ANAYLYSIS OF GRANULAR MATERIALS IN PENNSYLVANIA HIGHWAYS

by

Kevin P. Hammer

B.S. in Civil Engineering, University of Pittsburgh, 1986

Submitted to the Graduate Faculty of

School of Engineering in partial fulfillment

of the requirements for the degree of

Master of Science

University of Pittsburgh

2005

UNIVERSITY OF PITTSBURGH

SCHOOL OF ENGINEERING

This thesis was presented

by

Kevin Phillip Hammer

It was defended on

July 7, 2005

and approved by

Jeen-Shang Lin, Associate Professor, Civil and Environmental Engineering Department

Calixto I. Garcia, Research Professor, Materials Science and Engineering Department

Thesis Advisor: Luis E. Vallejo, Professor, Civil and Environmental Engineering Department

ABSTRACT

ANAYLYSIS OF THE CRUSHING OF GRANULAR MATERIALS IN PENNSYLVANIA HIGHWAYS

Kevin Phillip Hammer, M.S.

University of Pittsburgh, 2005

The purpose of this study is to investigate the phenomena of crushing of granular materials forming part of actual flexible pavement systems. Crushing can occur in granular bases during three separate phases, installation and placement, compaction and through everyday vehicular traffic. Crushing in the granular base material could result in a breakdown in strength of the pavement section which could results in pavement cracking or pavement failure. Crushing of the pavement base can occur as a result of a combination of compressive and shear stresses that result from traffic loading and/or construction equipment. Very few field studies exist on the crushing of granular materials under asphalt pavements. This investigation will analyze the crushing of granular bases forming part of actual flexible pavements.

For this research, granular materials from actual roadways were analyzed using sieve analysis, point load testing and fractal analysis. In addition to the roadway base samples obtained from actual flexible pavements, “baseline” samples were obtained from stockpiles of roadway base typically used by the Pennsylvania Department of Transportation (PENNDOT).

A comparison of the actual a road base samples with those “baseline” samples indicated that some crushing took place in the granular bases. This crushing, however, was not severe. This conclusion is based on the results from the sieve and fractal analyses used in the research program.

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PREFACE

I would like thank Professor Vallejo for his guidance during the preparation and writing of this thesis and my friend and colleague, Mr. Sebastian Lobo-guerrero for his meaningful advice. I would also like to thank Mrs. Norma Hammer for her editing and typing of various sections of this paper.

1.0 INTRODUCTION

Crushing can occur in pavement bases during three separate phases, installation and placement, compaction and through everyday vehicular traffic. Crushing of the pavement base can occur as a result of a combination of compressive and shear stresses that result from traffic loading and/or construction equipment. In previous studies, it has been shown that crushing of granular materials occurs in a roadway base by means of theoretical simulations using mathematical models. Theoretical crushing can be predicted using estimated stresses and strains and applying them to a model using a finite-element analysis. An experimental numerical analysis was developed by Lobo-Guerrero and Vallejo which produced a result showing the mechanics of crushing in a pavement structure.[1]

The purpose of this proposed study is to study the phenomena of crushing of granular materials, specifically in a field environment such as the behavior under an asphalt pavement due to traffic loading. The objectives were as follows:

1. Determine if crushing occurred and try to quantify such as Breakage Factor, Permeability and Fractal analysis.
2. Perform laboratory testing to observe grain size distributions before and after loading and;
3. Determine how the roughness and shape of the particles have been affected.

For this research, field obtained roadway base samples were analyzed using sieve analysis, point load testing and fractal analysis of the granular particles. In addition to the roadway base samples obtained from actual field conditions, three “baseline” samples were obtained from stockpiles of roadway base typically used by the Pennsylvania Department of Transportation (PENNDOT).

There are a number of tests and subsequent parameters that can be applied to granular soils to evaluate the amount of crushing that occurs when subject to loading. These parameters will be used to help evaluate the amount of crushing that occurs in Pennsylvania highways based on actual field conditions. Crushing in the granular base material could result in a breakdown in strength of the pavement section which could result in pavement cracking or pavement failure.

2.0 THE CRUSHING OF GRANULAR MATERIALS

2.1 THE PROCESS OF CRUSHING

2.1.1 Factors Affecting Crushing

As a result of the traffic loads and resulting stresses, particle breakage occurs. Grain crushing is influenced by grain angularity, grain size, uniformity of gradation, low particle strength, high porosity and by stress level.[2,3]

When loaded, soil particles undergo a deformation through compressibility and fragmentation to varying degrees that are related to the strength and stress-strain factors of the particles. The amount of particle crushing in a soil element under stress depends on particle size distribution, particle shape, state of effective stress, effective stress path, void ratio, particle hardness and the presence and/or absence of water.[4]

2.1.2 Shear Stress and Compressibility

The most important factor affecting both shear strength and compressibility is the fragmentation that occurs during changes in the state of stress. DeBeers performed one-dimensional compression testing on sands and determined that particle crushing in sands was negligible below 9.8 MPa (1,420 psi) and decreased drastically with increases in stress. He determined that crushing became substantial at stresses of about 15 MPA (2,100 psi) however, decreased progressively with increases in stress at pressures above 34 MPA (20,000 psi) or less. DeBeers (1963). However, DeBeers tests did not include stresses above 20,000 psi. Additional tests were conducted by Esterle as discussed by Hagerty, Hite and Ullrich, extending the stress range to 689 (100,000 psi). Contrary to Hite's tests, Esterle's testing concluded that there was an increase in particle crushing with increasing stress as measured to 689 MPA (100,000 psi). [5]

2.1.3 Previous Studies to Quantify Breakage

The amount of particle breakage that occurs during loading of a soil sample is defined by the particle size distribution curves measured before and after loading. Marsal and Lee and Farhoomand proposed breakage measure to quantify the amount of particle breakage that occurs in a particular sample. Marsal's breakage measure was based on an increase in percent passing a single sieve. Lee and Farhoomand proposed using a particle size scale using the ratio D_{15i}/D_{15f} in which D_{15i} = the diameter for which the original sample is finer and D_{15f}= the diameter for which 15% of the loaded sample is finer. Using a log scale, the breakage factor can be determined by the horizontal distance between particle size distribution curves at 15% finer. [6,7]

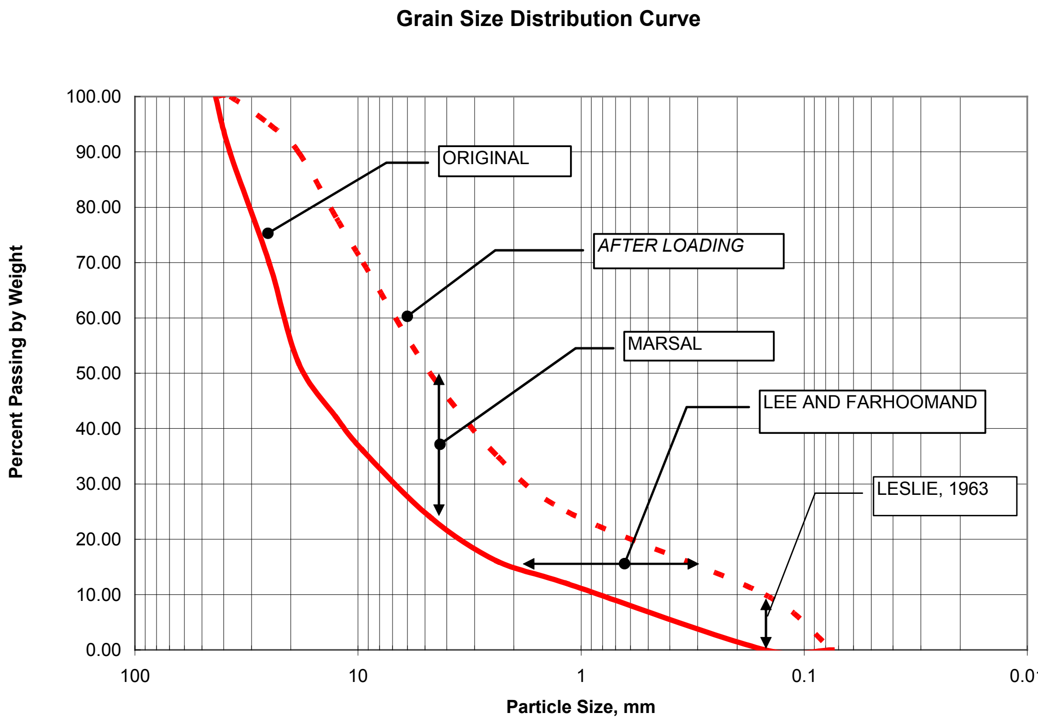

Figure 1 Typical Grain Size Distribution Curves Before and After Crushing

The potential for breakage of soil particle increases with an increase in size since the probability of having a defect in a particle increases with size. In addition, there are fewer defects in smaller particles that have been crushed and hence, less potential for additional breakage. Hardin proposed a definition and equation for relative breakage using the sieves analyses for the particle before and after crushing occurs. However using a silt particle with a maximum size of 0.074 mm, he was able to show that potential breakage of a particle is independent of grain size distribution. He also further pointed out that water greatly increases the crushability of sand particles.[8] Hardin proposed a breakage potential factor determined by using the grain size distribution curve of a particular sample. The breakage potential was determined by calculating the area between the grain size distribution curve and particles greater than a constant 0.074 mm. He further defined the relative breakage as the ratio of total breakage, Bt and breakage potential, Bp. Bt is measured as the area between the grain size distribution curves before and after loading. Hardin also proposed equations for the effects of void ratio, stress path and particle shape and hardness based on the use of the relative breakage and breakage potential measurements.

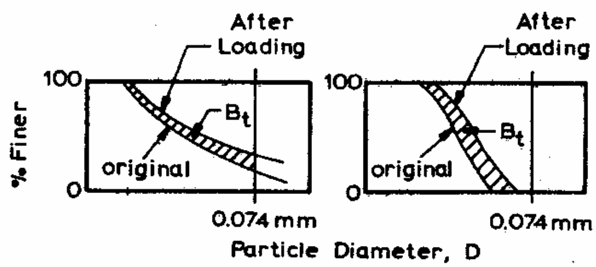

Figure 2 Hardin's Particle Breakage Factor

Lee and Farhoomand further theorized that there is an approximate linear relationship between Bt and Bp which suggests that the relative breakage is independent of particle size distribution after analyzing several grain size curves when the particle size is the only variable.[9]

2.1.4 The Effect of Particle Shape

The effect of particle shapes on crushing was examined during Hite's test and it was determined that the rough fractured glass particles exhibited a higher degree of crushing than the smoother Ottawa sand particles. Angular, rough-graded granular materials were crushed more extensively at a given stress, than were materials with smooth rounded grains.[10] Hite theorized that the more extensive crushing of angular particles was probably caused by a greater incidence of eccentric loading compared to that of the rounded, smoother particles. Hite's testing determined that particle crushing facilitates rearrangement of both fractured and unbroken particles. An increase in stress above the crushing stress results in additional fracturing and more extensive rearrangement of particles. Since the mean particle size is reduced and at the same time results in a reduction of the void ratio, the incident of article contact decreases and hence, a reduction in average interparticle stress. The reduction in stress leads to less additional crushing. A review of the one-dimensional compression tests showed that particle crushing increases greatly with increasing particle angularity and with increase in median particle size. Hite's testing also showed significant particle crushing at stress levels above 138 MPa (20,000 psi).[11]

Lee and Farhoomand used triaxial testing to illustrate that the higher the shear stresses in a particle, the greater the amount of particle crushing. They tested both angular and sub rounded particles and also determined that crushing was greater in the angular particles. They developed a ratio in which they compared the initial 15% size of the soil before testing and the 15% size after testing which was called the "relative crushing". It was determined that there was no increase in crushing as a result of an increase in principal stress, however, noticeable crushing was observed when there was an increase in shear stress. Their testing also theorized that the finer the soil, the less the amount of crushing occurs. [12,13]

Previous triaxial compression testing on granular soils was performed by Hall and Gordo and Marsal.[14] Hall and Gordon's results indicated that there was a considerable amount of crushing that occurred during the shearing stage of the test and that more crushing occurred for uniformly graded soil than for well-graded.[15] The particle sizes used during their testing was up to 3 inches in diameter. Marsal's results indicated that coarse material compressed and crushed more than fine material. Lee and Farhoomand determined that from previous investigations performed on granular materials that granular soils will compress and particles will crush as the stress on the soil increases and that the compression is not instantaneous but continues over a long period of time.[16]

Lee and Farhoomand investigated the effect of grain size, grain shape, distribution of grain sizes, stress level and Kc ratio on compressibility and particle crushing using laboratory triaxial tests on subrounded and angular particle shapes. The results of his test showed that the angular soil is considerably more compressive than the subrounded soil based on visual observations of photographs of the resulting particles and review of the grain size distribution curves. According to Lee and Farhoomand, one of the most important factors influencing the crushing of a mass of granular materials is the crushing resistance of the grains.[17] When a granular mass is subjected to a compressive load, the particles resist the load.

