voids Content of 44.7 or greater achieved permanent deformation Flow Number values Statistically Equal at a 95% Confidence Level to the crushed stone mixtures that used the unmodified PG64-22 asphalt binder. This would suggest that asphalt suppliers could utilize gravel aggregates of a lesser Uncompacted Voids Content (greater than 44.7) than the crushed stone mixtures as long as the asphalt binder grade used was a polymer modified PG76-22 asphalt binder.

7.3 – Summary of Statistical Analysis

The F- and t-Tests were used to determine if the gravel mixtures were Statistically Equal at a 95% Confidence Level with the crushed stone mixtures, as well as to determine if increasing the PG grade of the asphalt binder, when used with lesser angular gravels, could provide permanent deformation properties Statistically Equal to the crushed stone mixtures. The statistical analysis indicated:

1. Gravel mixtures that had an Uncompacted Voids Content of 47% (actually 46.9% rounded up) should provide similar permanent deformation properties similar to the crushed stone mixtures. As the asphalt binder was “bumped” up to the polymer modified PG64-22(ER) and PG76-22, this value decreased slightly, 46.7 and 46.3, respectively. However, as a conservative value, the data suggests that asphalt suppliers could use gravel mixtures on heavy volume traffic roads, greater than 30 million ESAL’s, as long as the Uncompacted Voids Content of the coarse aggregates was 47% or greater.
2. If an asphalt supplier is not capable of obtaining gravel sources with an Uncompacted Voids Content of 47% or greater, the asphalt supplier can “bump” the asphalt binder grade from an unmodified PG64-22 to a polymer modified PG64-22(ER) or a polymer modified PG76-22. If the asphalt supplier chooses to utilize the PG64-22(ER) instead of the PG64-22, the statistical analysis suggests the Uncompacted Voids Content can be reduced from the 47% to a value of 46% and still provide permanent deformation performance similar to the crushed stone mixtures. The statistical analysis also suggests that an asphalt supplier can use gravels with an Uncompacted Voids Content as low as 45%, if the asphalt supplier is using a polymer modified PG76-22. There is evidence showing that even lower Uncompacted Voids Content may be used (test data a low as 44.7%), however, additional testing would be required to verify what the actual lower value is.

CHAPTER 8

Conclusions and Recommendations

8.1 - Conclusions

A laboratory investigation was conducted to determine if gravel mixtures with Coarse Aggregate Angularity (CAA), determined using ASTM D5821, values of 100/95 could be utilized for heavy volume pavements (greater than30 million ESAL's) in New York State. The research project evaluated both aggregate angularity tests and asphalt mixture permanent deformation tests of crushed gravel and crushed stone aggregate sources. The research project also evaluated the impact of polymer modified asphalt binders, PG64-22(ER) and PG76-22, and how the modified binders influenced the permanent deformation performance. Based on the testing conducted in the study, the following conclusions are drawn:

- Aggregate testing showed that the Coarse Aggregate Angularity (CAA) values, as determined using ASTM D5821, *Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate*, did not correlate to angularity/texture type tests, such as the Uncompacted Void Content, AASHTO T326, *Uncompacted Void Content of Coarse Aggregate (As Influenced by Particle Shape, Surface Texture, and Grading),* or the Aggregate Imaging System (AIMS) device. This indicates that indexing crushed gravels according to ASTM D5821 may not necessarily provide adequate rutting resistance for heavy volume pavements, or in some occasions, may be under-determining the true angularity properties of the gravel and restrict the gravel from being used.
- The Uncompacted Void Content (UVC) parameters were found to be related to both the aggregate angularity and texture, as determined with the AIMS device. In fact, it was found that a stronger correlation was found between the UVC and AIMS Texture Index. The testing of the aggregate sources, both crushed gravels and crushed stone, showed that the crushed stone UVC ranged from 48.8% to 51%. Meanwhile, the crushed gravels had UVC values ranging from 45.5% to 47.9%.
- AIMS device testing of the crushed gravel and crushed stone aggregates showed that the crushed stone aggregates obtained average AIMS Angularity values 21% greater than the crushed gravels (2962 and 2331, respectively). Meanwhile, the crushed stone aggregates obtained average AIMS Texture values 34% greater than the crushed gravels (416 and 276, respectively).
- Asphalt Pavement Analyzer (APA) rutting properties suggests that the device is more sensitive to the asphalt binder stiffness than the range in aggregate angularities measured in this study. The APA rutting clearly decreased as the PG grade of the asphalt binder increased (i.e. – unmodified PG64-22 to polymer modified PG64-22 to polymer modified PG76-22). However, the APA did not correlate well with the aggregate angularities measured in the study. At times, asphalt mixture that had the same asphalt binder but with lower aggregate angularities achieved lesser APA rutting values than asphalt mixtures with higher

aggregate angularities. When comparing the APA rutting values to the CAA (ASTM D5821), UVA (AASHTO T326) and AIMS Angularity and Texture Index, in all cases poor correlations were found.

- The Flow Number permanent deformation results were found to be both sensitive to aggregate angularity, as determined using the UVC (AASHTO T326), and asphalt binder stiffness. Flow Number values increased as the UVC values increased and asphalt binder high temperature PG grade increased (i.e. – unmodified PG64-22 to polymer modified PG64-22(ER) to polymer modified PG76-22. However, it was also found that the Flow Number test did not correlate well to the AIMS Texture or Angularity Indexes. This may be due to the need for the interaction between both texture and angularity to generate shear strength and perhaps simply focusing on one does not truly represent the shear strength potential. Since the Flow Number was found to be sensitive to the aggregate angularities evaluated in this study, it was decided to further use the device during the statistical analysis portion of the project.
- Statistical analysis, using the F- and t-Tests, were conducted to evaluate; 1) At what aggregate angularity value did the crushed gravel mixtures behave Statistically Equal to the crushed stone mixtures, and 2) Is it possible to utilize lower aggregate angularity values, in conjunction with a "bump" in the asphalt binder PG grade, and still achieve permanent deformation properties similar to the crushed stone mixtures. The statistically analysis indicated;
    - It appears that permanent deformation properties were Statistically Equal at a 95% Confidence Level when the coarse aggregate portion of the asphalt mixture was able to achieve an Uncompacted Voids Content of 47% or greater (conservative approach). An Uncompacted Voids Content of 47% did not correlate to a specific CAA and was found to be source dependent, which is most likely a function of the raw stock gravel feed and crushing process of the gravel supplier.
    - When the asphalt binder grade is "bumped" from an unmodified PG64-22 to a polymer modified PG64-22(ER), the gravel mixtures that had an Uncompacted Voids Content of 45.9 or greater achieved permanent deformation Flow Number values Statistically Equal at a 95% Confidence Level to the crushed stone mixtures that used the unmodified PG64-22 asphalt binder. This would suggest that asphalt suppliers could utilize gravel aggregates of a lesser Uncompacted Voids Content (greater than 45.9) than the crushed stone mixtures as long as the asphalt binder used was a polymer modified PG64-22(ER).
    - When the asphalt binder grade is "bumped" from an unmodified PG64-22 to a polymer modified PG76-22, the gravel mixtures that had an Uncompacted Voids Content of 44.7 or greater achieved permanent deformation Flow Number values Statistically Equal at a 95% Confidence Level to the crushed stone mixtures that used the unmodified PG64-22 asphalt binder. This would suggest that asphalt suppliers could utilize gravel aggregates of a lesser Uncompacted Voids Content (greater than 44.7) than the crushed stone mixtures as long as the asphalt binder grade used was a polymer modified PG76-22 asphalt binder.

The test results generated during the study clearly shows that the asphalt mixture's rutting performance is a function of both the coarse aggregate's uncompacted void content and also the non-recoverable creep compliance of the asphalt binder. Figure 8.1 shows the relationship between the AMPT's Flow Number, using the test parameters in this study, and the uncompacted void content of the coarse aggregates and the non-recoverable creep compliance. The correlation between the Predicted, using the non-linear regression function in Excel, and the Measured was good at 0.71. Although some scatter exists at the higher Flow Number values, the figure does provide evidence that it would be prudent to consider both coarse aggregate angularity, as determined in AASHTO T326, and the non-recoverable creep compliance when selecting asphalt mixtures to resist permanent deformation.

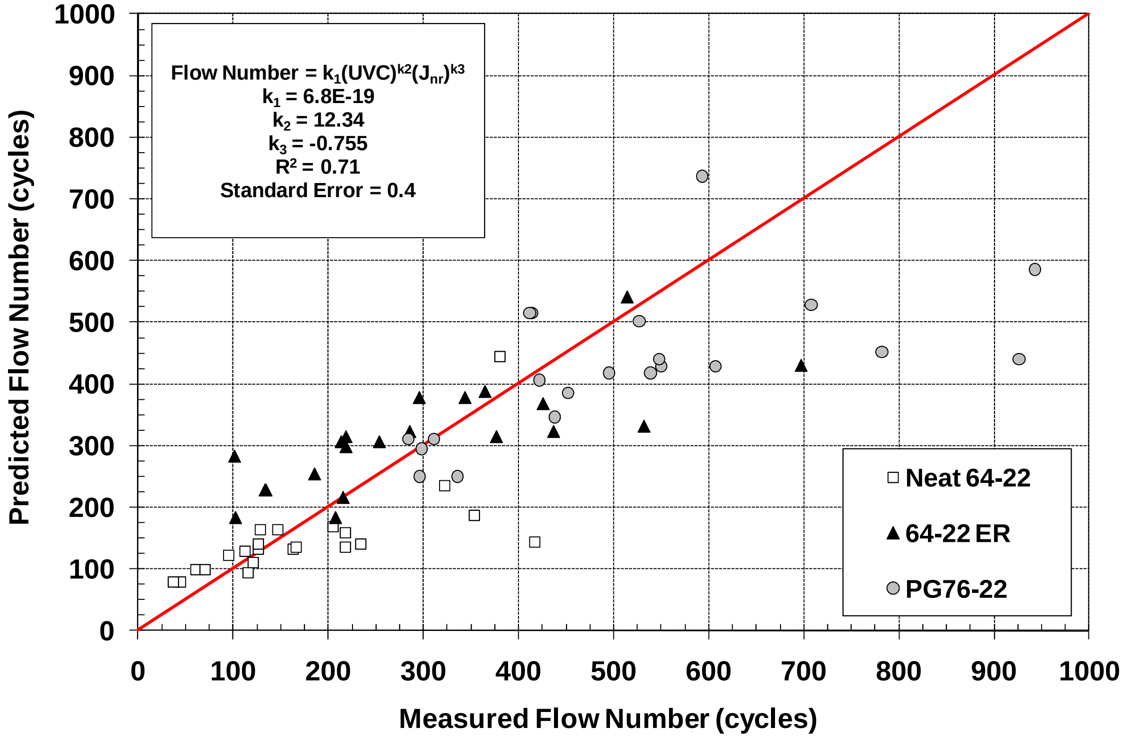


Figure 8.1 – Predicted vs Measured Flow Number Using Full Dataset

8.2 – Recommendations

Based on the testing conducted during this project with the selected materials utilized, the following recommendations are provided:

1. NYSDOT should adopt AASHTO T326, Uncompacted Void Content of Coarse Aggregate, test procedure to measure the texture and angularity of crushed aggregates (gravel and stone).
2. Gravel aggregate mixtures, obtaining an Uncompacted Void Content (UVC) of 47% or greater, can be used in asphalt mixtures for traffic levels greater than 30 million ESAL's.
3. The "bumping" of asphalt binders provided additional permanent deformation resistance, above that of the unmodified PG64-22 asphalt binder used in the study. Based on the results generated and analyzed during this project, Table 8.1 and/or 8.2 is recommended for use on pavements with traffic levels greater than 30 million ESAL's. The table provides recommendations for appropriate Uncompacted Void Content levels and the appropriate asphalt binder to be used.

To implement these recommendations, NYSDOT will need to change their current specifications and mixture design practices.

Table 8.1 – Recommended Gravel Aggregate Angularity and Asphalt Binder Grade for >30 Million ESAL Pavements

Minimum Uncompacted Void Content, % (AASHTO T326)	Minimum Asphalt Binder Grade
≥ 47%[1]	Unmodified PG64-22
≥ 46%	Polymer Modified PG64-22 (ER)
≥ 45%	Polymer Modified PG76-22

[1] - When staying within binder grade and not "bumping"

Table 8.2 – Recommended Gravel Aggregate Angularity and Multiple Stress Creep Recovery (MSCR) Non-recoverable Compliance (J_{nr}) for >30 Million ESAL Pavements

Minimum Uncompacted Void Content, % (AASHTO T326)	Minimum Jnr (Pa) at 64°C
≥ 47%[1]	≤ 2.50 Pa
≥ 46%	≤ 1.0 Pa
≥ 45%	≤ 0.60 Pa

[1] - When staying within binder grade and not "bumping"

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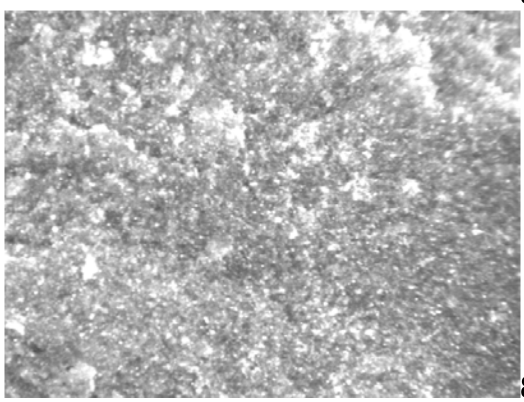
APPENDIX A – AIMS PICTURES OF GRAVEL BLENDS (VARYING UNCRUSHED COUNTS)