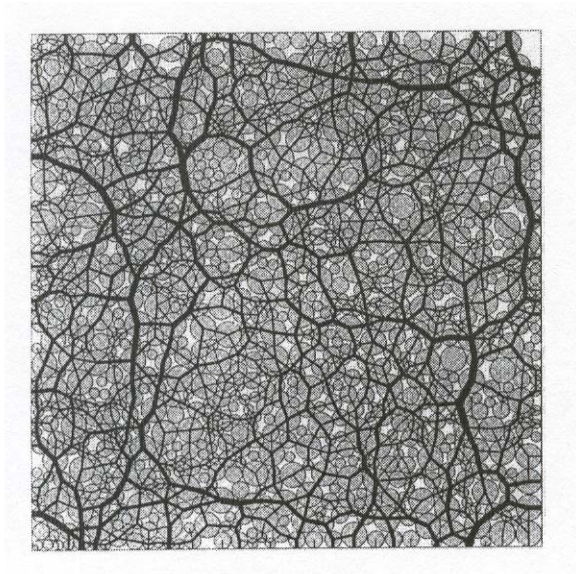

Figure 3 Force Chains in two-dimensional static assembly of discs in a horizontal container with three fixed walls and one piston, on which a fixed vertical force is applied

They were able to further verify that granular soils undergo compression when subject to applied stresses which directly relates to breakage. They also determined that uniform soils crush more than well graded soils and that the increase in particle crushing constantly decreases indefinitely.

Particles do not equally share in the bearing of loads. Some particles carry more load than others and in addition, some particles can actually be removed without affecting the mechanical equilibrium of the packing. The particles with highly loaded contacts are usually aligned in chains. Crushing starts when these highly loaded particles fail and break into smaller pieces that fall into the voids of the original material. These load changes in intensity and direction as the crushing develops in the material. On crushing, fines are produced and the grain size distribution curve becomes less steep. Consequently, with continuing crushing, the soil becomes less permeable and more resistant to crushing. Therefore, grain size distribution is a suitable measure of the extent of crushing. [18]

As mentioned before, compression induced crushing of granular material (as in the unbound granular base under an asphalt pavement) causes a decrease in volume of the original assembly of grains (settlement if the grains are laterally confined.). This decrease in volume is the result of the breakage of some of the grains. The grains that break could consist of pieces that are large enough to occupy the space of the original particles, or they can break into multiple small pieces. If the breakage produces few large pieces, this type of breakage will not cause a substantial volume decrease of the original granular structure. On the other hand, if the grains that break produce multiple small pieces that are small enough to migrate to the adjacent voids in the granular structure, then the decrease in volume of the original structure will be substantial. The compressive pressure acting on the small broken grains aids in their migration inside the granular mass. Also, the number of broken grains will be a function of the level of compressive force acting on the granular assembly. The larger this compressive force, the larger will be the number of broken grains.[19]

Lade et al. (1996) found that if a uniform granular material is crushed, the resulting grain size distribution approaches that of a well graded soil for very large compressive loads. Bolton (1999) established that the grain size distribution of a granular assembly that has been crushed under large compressive loads is a fractal distribution.[20] A well-graded particle distribution or a fractal distribution represents a granular structure that is made of grains of all sizes including the original unbroken grains. These original large grains did not break based on the fact that with more small size particles surrounding them, the average contact stress acting on these large grains tends to decrease. However, before the granular structure reaches a well graded or a fractal particle size distribution, the granular structure will experience gradual changes in particle sizes depending on the magnitude of the compressive load applied to it.[21]

3.0 ESTIMATED STRESSES BELOW PAVEMENT SURFACE

3.1 TYPICAL PAVEMENT CONSTRUCTION

Most flexible pavements installed in highways across the State of Pennsylvania consist of a granular base on top of a compacted subgrade, with an asphalt overlay placed on top of the granular base. The combination of the asphalt layers and granular layer provides a structural bond which makes it capable of withstanding repeated loads such as those which occur from vehicular traffic. A geotextile fabric is sometimes used on top of the existing subgrade for additional strength where the subgrade may be soft.

3.1.1 Pavement Thickness

The thickness of the granular base typically ranges from 6-inches to 12-inches, depending on the anticipated traffic loads. The asphalt layer on top of the granular base typically ranges in thickness from 6-inches to 12- inches. In some instances, based on field observations, asphalt pavement thickness could be as thick as 18-inches. The granular serves two main purposes. The first purpose is to provide the structural strength. For the most part, it is cheaper to install a granular material in a pavement section than an asphalt material. The gradation of the granular base is typically well-graded so that good compaction can be obtained resulting in minimizing voids.

3.1.2 Drainage Medium

The second purpose is as a drainage medium. The porous granular material allows water to flow through and minimizes the potential damage that can occur due to the freeze thaw cycle.

3.2 ESTIMATING STRESS BELOW PAVEMENT SURFACE

3.2.1 Design Loads

Granular materials underlying the asphalt layer, are subjected to both static and dynamic loads resulting from vehicular traffic. Figure 4 provides a typical pavement cross-section illustrating each layer of asphalt and base and illustrating the transfer of stresses.

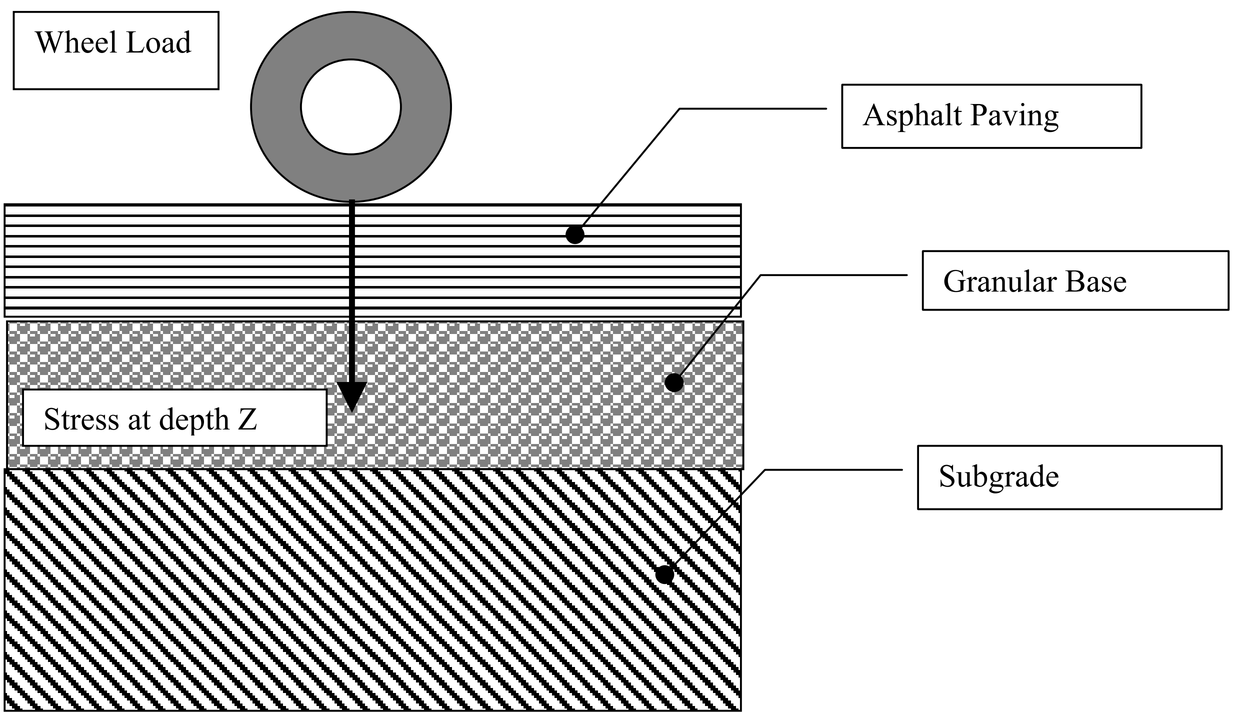

Figure 4 Typical Pavement Section

Tractor trailer axle loads could be as high as 40,000 lbs or 20 kips per wheel.[22] The above Figure 4 provides the general composition of the pavement structure. The actual pavement and base thickness will vary depending on the proposed use and design loads of the roadway.

3.2.2 Stress Analysis

Although estimating the stress level in a granular base below an asphalt layer is complex, it can be assumed for the purpose of this study that the granular material base combined with the asphalt overlay is elastic and homogenous. Using this assumption, the stress at any given point below grade can be computed using the Boussinesq equation under a static load.

Considering an asphalt pavement thickness of 15-inches, the computed stress at the center of the granular base material is 324 kPa. Stresses at various depths are summarized in Table 1.

Table 1 Vertical Stresses in Granular Subgrades

Depth (in)	12	14	16	18	20	22	24
Stress (kPa)	355	344	335	329	324	320	317

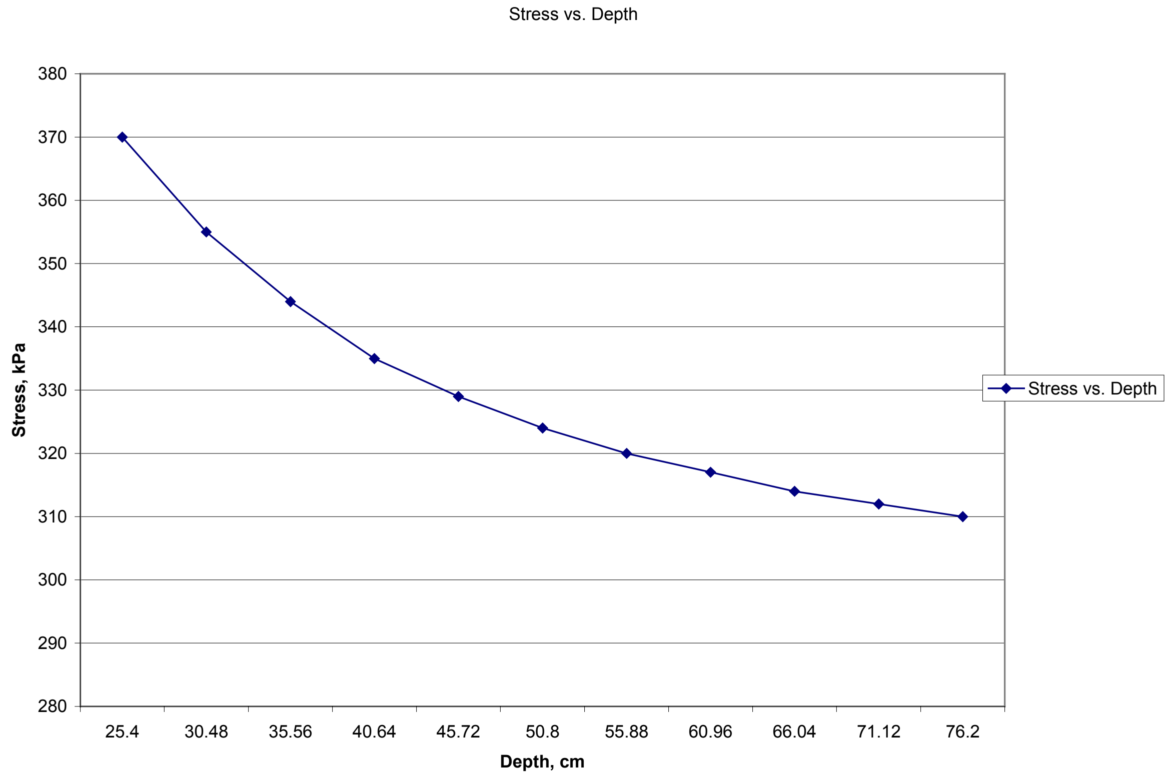

Figure 5 Estimated Stress vs Depth Below Pavement Surface

Induced stresses in a pavement structure can be further illustrated in Figure 6. Before the wheel reaches a point D on the granular base, this point is subjected to a combination of normal and shear stresses. When the wheel is directly on top of point D, the granular base is subjected to a normal stress only. After the wheel passes point D, the granular base at this point is subjected again to a combination of normal and shear stresses.

However, when the wheel load is beyond point D, the shear stress changes direction. The analysis presented in Figure 6 is for two-dimensional conditions only.