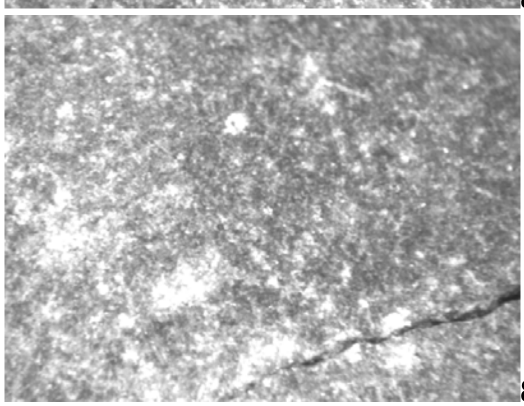
GRAVEL G1

88% Crushed - ¾” - ½” Angularity = 1889.13

88% Crushed - ¾” - ½” Angularity = 2229.60



88% Crushed - ¾” - ½” Texture = 297

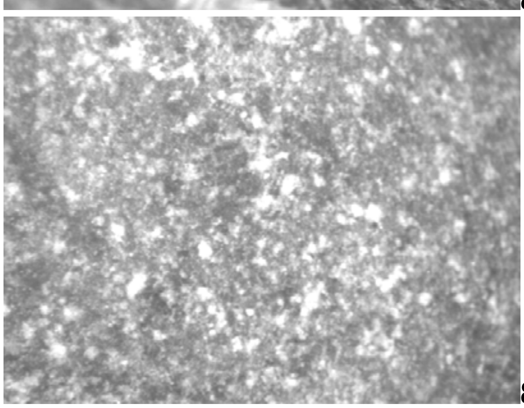


88% Crushed - ¾” - ½” Texture = 277

88% Crushed - ½” - 3/8” Angularity = 2135.81

88% Crushed - ½” - 3/8” Angularity = 2206.11

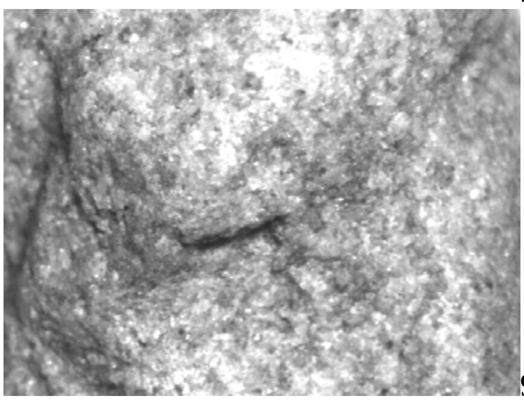
88% Crushed - ½” - 3/8” Texture = 278.5



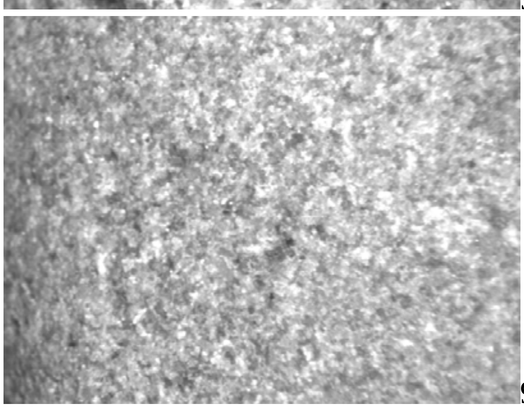
88% Crushed - ½” - 3/8” Texture = 278.5

91% Crushed - ¾” - ½” Angularity = 2119.82

91% Crushed - ¾” - ½” Angularity = 2061.03



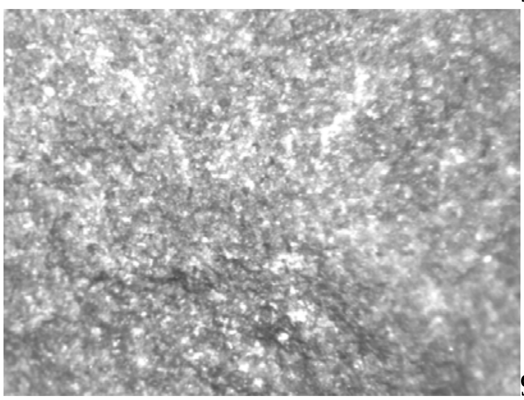
91% Crushed - ¾” - ½” Texture = 271



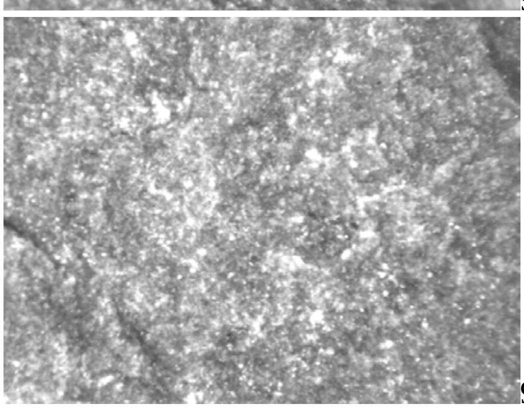
91% Crushed - ¾” - ½” Texture = 263

91% Crushed - ½” - 3/8” Angularity = 2611.23

91% Crushed - ½” - 3/8” Angularity = 2035.19



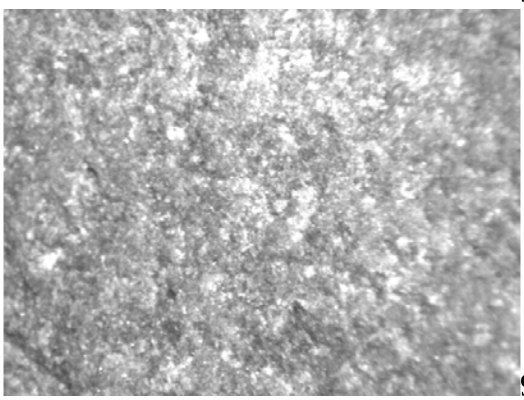
91% Crushed - ½” - 3/8” Texture = 270.4



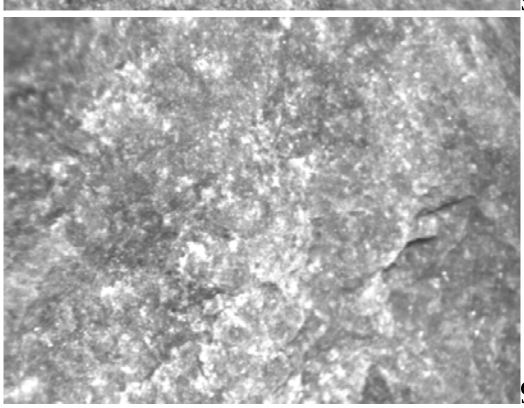
91% Crushed - ½” - 3/8” Texture = 282.5

94% Crushed - ¾” - ½” Angularity = 2285.56

94% Crushed - ¾” - ½” Angularity = 1968.13



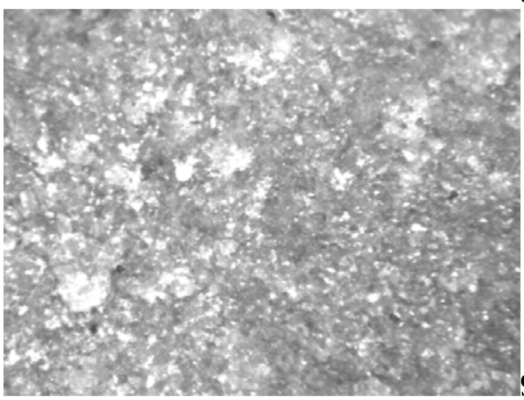
94% Crushed - ¾” - ½” Texture = 267.5



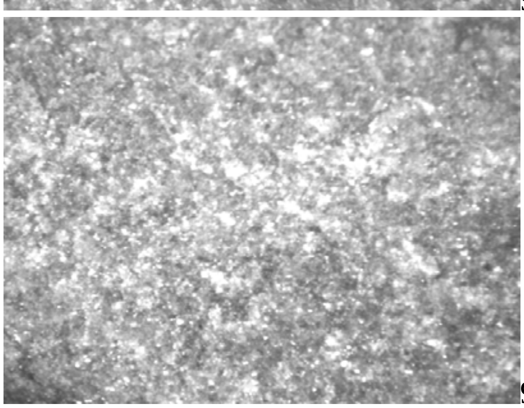
94% Crushed - ¾” - ½” Texture = 258.5

94% Crushed - ½” - 3/8” Angularity = 2612.78

94% Crushed - ½” - 3/8” Angularity = 1755.91



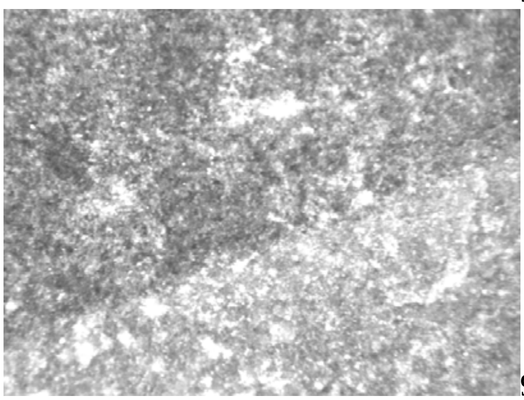
94% Crushed - ½” - 3/8” Texture = 266.5



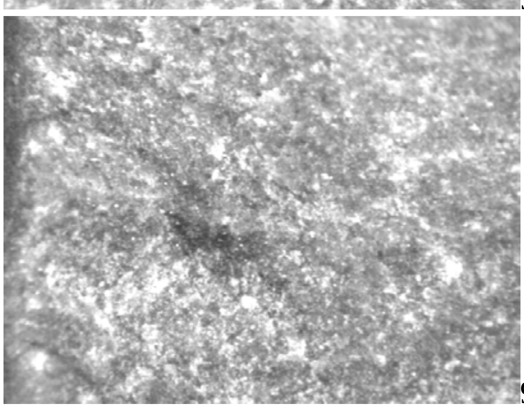
94% Crushed - ½” - 3/8” Texture = 280.5

97% Crushed - ¾" - ½" Angularity = 2703.12

97% Crushed - ¾" - ½" Angularity = 2070.36



97% Crushed - ¾" - ½" Texture = 264

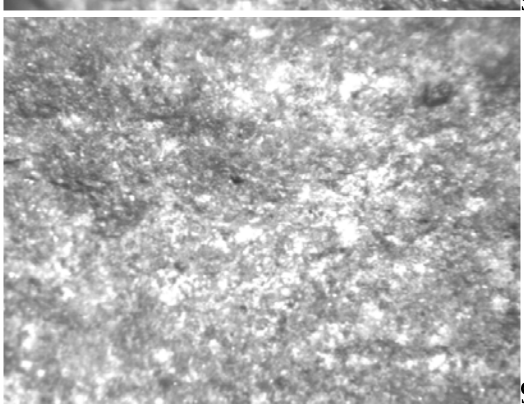


97% Crushed - ¾" - ½" Texture = 264

97% Crushed - ½” - 3/8” Angularity = 2093.95

97% Crushed - ½” - 3/8” Angularity = 2747.46

97% Crushed - ½” - 3/8” Texture = 325.5

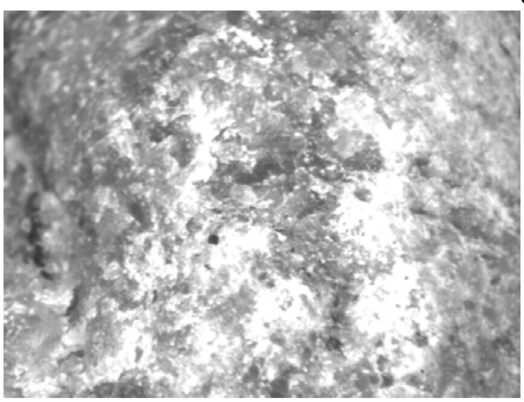


97% Crushed - ½” - 3/8” Texture = 301.5

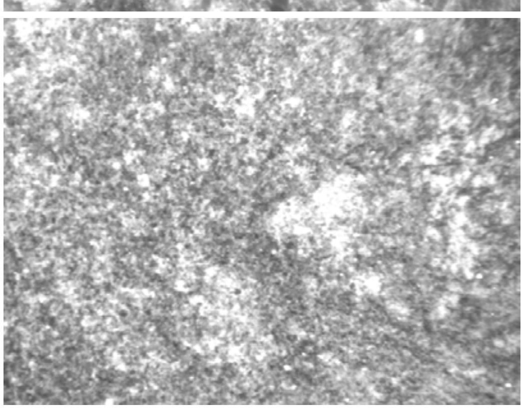
GRAVEL G2

88% Crushed - ¾” - ½” Angularity = 2386.77

88% Crushed - ¾” - ½” Angularity = 2265.47



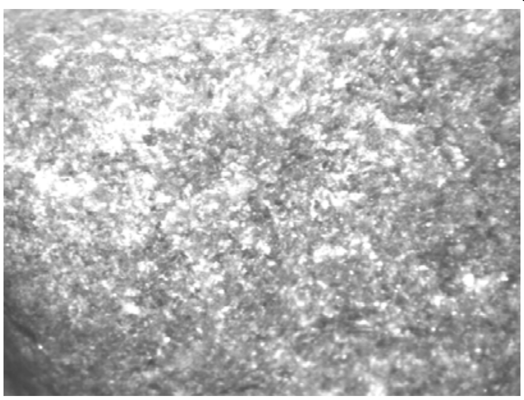
88% Crushed - ¾” - ½” Texture = 343.5



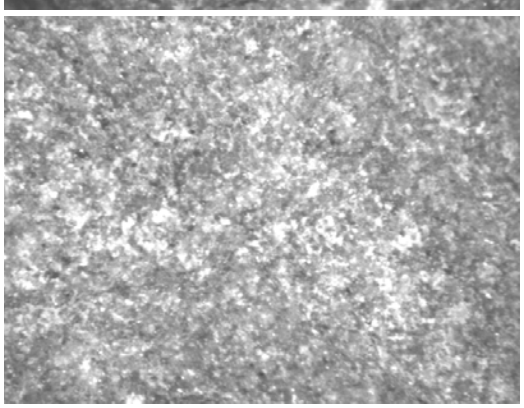
88% Crushed - ¾” - ½” Texture = 331.5

88% Crushed - ½” - 3/8” Angularity = 2212.68

88% Crushed - ½” - 3/8” Angularity = 2158.26



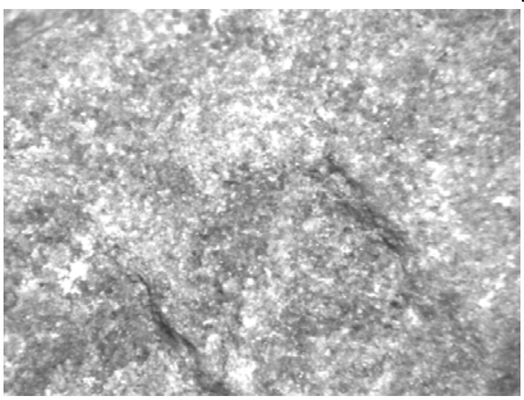
88% Crushed - ½” - 3/8” Texture = 293.5



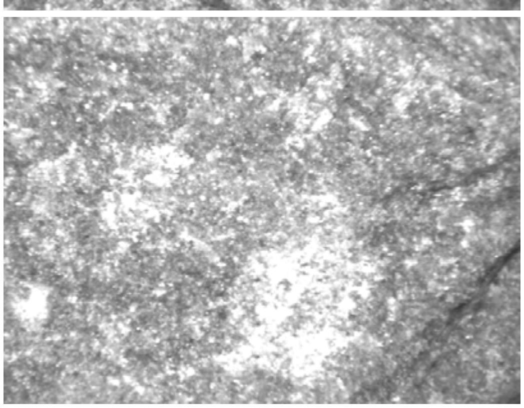
88% Crushed - ½” - 3/8” Texture = 274

91% Crushed - ¾” - ½” Angularity = 2110.72

91% Crushed - ¾” - ½” Angularity = 2254.08



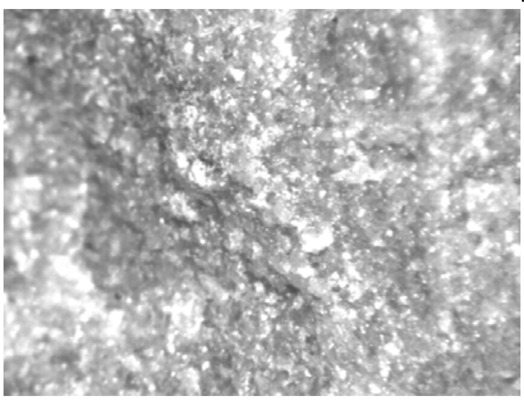
91% Crushed - ¾” - ½” Texture = 277