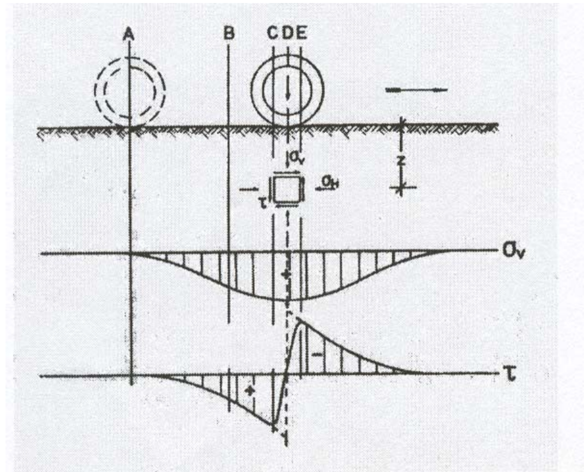

Figure 6 Stresses induced in a granular base by a moving wheel[23]

3.3 TYPICAL PAVEMENT DESIGN

3.3.1 Pavement Design and Granular Base

Although pavement design is beyond the scope of this research, it is important to understand the guidelines used for design of flexible pavements so that loads and stress placed on the granular base can be estimated with some degree of certainty. It is also important to understand the role of granular base material in pavement design and how it is affected by crushing.

3.3.2 Development of the Design Procedure

The Pennsylvania Department of Transportation Pavement Design Method as presented in PENNDOT Publication 13, Design Manual Part 2, Chapter 11, is based on pavement design methods established for both rigid and flexible pavements.[24] The PENNDOT method was established as a result of AASHTO Road tests conducted at Ottawa, Illinois from 1956 to 1960.

The American Association of State Highway and Transportation Officials (AASHTO) has developed an Interim Guide for Design of Pavement Structures. Although the AAHSTO method is empirical, it is time proven and has gained national acceptance. The AASHTO method uses subgrade strength, expected traffic intensities, desired pavement life, pavement drainage conditions and climate to develop an accurate model for predicting pavement behavior.[25,26]

3.3.3 Subgrade Characterization

The AASHTO method requires that a Soil Support Value (SSV) be determined. The SSV can be estimated using approximate correlations from several commonly used measurements such as, vane shear tests or a known CBR value. In addition, a soft subgrade can be strengthened by additional compaction and proof rolling and also by adding a synthetic geotextile. However, even if there is a low CBR value, the pavement section can be increased to minimize the stress on the subgrade and more evenly distribute the load. AASHTO has also developed a structural number to indicate the combined structural capacity of all pavement layers above the subgrade.[27]

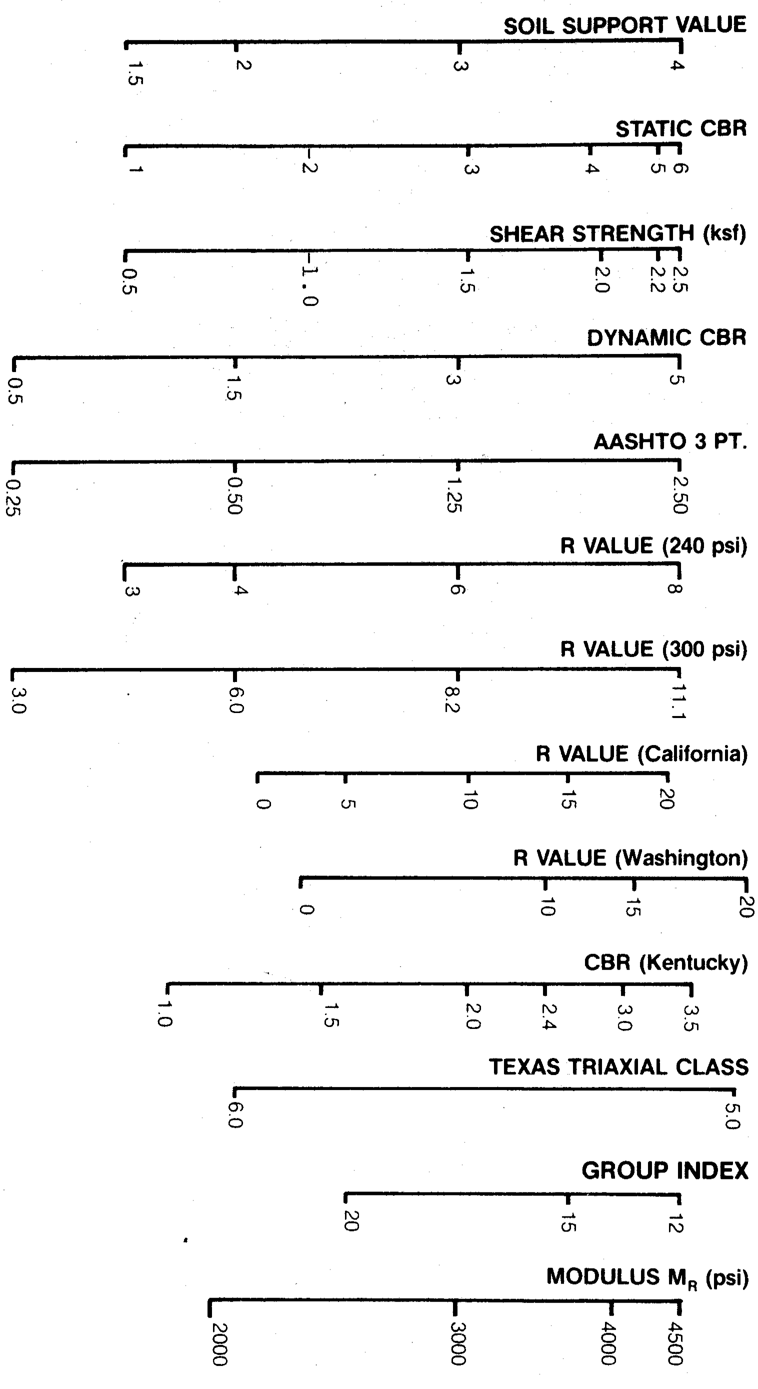

Figure 7 Typical Relationship Between Several Measures of Soil Strength and Soil Support Value[28]

Figure 7 above provides guidelines for estimating the subgrade strength for design purposes and should be used with appropriate testing by a soils engineer.

3.3.4 Traffic Intensities

Two essential design parameters used in pavement design are traffic intensity and/or traffic volume and loads expected over the design life of the pavement. Typically, expected volumes are determined based on estimated and actual traffic counts determined through a detailed traffic study of the proposed route. The second parameter used in pavement design is expected wheel load. Typically, the heaviest expected wheel load expected over a particular roadway section tends to govern the design of the section so that excessive stress levels will not cause excessive permanent deflection. The design of the pavement section must result in a section that would limit the stress on the existing subgrade. AASHTO has developed equivalence factors for use in the design of flexible pavements. The equivalence factors allow a designer to convert varying weights of expected vehicle traffic over a particular section of roadway to an equivalent 18-kip single-axle load. Tractor trailer loads could extend up to 40,000 lbs per axle. However, the truck load is converted to an 18-kip single axle load as part of the calculation.[29]

Single Axles, p_t = 2.0

Axle Load	Weighted Structural Number, SN					
Kips	1	2	3	4	5	6
2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	0.002	0.003	0.002	0.002	0.002	0.002
6	0.01	0.01	0.01	0.01	0.01	0.01
8	0.03	0.04	0.04	0.03	0.03	0.03
10	0.08	0.08	0.09	0.08	0.08	0.08
12	0.16	0.18	0.19	0.18	0.17	0.17
14	0.32	0.34	0.35	0.35	0.34	0.33
16	0.59	0.60	0.61	0.61	0.60	0.60
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.59	1.56	1.55	1.57	1.60

Figure 8 18 kip single axle load equivalence factors[30]

3.3.5 Pavement Drainage

The AASHTO method assumes that the strength of the subgrade and the base will remain fairly constant over the design life of the pavement. Therefore, the pavement must be designed so that surface water will flow toward the shoulder area so it can be collected by inlets. In addition, high groundwater tables must be controlled through the use of under drains or other suitable methods. The granular base must also be graded such that water can drain effectively to minimize potential freeze/thaw damage.[31]

3.3.6 Climatic Conditions

AASHTO also uses a "Regional Factor" for use in the pavement design. A Regional Factor is selected based on the local climatic conditions such as amount of precipitation, location of water table, presence of adequate drainage, duration of freezing cycle and frost depth. A Regional Factor of 1.5 is typically used for Pennsylvania.[32] U.S. annual mean precipitation is shown in Figure 9 below.

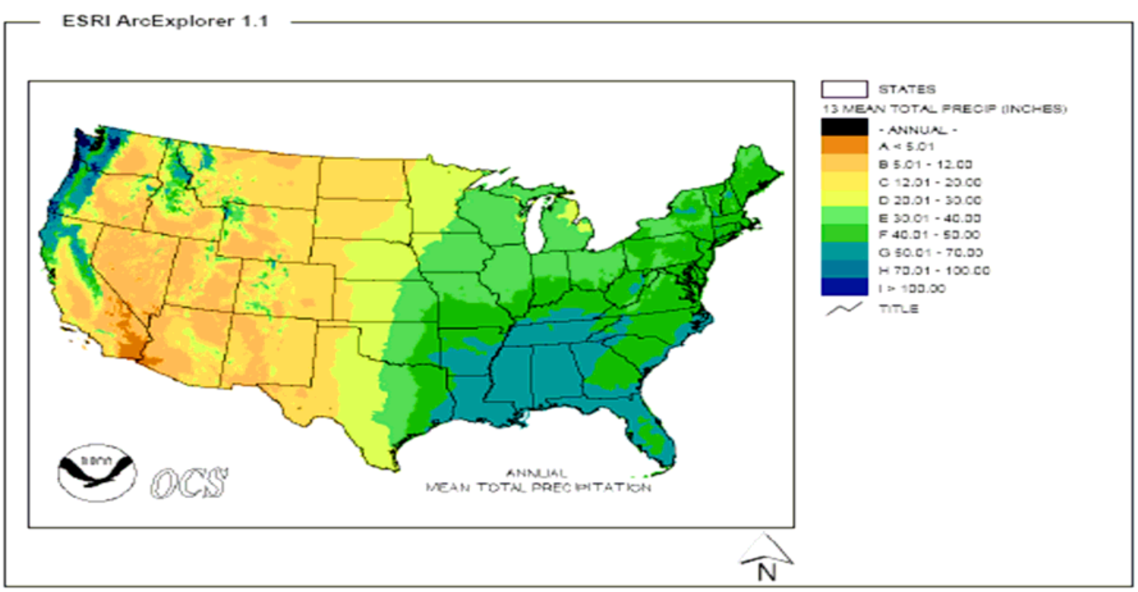

Figure 9 United States Annual Mean Total Precipitation

3.3.7 Selection of Granular Base Material

In designing a pavement section, standard methods of aggregate selection is important to provide the most effective base. Aggregates with gradations with a large fraction of coarse material are difficult to compact and result in a rough riding surface. Aggregates with a large percentage of fines although are easier to compact, they tend to lose bearing capacity when wet. Therefore, aggregates with a balance with the right amount of fines and coarse fraction, such as open to well-graded, should be used since they are more easily compacted, less moisture sensitive and result in a smoother riding surface. Aggregates with no more than 8% of fines passing the No. 200 sieve are preferable. Based on this analogy, if crushing occurs in the base, the pavement section is compromised.[33]

Most of the time, a geotextile material is used below the granular base to provide extra strength.

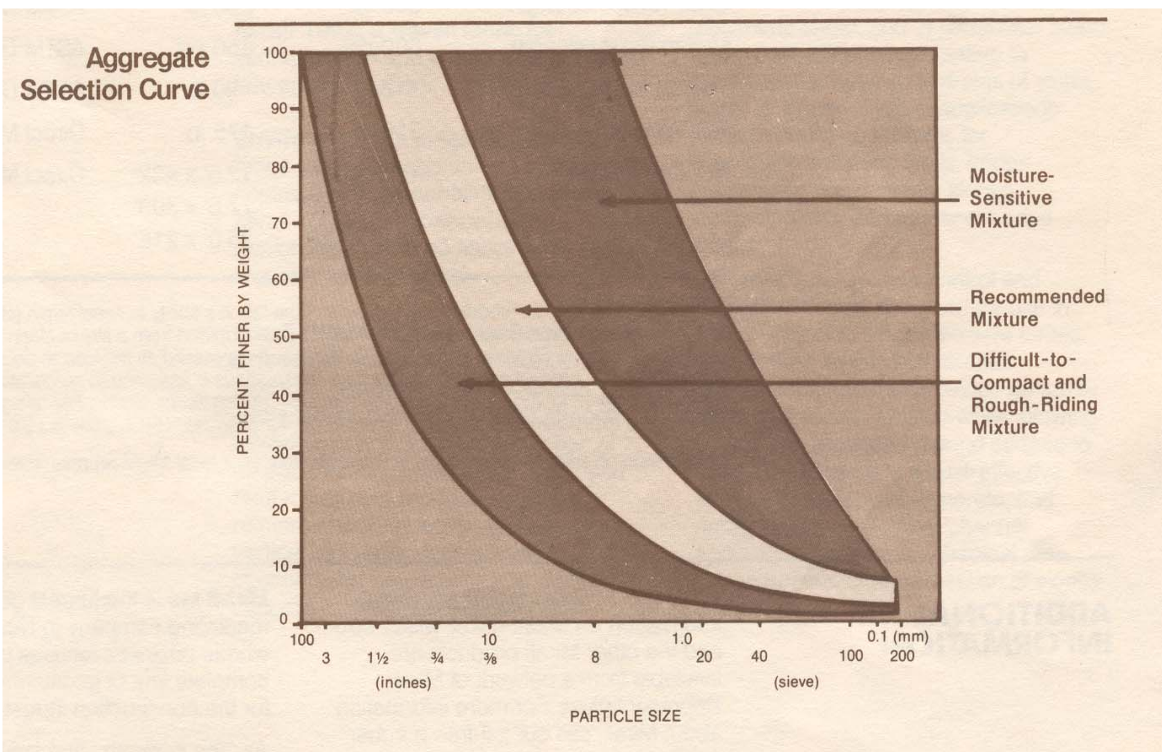

Figure 10 Recommended Aggregate Gradation for Pavement Design

3.3.8 Estimating The Structural Number, SN

After the designer has chosen the appropriate Soil Support Value, SSV, Equivalent 18 kip single-axle load and Regional Factor, a nomograph is used to determine the required Structural Number, SN.

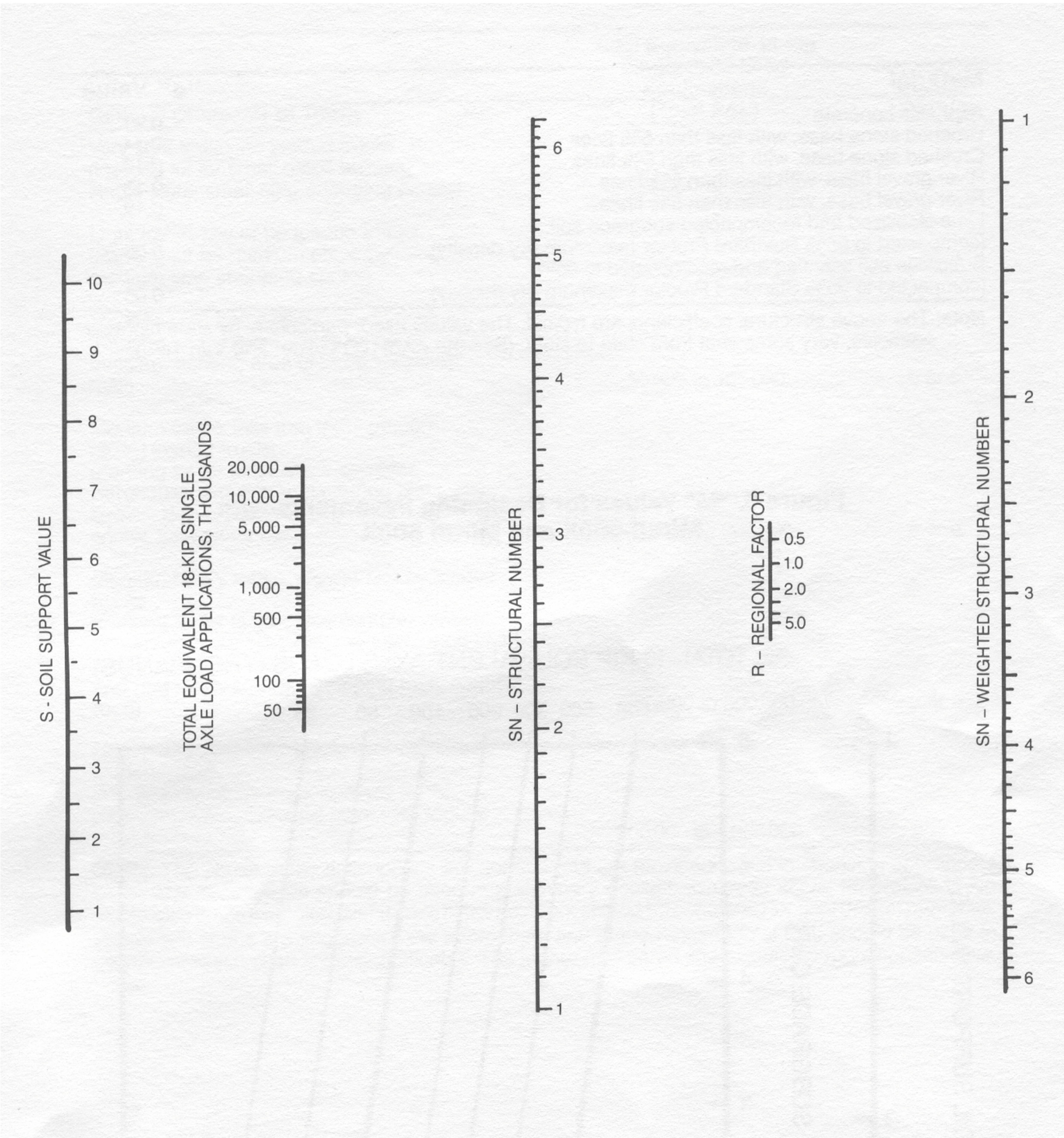

Figure 11 Flexible Pavement Design Nomograph[34]

4.0 FIELD SAMPLE CHARACTERIZATION

4.1 STANDARD PENNDOT BASE 2A GRADATION

The following chart indicates the minimum gradation requirements of granular base.

Table 2 PENNDOT Type 2A Gradation[35]

AASHTO No.				Percent Passing							
	2”	1 ½”	1”	¾”	½”	3/8”	#4	#8	#16	#100	#200
2A	100			52-100		36-70	24-50	16-38	10-30		0-10

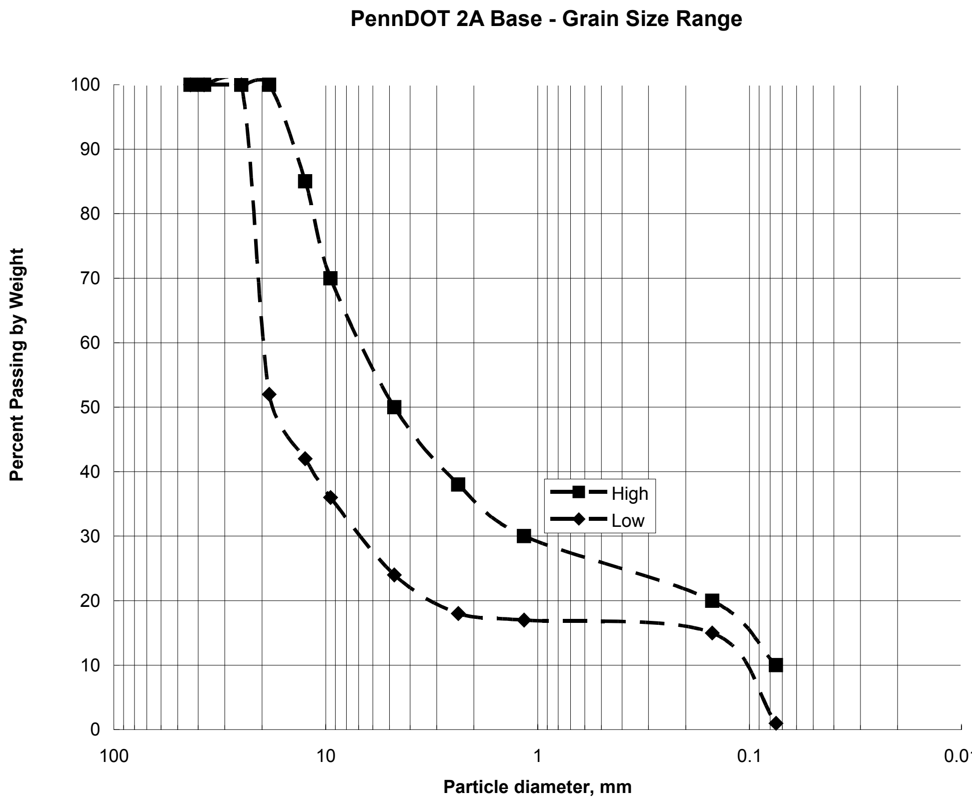

Figure 12 PENNDOT 2A Base – Grain Size Range

As shown in the above Grain Size Distribution chart, there is a range of particle sizes that can be used to form a granular base. Although true before and after conditions for each roadway pavement site were not obtained, a "baseline" borrow sample was used using granular material in an unused state to help simulate the gradation of a typical roadway base prior to installation. The above Table indicates the required gradation of a typical pavement base material used by the Pennsylvania Department of Transportation. The gradation of the field sample obtained from the borrow area, closely matches the required gradation. It is the purpose of this study to evaluate the relative changes in gradation and particle roughness measured by sieve analysis and fractal dimension to provide some insight into particle crushing.

4.2 OBTAINING THE FIELD SAMPLES

4.2.1 Field Samples

Field samples of roadway base material were obtained from two separate site locations, Campbell's Run Road and Worthington Avenue, both in Pittsburgh, Pennsylvania. Each sample was a grab sample collected in a 5-gallon bucket.

4.2.2 Campbell's Run Rd Samples S-1 and S-6

Campbell's Run Road is a two-lane highway and is classified as a collector road based on the design hourly volume (DHV) traffic as defined in the *Pennsylvania Department of Transportation, Design Manual, Part 3*.[36] Collector roads could typically have a DHV of 400 or higher. It is important to note the DHV for each roadway to illustrate the volume of traffic and corresponding loads on the pavement. A location map is shown in Figure 13.

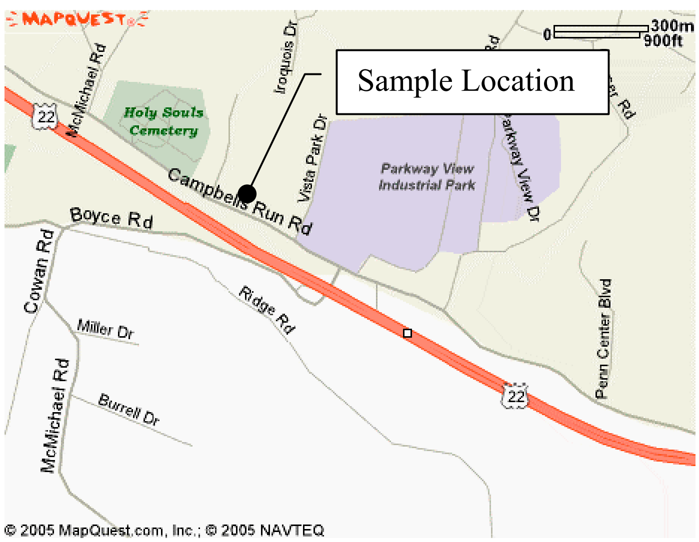

Figure 13 Location of Campbells Run Road in Pittsburgh, PA

The observed pavement section consisted of an 18-inch thick layer of asphalt on top of a minimum 10-inch layer of granular base. The pavement width at the Campbell's Run Road site was measured to be 22 feet with a 4 foot paved shoulder.

Figure 14 Campbells Run Road – Pittsburgh, PA

Photographs of the pavement section are shown in Figure 15. Two samples were obtained from the site and subject to laboratory testing. The moisture content of the samples was relatively dry.