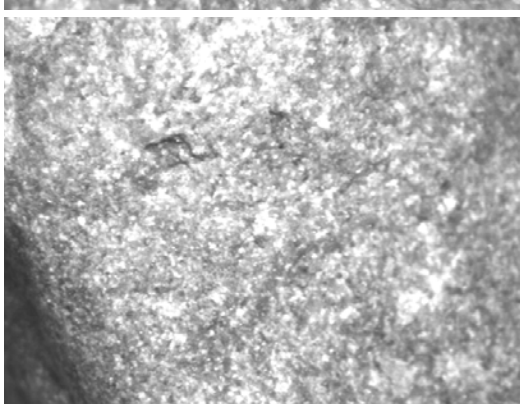
91% Crushed - ¾” - ½” Texture = 281

91% Crushed - ½” - 3/8” Angularity = 2238.33

91% Crushed - ½” - 3/8” Angularity = 2070.94



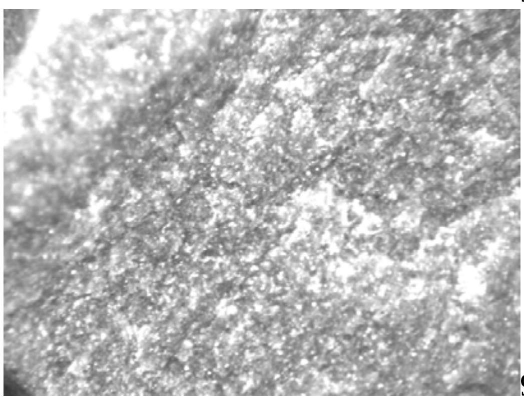
91% Crushed - ½” - 3/8” Texture = 317



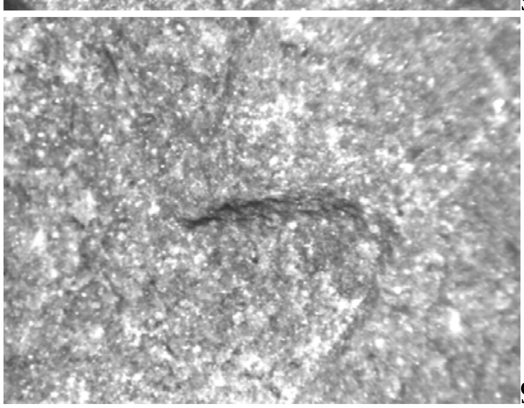
91% Crushed - ½” - 3/8” Texture = 290

94% Crushed - ¾” - ½” Angularity = 2199.07

94% Crushed - ¾” - ½” Angularity = 2102.2



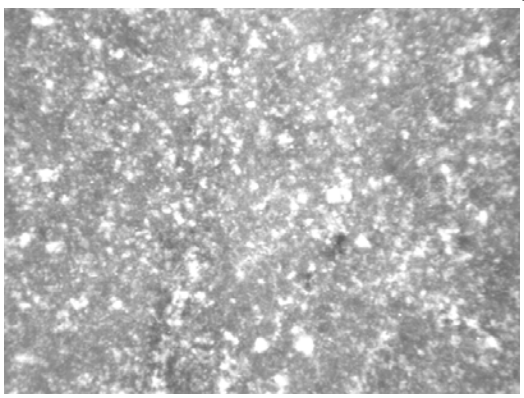
94% Crushed - ¾” - ½” Texture = 335



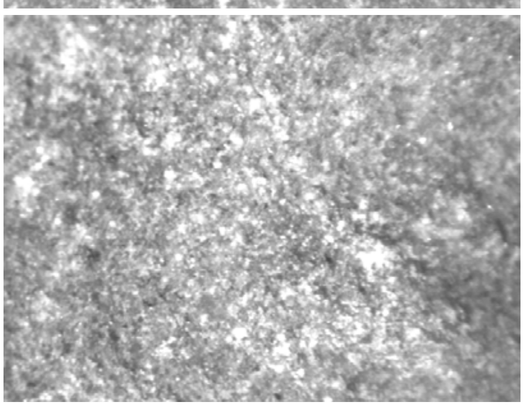
94% Crushed - ¾” - ½” Texture = 315

94% Crushed - ½” - 3/8” Angularity = 2303.3

94% Crushed - ½” - 3/8” Angularity = 2324.2



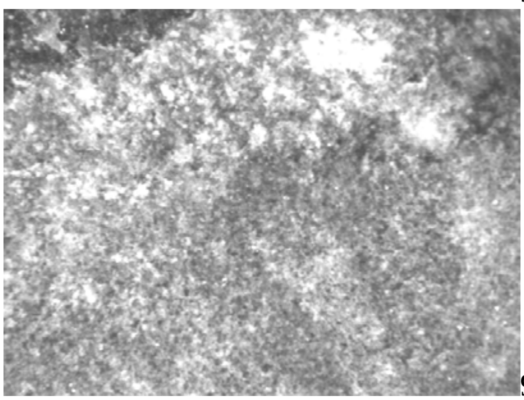
94% Crushed - ½” - 3/8” Texture = 270.5



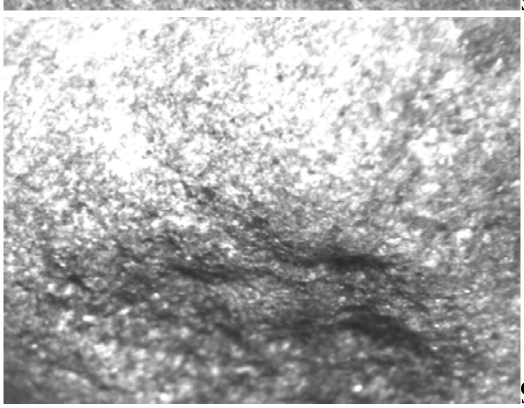
94% Crushed - ½” - 3/8” Texture = 288.5

97% Crushed - ¾” - ½” Angularity = 2206.48

97% Crushed - ¾” - ½” Angularity = 2266.69



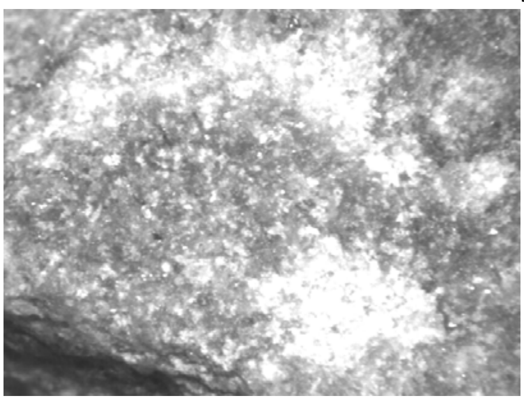
97% Crushed - ¾” - ½” Texture = 277



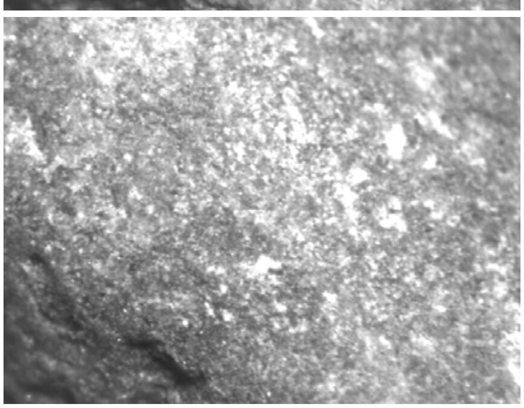
97% Crushed - ¾” - ½” Texture = 295

97% Crushed - ½” - 3/8” Angularity = 2074.12

97% Crushed - ½” - 3/8” Angularity = 2026.57



97% Crushed - ½” - 3/8” Texture = 272.5

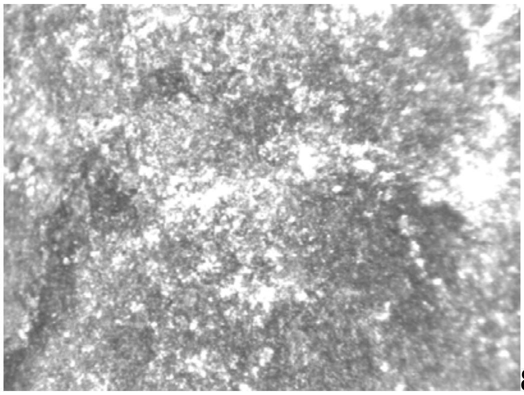


97% Crushed - ½” - 3/8” Texture = 278

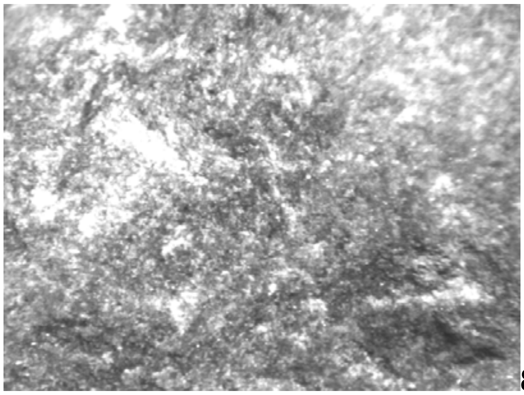