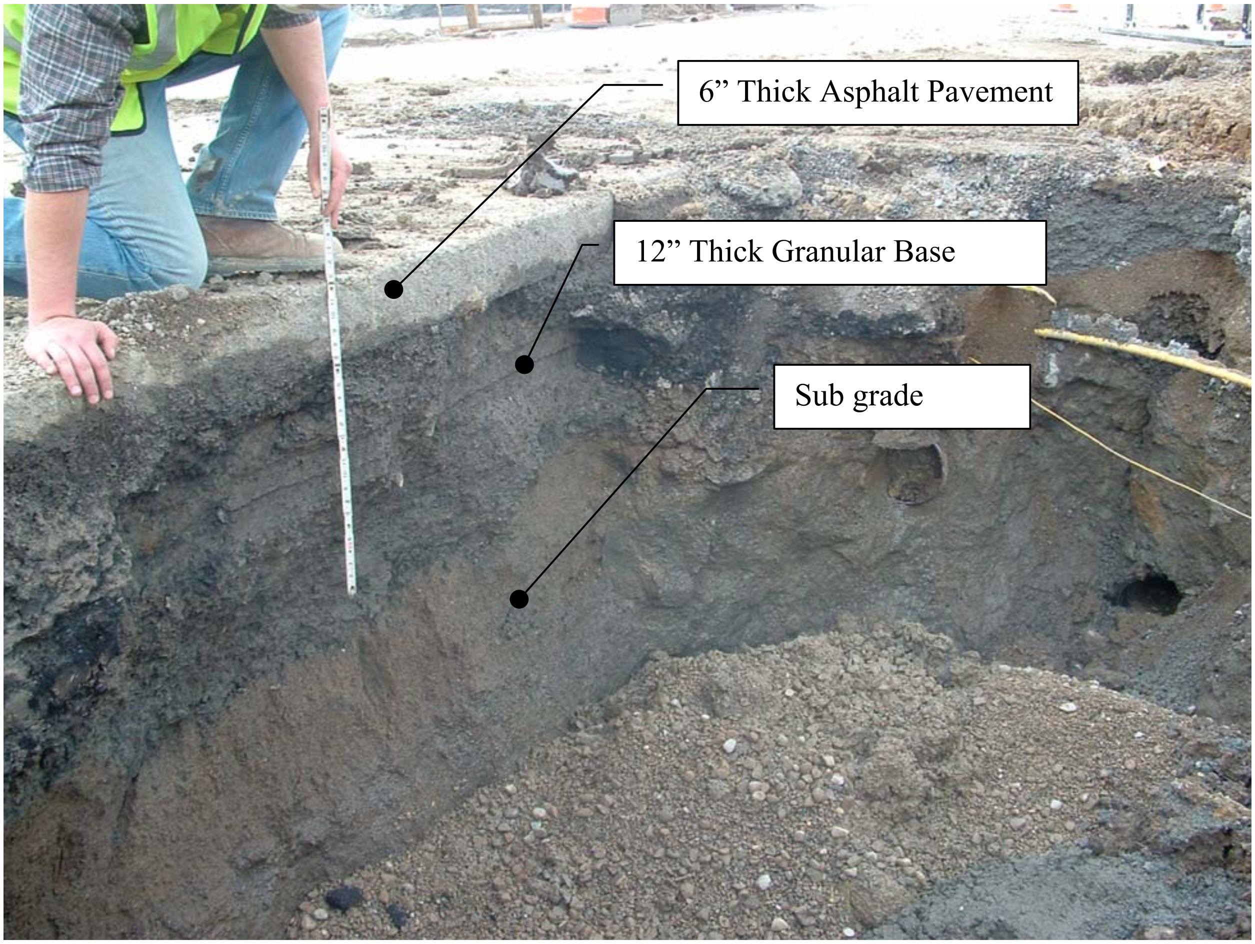

Figure 15 Campbells Run Road Sample Locations

Campbells Run Road was undergoing rehabilitation when the samples were taken. The first sample was obtained from a utility trench cut through the existing pavement cart way. The second sample was collected from a cross-cut excavated through the traveled cart way.

The sample was collected by hand using a shovel after the asphalt surface had been removed.

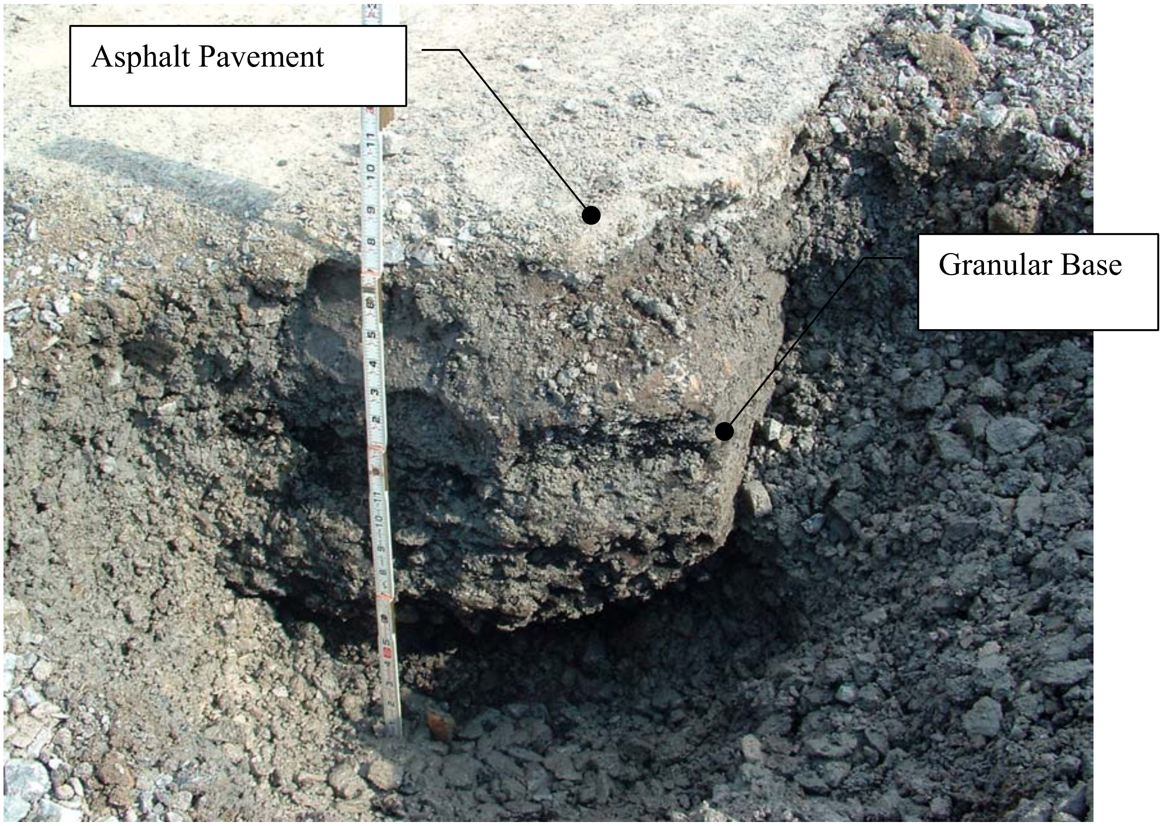

Figure 16 Sample Location at Campbells Run Road

A photograph of the sample location S-5 is shown in Figure 16 above.

4.2.3 Worthington Avenue Samples S-3 and S-4

The second site location used for sampling was Worthington Avenue located in Jefferson Borough, Allegheny County Pennsylvania (near Pittsburgh). Worthington Avenue is also classified as a collector road. Based on information obtained from the Allegheny County Department of Public Works, the pavement was constructed in 1987 as part of a rehabilitation project.[37] Worthington Avenue is a two-lane highway with a pavement width measured at 22 feet with a 4 foot paved shoulder. A site location map is shown in Figure 17.

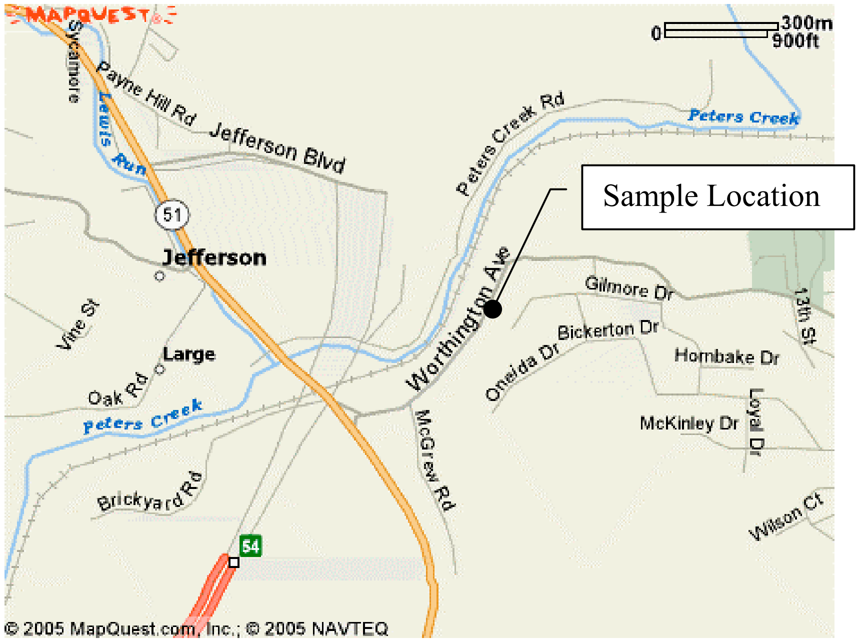

Figure 17 Worthington Avenue Site Location in Pittsburgh, PA

An 11-inch layer of asphalt was measured in the field on top of a 10-inch thick minimum granular base. Based on visual observation of the pavement, it did not appear that a geotextile fabric was used.

Photographs of the pavement section obtained at the Worthington Avenue site are shown in Figure 19 and 20.

Figure 18 Worthington Avenue

Two samples were obtained from the Worthington Avenue site. The samples were obtained at a location where a slope failure has occurred as shown in the figure above. As part of the slope failure, stress cracks were present in the pavement and a large pavement failure occurred in which a portion of the pavement layers and granular base could be observed. The granular base remained intact as shown in Figure 19. Although the source of the pavement distress has not been determined through any geotechnical analysis, it appears that the distress was due to failure of a nearby crib retaining wall.

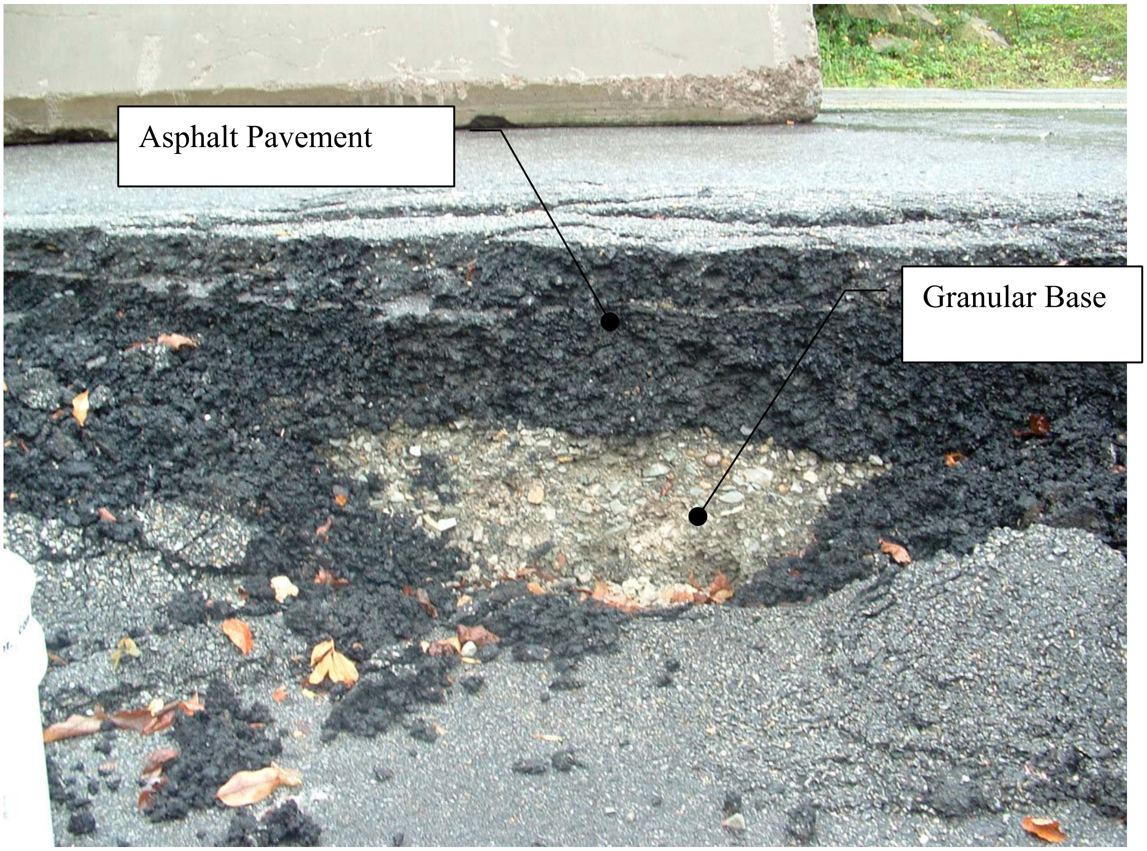

Figure 19 Pavement Cross-Section –Worthington Avenue

The location of the failure was near the edge of the pavement within the traveled cart way. Based on visual observation, it appeared that a single asphalt layer approximately 18-inches thick was placed on top of the granular base.