GRAVEL G3

88% Crushed - ¾” - ½” Angularity = 2421.34

88% Crushed - ¾” - ½” Angularity = 2056.03



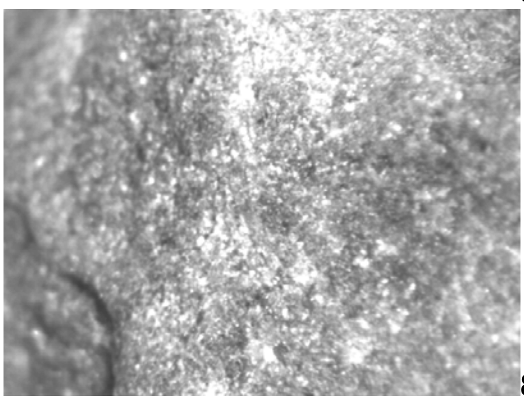
88% Crushed - ¾” - ½” Texture = 328.5



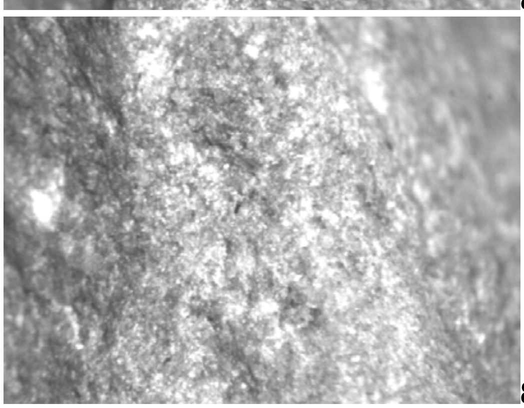
88% Crushed - ¾” - ½” Texture = 356

88% Crushed - ½” - 3/8” Angularity = 2561.89

88% Crushed - ½” - 3/8” Angularity = 2898.33



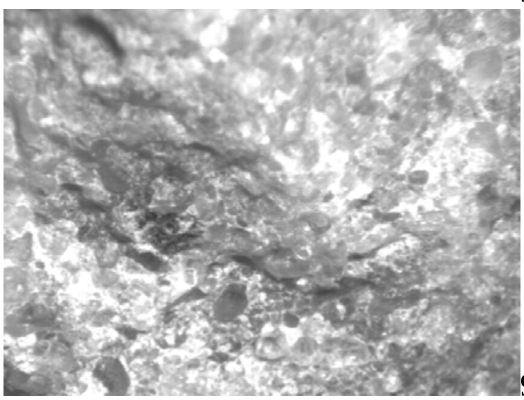
88% Crushed - ½” - 3/8” Texture = 239.5



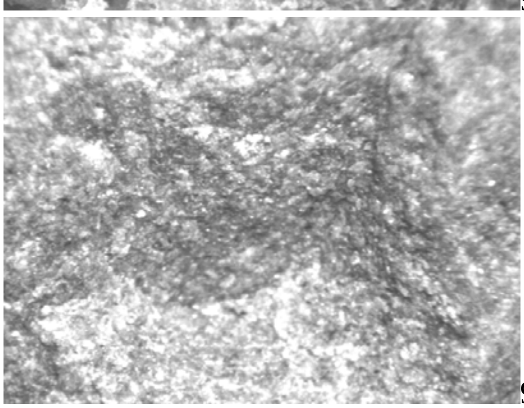
88% Crushed - ½” - 3/8” Texture = 291

91% Crushed - ¾” - ½” Angularity = 2795.30

91% Crushed - ¾” - ½” Angularity = 2013.02



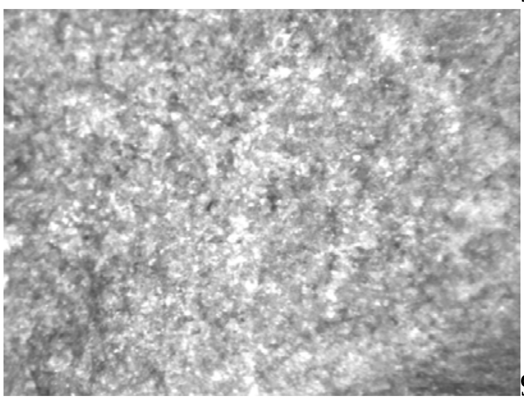
91% Crushed - ¾” - ½” Texture = 319.5



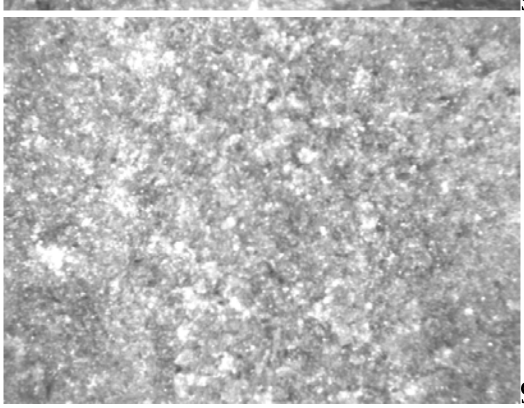
91% Crushed - ¾” - ½” Texture = 327.5

91% Crushed - ½” - 3/8” Angularity = 2141.94

91% Crushed - ½” - 3/8” Angularity = 2389.93



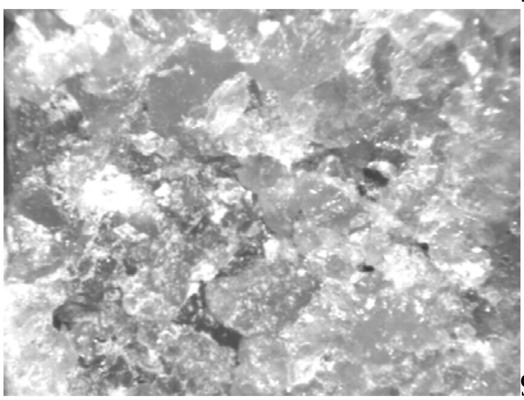
91% Crushed - ½” - 3/8” Texture = 263.5



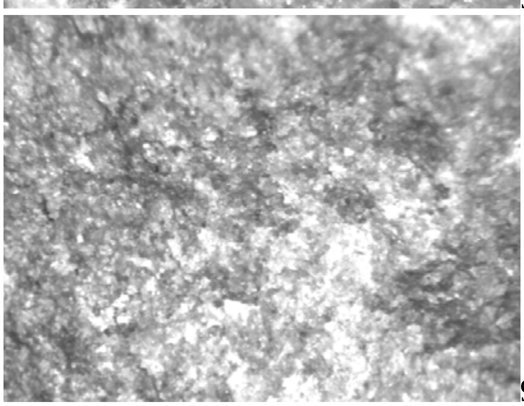
91% Crushed - ½” - 3/8” Texture = 250.5

94% Crushed - ¾” - ½” Angularity = 2590.85

94% Crushed - ¾” - ½” Angularity = 2252.47



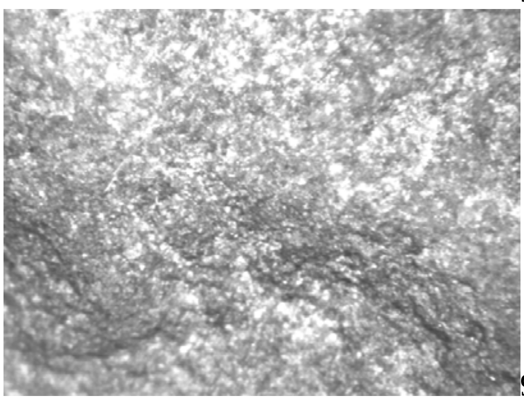
94% Crushed - ¾” - ½” Texture = 312



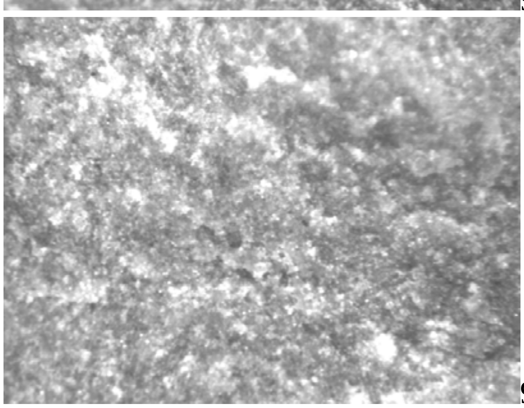
94% Crushed - ¾” - ½” Texture = 323

94% Crushed - ½” - 3/8” Angularity = 2107.41