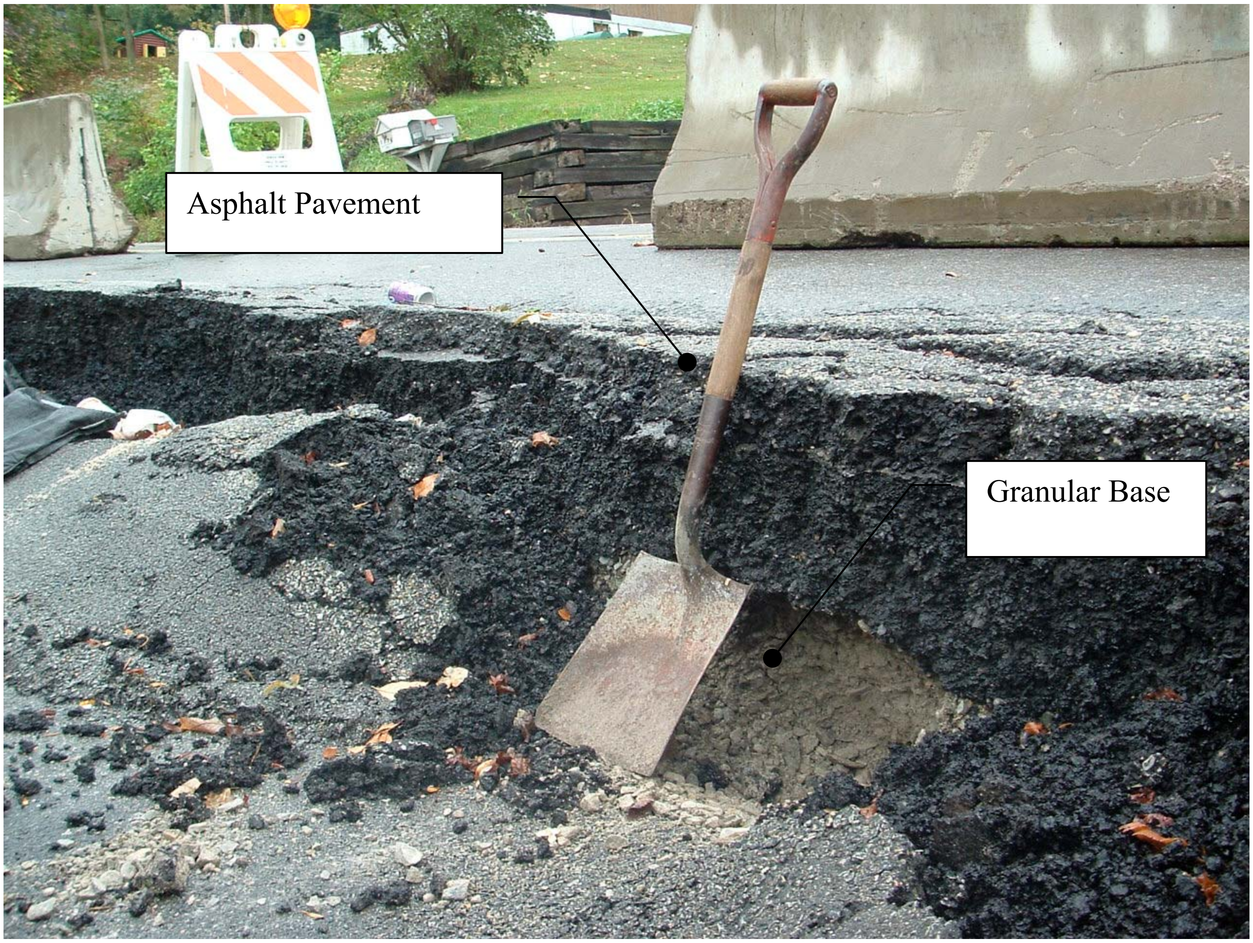

Figure 20 Pavement Cross-Section Worthington Avenue

4.2.3 Stockpile Samples S-2 and S-5

Two additional "baseline" samples were obtained from stockpile locations typically used as borrow sources for Pennsylvania Department of Transportation standard granular base material. The baseline material was used for a general comparison to the field obtained samples from the

roadway locations. PENNDOT 2A base material is widely used in roadway pavements throughout Pennsylvania. Typical gradation has been previously shown in Section 4.1.

5.0 LABORATORY TESTING

5.1 SAMPLE CHARACTERIZATION

The field samples were subject to laboratory testing to evaluate soil classification, crushing and observe changes in particle roughness.

5.1.1 Sample Classification

The samples were also classified using the Unified System Classification of Soils (USCS Classification) system which is the most widely used soil classification system. In the Unified System, the coarse-grained soils are divided into sub-classifications.[38]

1. Gravel and gravelly soils, symbol G
2. Sands and Sandy Soils, Symbol S
    a. Well-graded, fairly clean material, symbol W.
    b. Well-graded material with excellent clay binder, symbol C
    c. Poorly graded, fairly clean material, symbol P
    d. Coarse materials containing fines, not included in previous groups, symbol M

The grain size distribution curves also provide a range of particle sizes that are defined by the Pennsylvania Department of Transportation Standard base material, Type 2A. Although the grain size distribution curves provide a relative analysis of material gradation, it is difficult to ascertain strictly from the curves if crushing has occurred due to the wide range of aggregate that can be used as standard base. However, since two stockpile samples were obtained, a relative comparison can be made to ascertain evidence of crushing in the field samples.

5.1.2 Soil Grain Properties

Particles having a diameter greater than about 5 mm are typically classified as gravel. If the grains are visible to the naked eye, but less than 5 mm, the soil is typically classified as sand. The shape of the coarse-grained particles in a soil sample has an influence on the density and stability of the soil material. Coarse-grained soils can be described as rounded, sub rounded, angular and sub angular as illustrated in Figure 21 below.

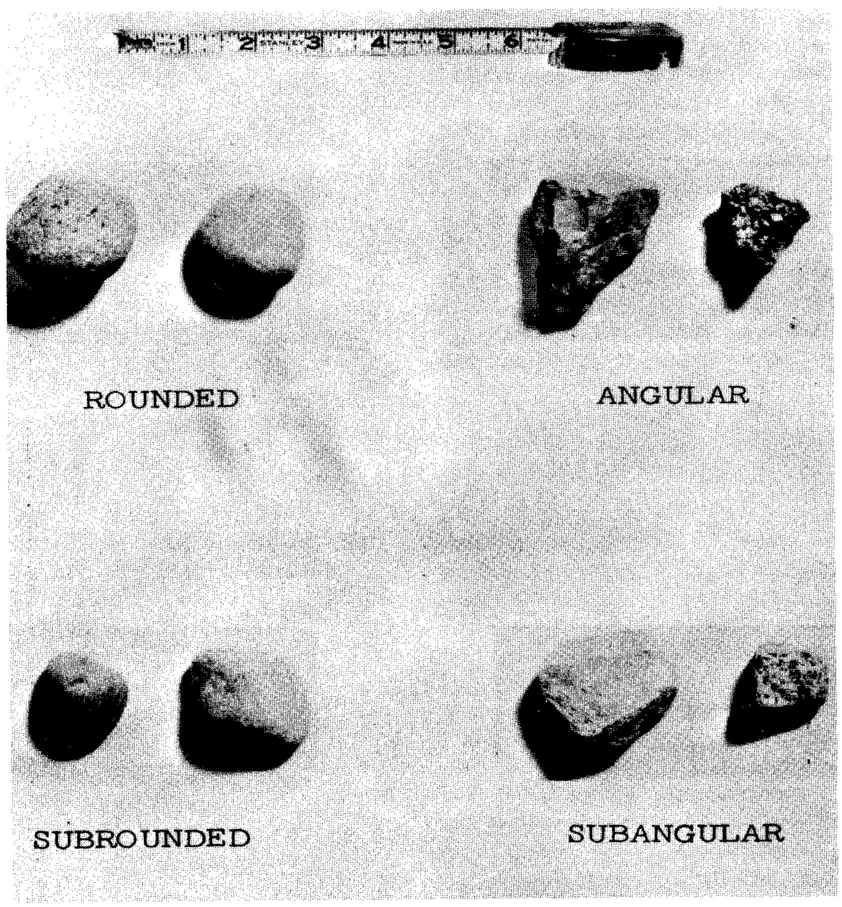

Figure 21 Descriptions of Shapes of Coarse-Grained Particles[39]

The granular material obtained from the sample locations was for the most part classified as angular with minimal sub angular, sub rounded and rounded particles.

5.1.3 Sieve Analysis

One of the most important properties of coarse grain soils is the particle size distribution. The finest sieve commonly used is the No. 200 U.S. Standard Sieve. The width of the opening of the No. 200 sieve is 0.075 mm. The No. 200 Sieve has been accepted as the standard boundary between coarse-grained and fine-grained soils. The particle size distribution of soil sizes less than 0.075 mm is determined by a hydrometer test. However, for the purpose of this study, only particle sizes greater than 0.075 mm will be analyzed. The samples were first analyzed for grain size distribution using the procedures outlined by ASTM D 4221. Since it was more important for this particular study to analyze the coarse fraction of the samples, a hydrometer test was not performed on the fines. A total of six sieve analyses were performed, one for each sample.

5.1.4 Uniformity Coefficient

The particle size characteristics of soils can be compared most conveniently by certain numerical values obtained from the grain size distribution curves. The three most commonly used by engineers are designated as D_{10}, the effective grain size, Cz, the coefficient of curvature and Cu, the uniformity coefficient. The uniformity coefficient is described as follows:[40]

$$Cu = D_{60}/D_{10} .$$

The effective size is the diameter of the particle corresponding to P=10% on the particle size plot. The effective size indicates that 10% of the particles are finer and 90% of the particles are coarser than the effective size.

Table 3 Uniformity Coefficient Results

Sample	**D_{10}**	**D_{60}**	**Cu**
Campbells Run S-1	0.4 mm	9.0 mm	22.5
Stockpile Sample S-2	**2.0 mm**	**15.5 mm**	**7.75**
Worthington Avenue S-3	0.85 mm	12.0 mm	14.1
Worthington Avenue S-4	1.75 mm	12.0 mm	6.85
Campbells Run S-5	1.0 mm	12.0 mm	12.0
Stockpile Sample S-6	**1.25 mm**	**14.0 mm**	**11.2**

5.1.5 Coefficient of Curvature

The second coefficient calculated based on the grain size distribution curve is the coefficient of curvature which is defined as follows:[41]

$$Cz=(D_{30})^2/(D_{60} x\ D_{10})$$

The effective size is the diameter of the particle corresponding to P=10% on the particle size plot and D_{60} in which 60% of the particles are finer.

Table 4 Coefficient of Curvature Results Table 4

Sample	**D_{10}**	**D_{30}**	**D_{60}**	**Cz**
Campbells Run S-1	0.4 mm	1.9 mm	9.0 mm	1.00
Stockpile Sample S-2	**2.0 mm**	**7.0 mm**	**15.5 mm**	**1.5**
Worthington Avenue S-3	0.85 mm	4.0 mm	12.0 mm	1.56
Worthington Avenue S-4	1.75 mm	4.5 mm	12.0 mm	0.96
Campbells Run S-5	1.0 mm	4.5 mm	12.0 mm	1.68
Stockpile Sample S-6	**1.25 mm**	**6.0 mm**	**14.0 mm**	**2.05**

The uniformity coefficient is more commonly used to identify samples as poorly graded. In well graded gravels, Cu is greater than 4 and C_Z is between 1 and 3. The Cu of all samples tested was greater than 4. However, the value of Cz varied with two field samples resulting in values less than 1.0.

5.2 SIEVE ANALYSIS RESULTS

5.2.1 Campbells Run Rd Samples

Figure 22 below shows the sieve analysis results from the Campbell's Run Road sample S-1. The sample was plotted on the same graph as the baseline stockpile samples. The graph also provides the range of particle sizes that fall within the gradation of the PENNDOT standard for comparison.

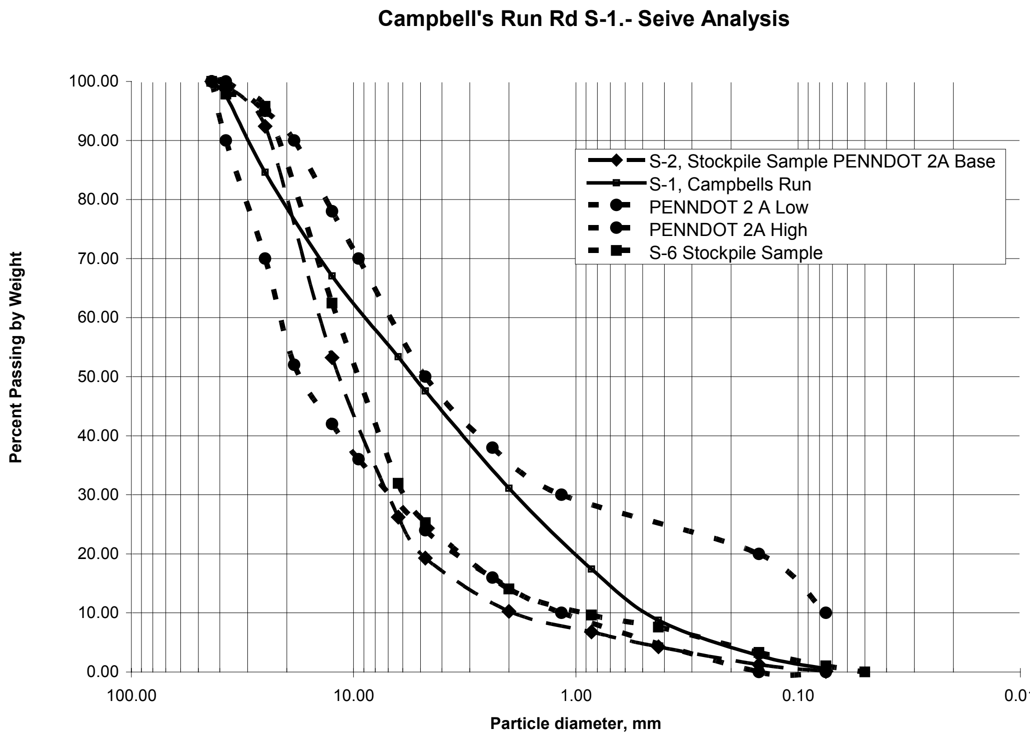

Figure 22 Sieve Analysis Campbells Run Rd Sample S-1

Based on a review of the above Figure 22, the grain size distribution curve of the sample is less steep and it appears that some crushing has occurred as shown in the smaller size particles in the field sample at sizes less than 10 mm.

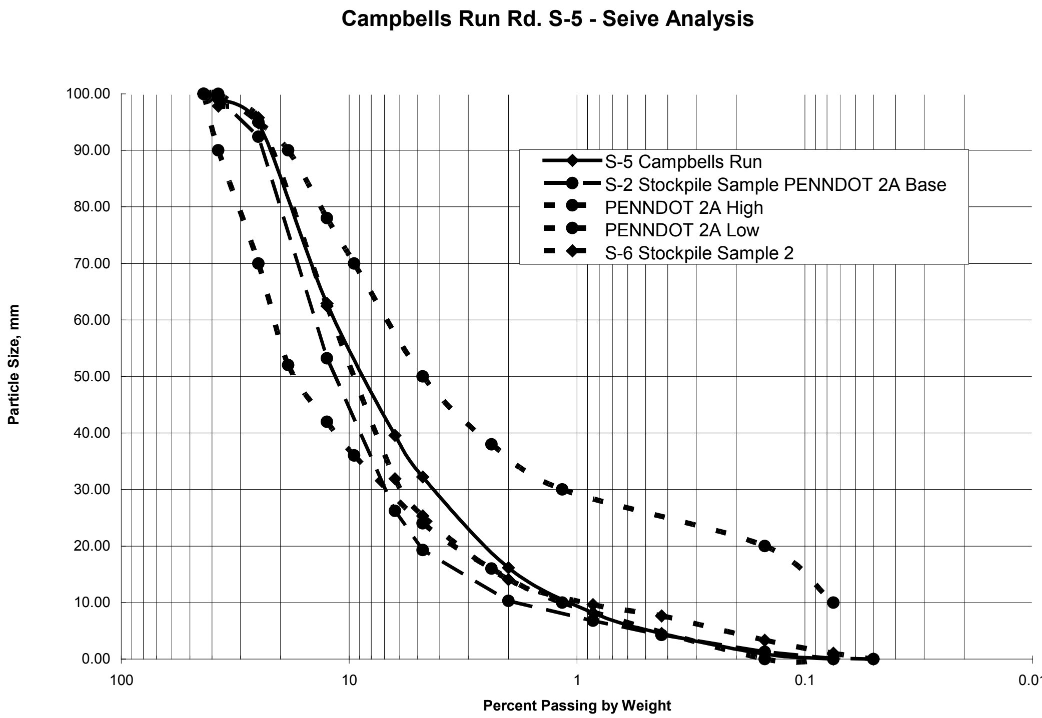

Figure 23 Sieve Analysis Campbells Run Rd Sample S-6

The field sample had larger particles than the baseline samples for particle sizes above 10 mm which could be due to a slight difference in initial gradation of the samples since both samples fall within the PENNDOT standard.

Figure 23 above shows the sieve analysis results from the Campbells Run Sample S-6. The sample was plotted on the same graph as the baseline stockpile samples. The graph also provides the range of particle sizes that fall within the gradation of the PENNDOT standard.

Using the stockpile samples as a baseline, it appears that a small amount of crushing may have occurred as shown in the smaller size particles in the field samples at sizes less than 10 mm. However, the gradation of the field sample is very close to the baseline sample gradation.

5.2.2 Worthington Avenue Samples

Figure 24 below shows the sieve analysis results from the Worthington Avenue sample S-3. The sample was plotted on the same graph as the baseline stockpile samples. The graph also provides the range of particle sizes that fall within the gradation of the PENNDOT standard.

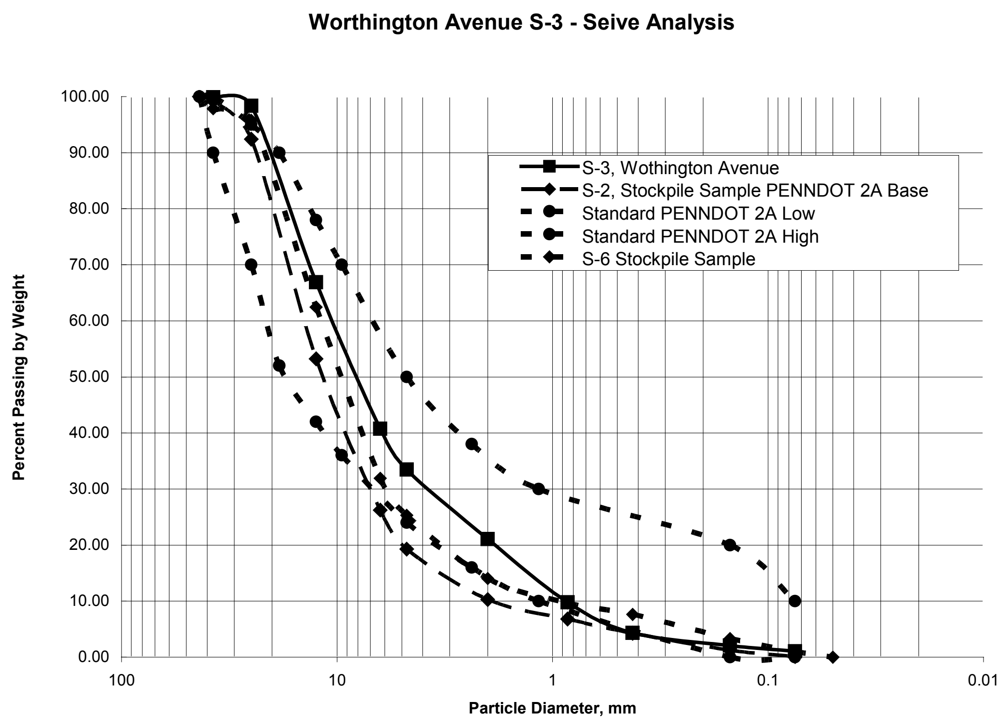

Figure 24 Sieve Analysis Worthington Avenue Sample S-3

A review of the above Figure 24, it appears that a small amount of crushing may have occurred as shown in the smaller size particles in the field samples at sizes less than 10 mm. As with the Campbells Run sample S-6, the gradation of the field sample is very close to the baseline samples.

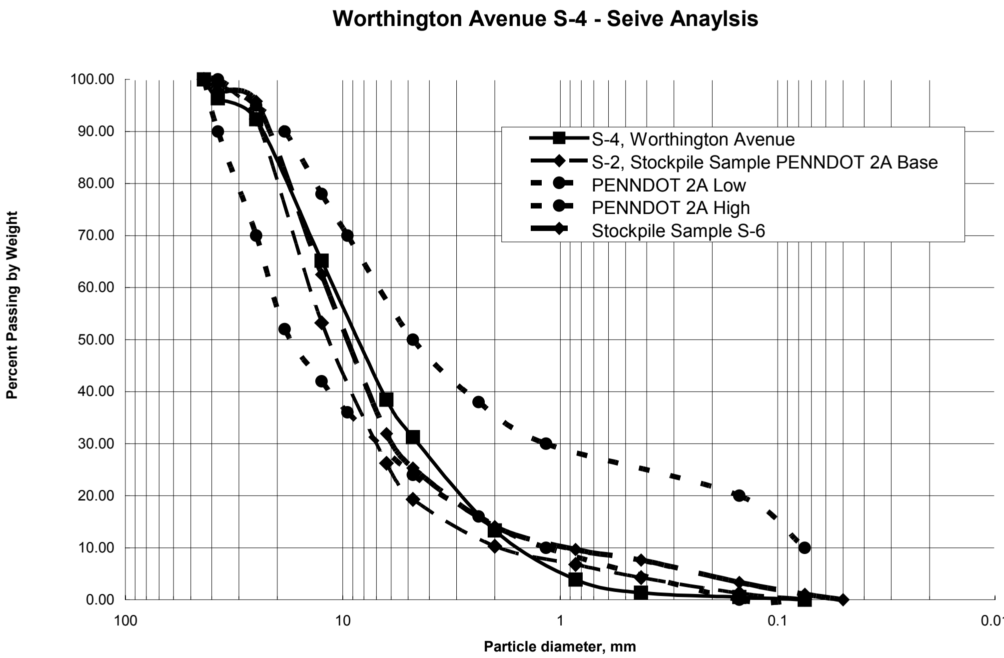

Figure 25 Sieve Analysis Worthington Avenue Sample S-4

Figure 25 above shows the sieve analysis results from the Worthington Avenue Sample S-4. The sample was plotted on the same graph as the baseline stockpile samples along with a graph of the range of particle sizes that fall within the gradation of the PENNDOT standard.

A review of the above Figure 25 , it appears that no crushing has occurred since the sample is very close to the baseline samples.

5.3 SUMMARY

A review of the gradation results indicates that minimal crushing has occurred in the field samples. A comparison of the field obtained samples with the standard “baseline” samples show that there were smaller particles comprising the field samples in 3 out of the 4 samples tested. All gradations of the samples result in particle sizes within the standard range of the Pennsylvania Department of Transportation standard. It appears that the base material suppliers provide material graded more toward the larger particle size. The reason for supplying larger material could be to allow for some degree of crushing occurring during transportation, placement and traffic loading.

In addition, the mineral compositions of the aggregates are mainly from sedimentary rock such as crushed limestone, sandstone and river gravel. This type of rock has a tendency to break down more as opposed to igneous or metamorphic minerals.

6.0 POINT LOAD TEST

6.1 POINT LOAD TEST BACKGROUND AND THEORY

6.1.1 Failure Theory

Particle fracture plays a significant role in the behavior of crushable soils. The failure of a spherical particle under compression is actually a tensile failure. [37] The particle size distributions of broken particles tend to be self similar or fractal. McDowell and Bolton proposed a study of the influence of grain strength on the internal angle of friction for dilatants, crushable aggregates. Applying a diametral force “F” on a particle of diameter “d”, a characteristic tensile stress can be defined as:[42]

$$\sigma = F/d^2$$

As reported by McDowell and Bolton, Lee previously determined that under normal load F, the tensile stress at failure is determined by using the above equation with the force F obtained at failure. Lee concluded that the average tensile strength of particles was a function of the diameter, d and that for a particle of a given size and mineralogy, the tensile strength is not constant but has a certain standard deviation about some mean. He also found that the average tensile strength is a function of the particle size d.

McDowell and Bolton further quantified the analysis by using fracture probability and they expanded on Weibull’s ceramic theory stating that a block under tension will retain its strength as long as all the parts comprising the block remain intact. Weibull previously stated that tensile strength of a block sample can be expressed as a function of the volume ratio at that particular strength as compared to a theoretical volume ratio. His theoretical volume ratio was defined as the value of the tensile strength in which 37% of the total number of tested samples survives. Refer to Figure 26 below.