94% Crushed - ½” - 3/8” Angularity = 2158.18



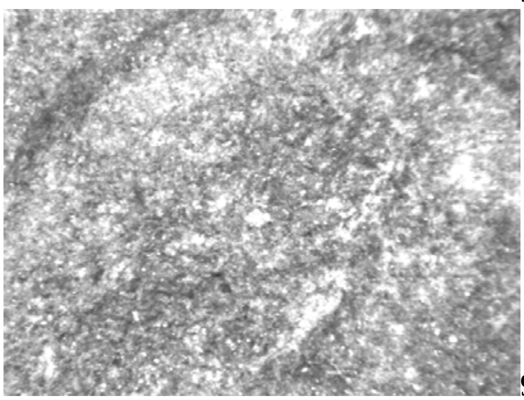
94% Crushed - ½” - 3/8” Texture = 304.5



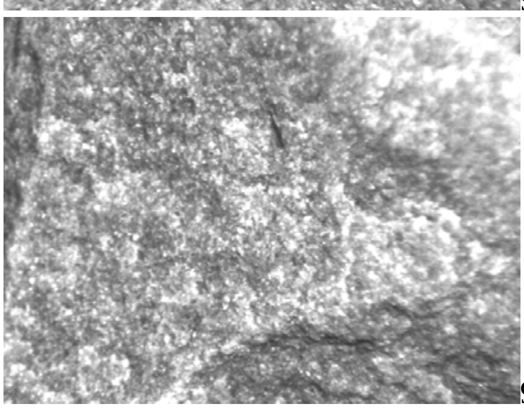
94% Crushed - ½” - 3/8” Texture = 271.5

97% Crushed - ¾” - ½” Angularity = 2299.98

97% Crushed - ¾” - ½” Angularity = 1940.77



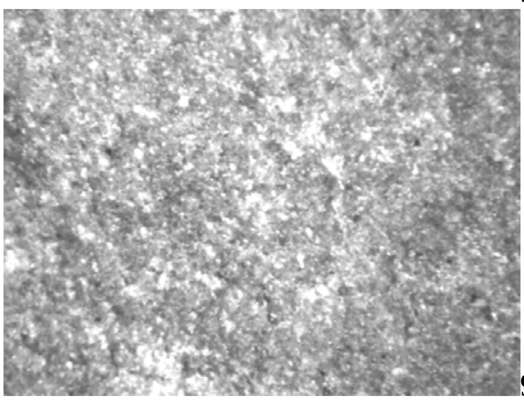
97% Crushed - ¾” - ½” Texture = 323.5



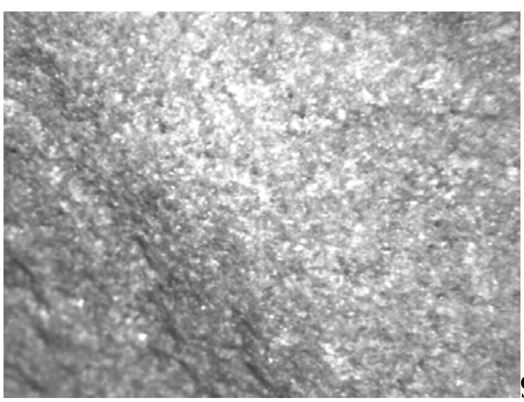
97% Crushed - ¾” - ½” Texture = 348

97% Crushed - ½” - 3/8” Angularity = 2085.40

97% Crushed - ½” - 3/8” Angularity = 2343.69



97% Crushed - ½” - 3/8” Texture = 293

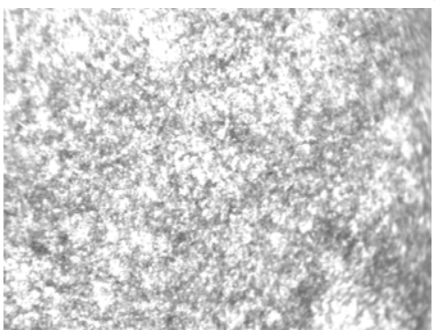


97% Crushed - ½” - 3/8” Texture = 226

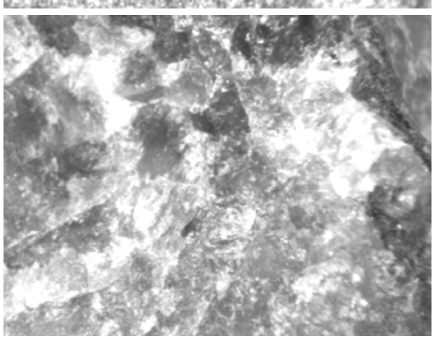
GRAVEL G4

88% Crushed - ¾” - ½” Angularity = 2228.34

88% Crushed - ¾” - ½” Angularity = 2245.61



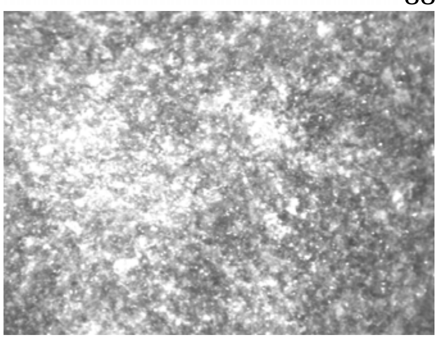
88% Crushed - ¾” - ½” Texture = 330.5



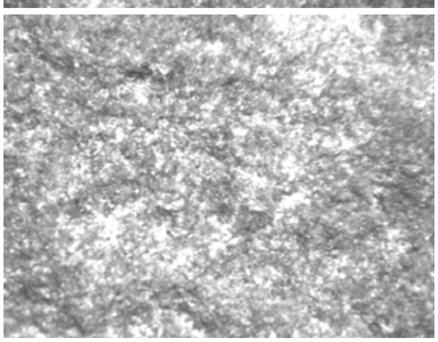
88% Crushed - ¾” - ½” Texture = 300.5

88% Crushed - ½” - 3/8” Angularity = 2303.15

88% Crushed - ½” - 3/8” Angularity = 2138.38



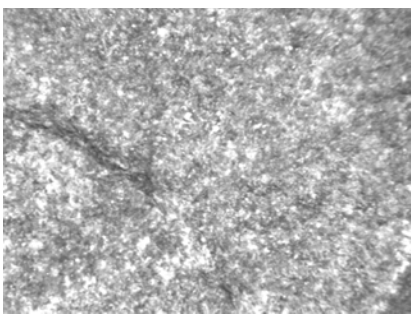
88% Crushed - ½” - 3/8” Texture = 327.5



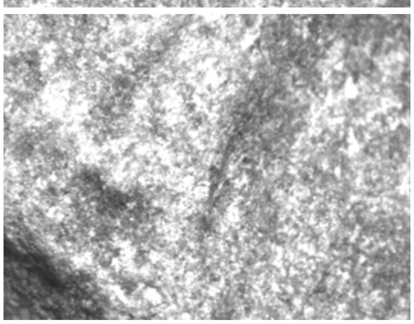
88% Crushed - ½” - 3/8” Texture = 323

91% Crushed - ¾” - ½” Angularity = 2279.12

91% Crushed - ¾” - ½” Angularity = 2160.81



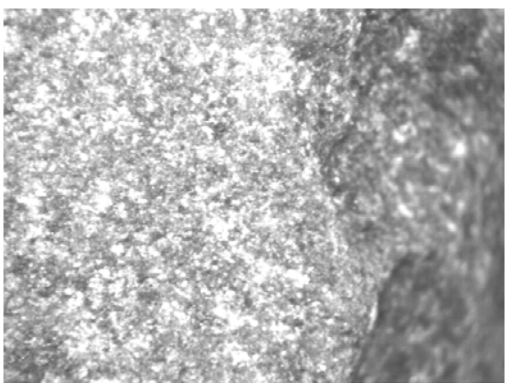
91% Crushed - ¾” - ½” Texture = 311.5



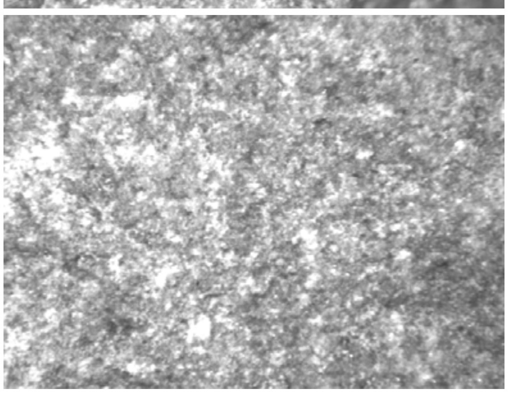
91% Crushed - ¾” - ½” Texture = 350

91% Crushed - ½” - 3/8” Angularity = 2030.04

91% Crushed - ½” - 3/8” Angularity = 2182.79



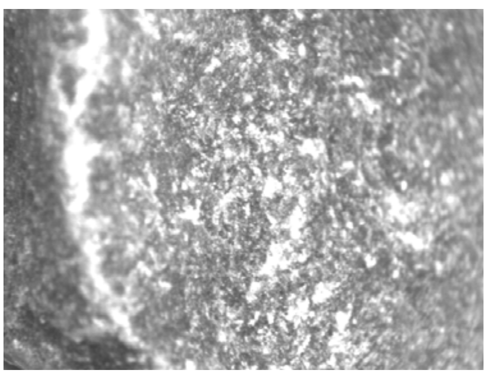
91% Crushed - ½” - 3/8” Texture = 315.5



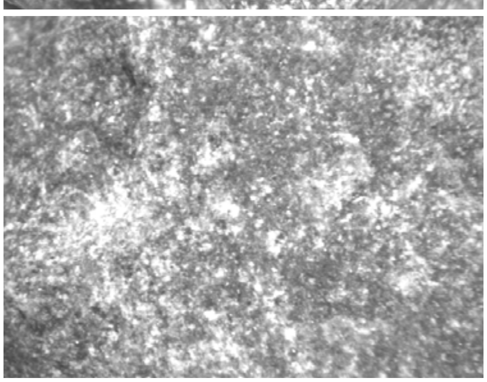
91% Crushed - ½” - 3/8” Texture = 322.5

94% Crushed - ¾” - ½” Angularity = 2419.27

94% Crushed - ¾” - ½” Angularity = 2265.26



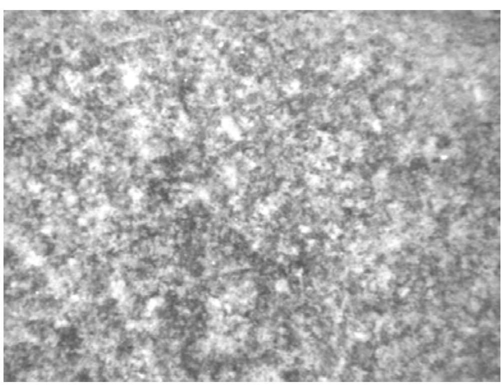
94% Crushed - ¾” - ½” Texture = 350.5



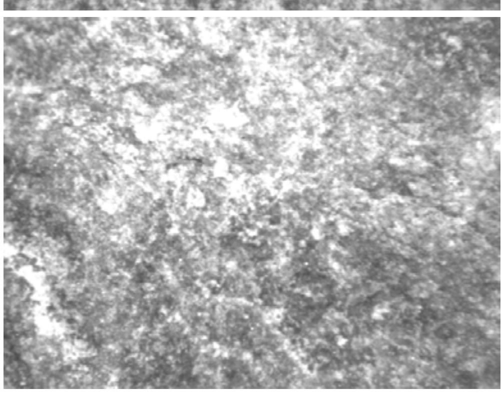
94% Crushed - ¾” - ½” Texture = 327

94% Crushed - ½” - 3/8” Angularity = 2047.43

94% Crushed - ½” - 3/8” Angularity = 2274.38



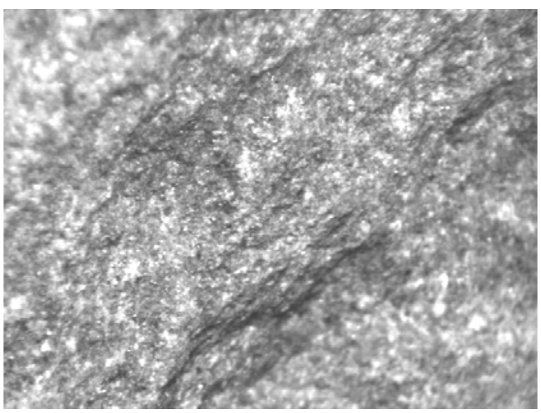
94% Crushed - ½” - 3/8” Texture = 317



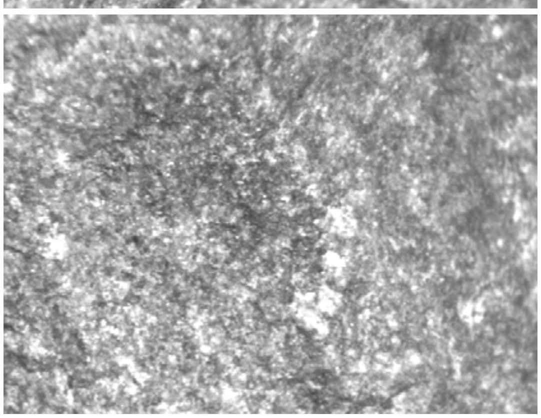
94% Crushed - ½” - 3/8” Texture = 309

97% Crushed - ¾” - ½” Angularity = 2195.06

97% Crushed - ¾” - ½” Angularity = 2257.69



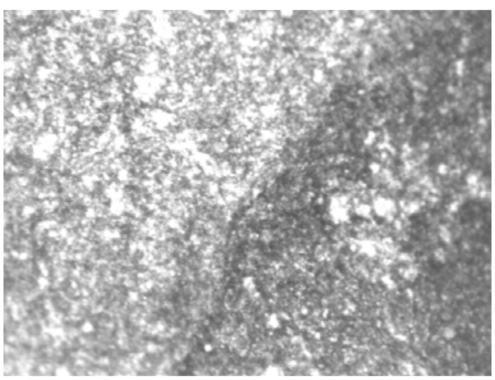
97% Crushed - ¾” - ½” Texture = 335.5



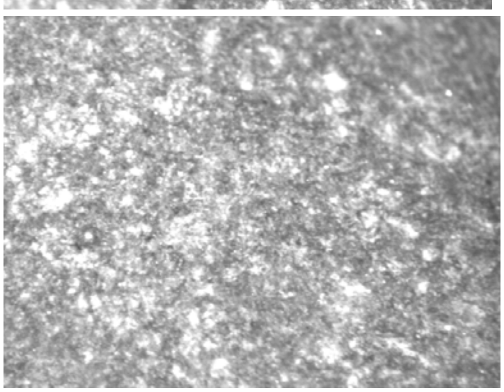
97% Crushed - ¾” - ½” Texture = 320.5

97% Crushed - ½” - 3/8” Angularity = 2307.57

97% Crushed - ½” - 3/8” Angularity = 2173.10



97% Crushed - ½” - 3/8” Texture = 344.5



97% Crushed - ½” - 3/8” Texture = 311.5