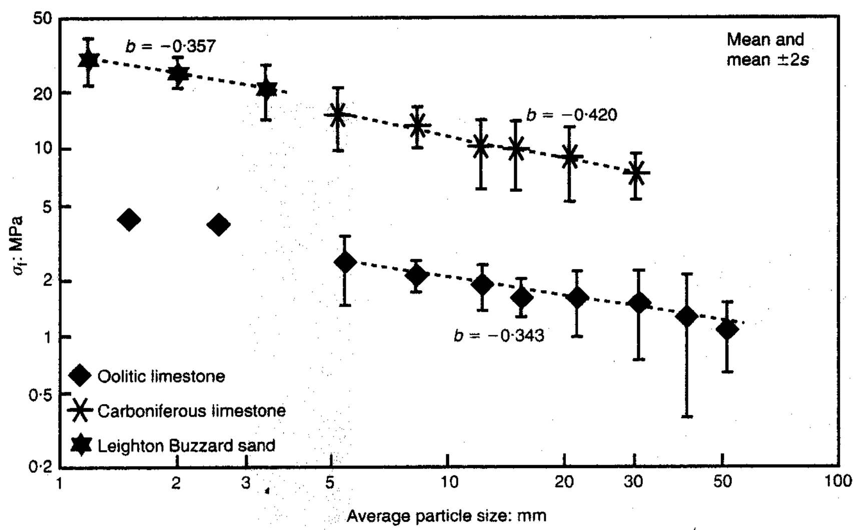

Figure 26 Mean Strength as a Function of Particle Size (Reproduced from Lee, 1992)[43]

6.1.2 Effect of Particle Size on Strength

In summary, by using ceramic materials as a testing medium, it was determined that small samples were stronger than large samples due to the fact that there was an increase in flaws in the larger samples and hence, more possibility of fracture.

McDowell and Bolton further proved that the evolution of the linear portion of the normal compression curve follows closely to the evolution of the fractal distribution of particle sizes to which point the uniformity coefficient tends towards a constant value. They also concluded that for an aggregate subjected to one-dimensional normal compression, the yield strength is proportional to the grain tensile strength.[44] The point-load test was used so that the tensile strength of the particles can be plotted versus the diameter and also to observe how the particles undergo crushing resulting from point loads.

6.2 POINT LOAD TEST PROCEDURE

6.2.1 Equipment and Procedure

Individual particles were placed in a point-load test apparatus and a constant vertical compressive load was applied. Prior to testing each particle was measured for diameter so that the stress can be computed. A vertical force was applied to the samples while the force value was observed in the meter until failure of the sample occurred. Tensile forces acting on the particle caused the particle to break.

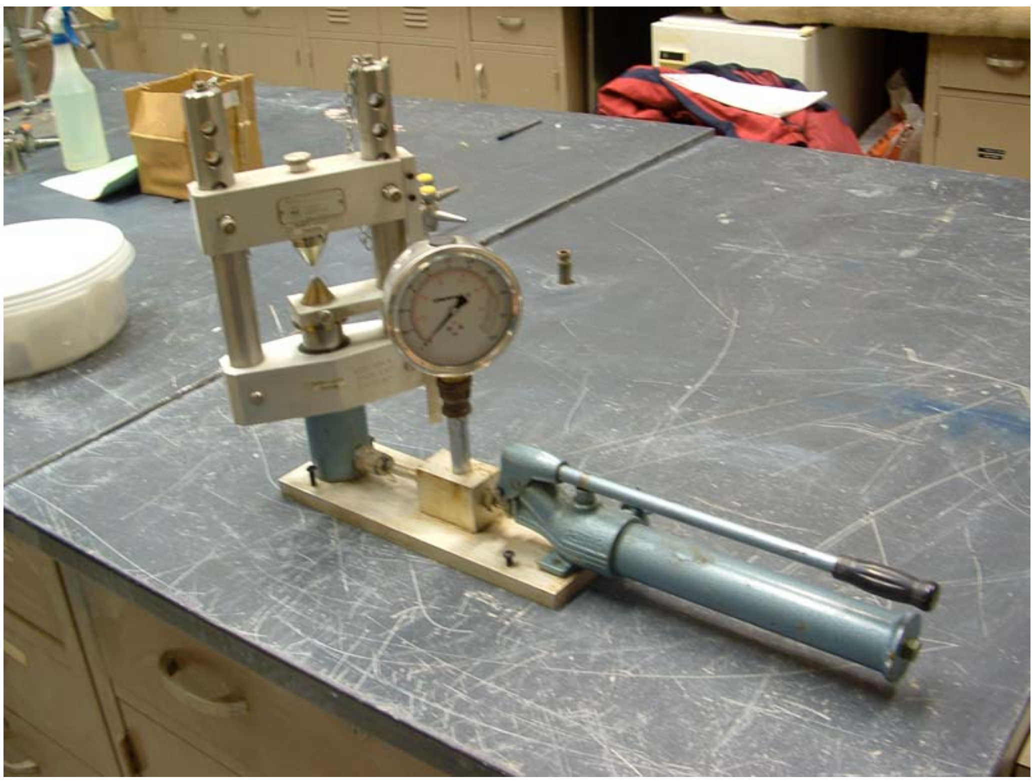

Figure 27 Point Load Test Apparatus

6.2.3 Graphs of Results

The samples were tested in both a dry and saturated state. The point load test results were plotted on log-log paper shown on the following graphs. A review of the figures below indicates that the particles in a dry state exhibited higher shear stress for the samples tested.

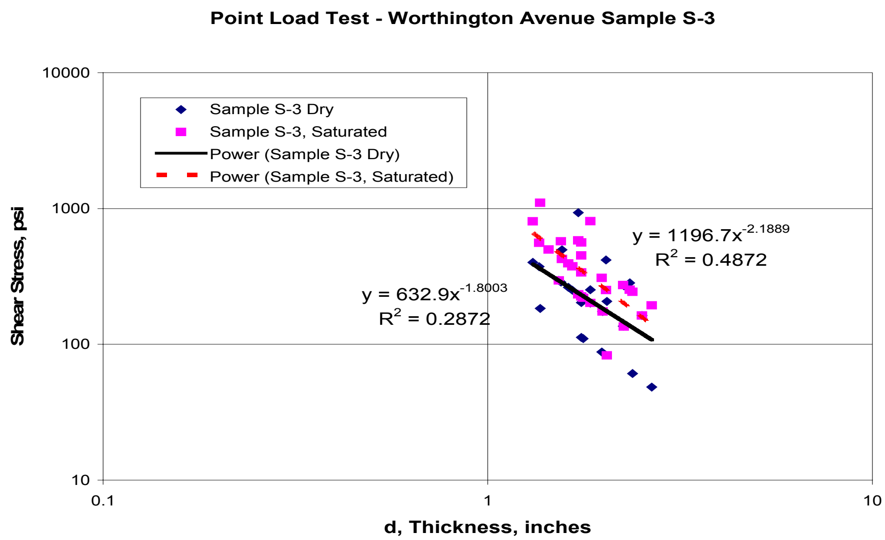

Figure 28 Point Load Test – Worthington Avenue Sample S-3

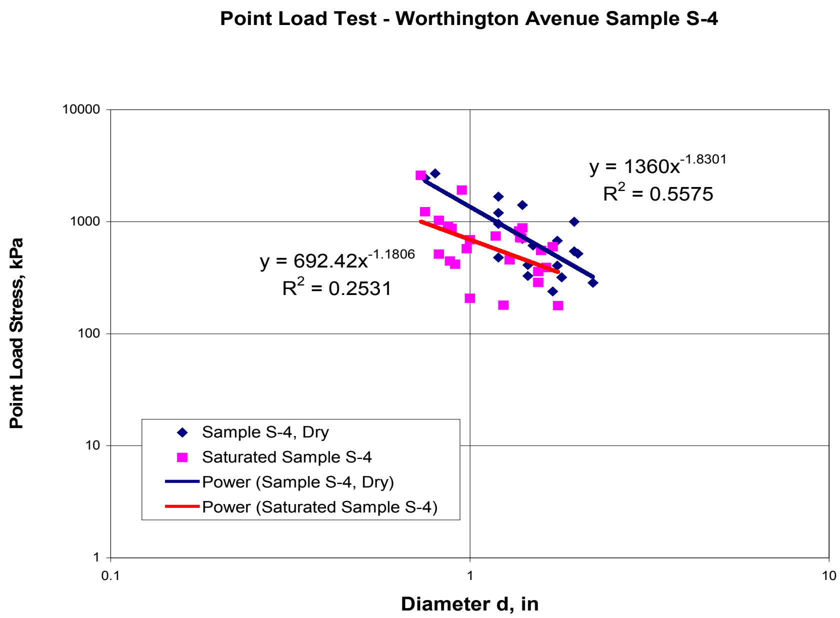

Figure 29 Point Load Test – Worthington Avenue Sample S-4

6.2.2 Point Load Test Sample Photographs

Photographs of the sample particles were obtained for visual observation after the point load test was conducted. The point load applied to the individual particles resulted in breakage occurring into both two and three piece segments. The sample breakage is illustrated in Figure 30.

Figure 30 Rock Fractures Resulting From Point Load

Figure 31 Fractured Rock From Point Load Testing

The above photographs illustrate the fractures that occurred to the particles after the particles were subjected to a point load until failure occurred. The particles fractured in both 2 and 3 piece individual segments.

6.3 SUMMARY

A review of the curves plotted for the samples tested using the point load tester showed that the overall trend is toward a decrease in tensile strength with increasing particle size which is in agreement with what was postulated. The theory is based on the fact that there are more flaws, hence more potential failure planes in larger particles than smaller particles. The phenomenon was observed in both the dry and wet samples.

Previous laboratory tests on granular soils performed for a rock fill dam concluded that the addition of water increased the rate and amount of crushing on the soils.[45] However, the point load test results from two separate samples analyzed in both the dry and saturated states indicate conflicting results. It cannot be concluded with any degree of confidence if saturated granular material is more susceptible to crushing than dry material based on the results of the point load test. Although a capillary effect can occur in the particles increasing the particles potential strength through suction, if too much water is present within the sample it can have a reverse strength effect by decreasing shear strength. Therefore, the variable water content along with the humidity in the air resulted in erratic results for the saturated samples.

7.0 ABRASION AND IMPACT TESTING USING THE LOS ANGELES MACHINE

7.1 ABRASION AND IMPACT TEST BACKGROUND AND THEORY

A baseline sample, S-2, was tested for resistance to degradation using the Los Angeles testing machine. The abrasion test is for aggregate smaller than 37.5 mm (1 ½ in).

This test measures degradation of the aggregate resulting from a combination of dynamic actions including abrasion, impact and grinding by means of impact of steel spheres inside of a steel drum. As the drum rotates, a shelf inside the drum picks up the sample and steel spheres and drops them creating a crushing effect.

This test is mainly used as an indicator of the relative competence of a specific aggregate sample with a certain mineral composition.

The machine consists of a hollow steel drum with a wall thickness of not less than 12.4 mm and closed at both ends. The drum is mounted on steel shafts which allow the cylinder to rotate in a horizontal axis. A removable steel plate inside the drum extends the full length of the drum and extends outward into the drum for a distance of 89 +/- 2 mm. The machine is counterbalance and driven to maintain a substantially uniform speed.

The charge consisting of the steel spheres depends on the grading of the sample. A predetermined sample mass of 5,000 grams is typically used, however, due to limited sample material, a sample mass of 2,500 grams was used. Since the material used in the test was a maximum of ¾ inches, 11 steel spheres were used as described in the ASTM C-131-03 test method procedure. The sample and the charge are placed in the drum and the machine is rotated at a speed of 30 to 33 revolutions per minute for 500 revolutions. The sample is then removed from the drum and a sieve analysis is performed so that it can be compared to the sample gradation prior to testing.

7.2 CALCULATIONS AND RESULTS

The loss difference between the original mass and the final mass of the test sample was calculated as a percentage of the original mass so that the percent loss can be determined.

The sample grading was as shown in the following Table 5:

Table 5 Sample Grading and Mass

Passing (Sieve)	Retained On (Sieve)	Grading (Mass, grams)
19.0 mm	12.5 mm	1,250
12.5 mm	9.5 mm	1,250
	Total	2,500

The final sample mass was determined to 2,214 grams. Therefore the percent loss calculated was determined to be 11.4 %. The required PENNDOT specification for this type of material is 55% so that the result is well within the required range.

8.0 MODIFIED PROCTOR

8.1 MODIFIED PROCTOR TEST BACKGROUND AND THEORY

Compaction is defined as “an artificial means of densifying soil by ramming, pressing or vibrating. During this process, air is expelled and the degree to which the soil grains may be brought in close contact depends mainly on the water content, on the nature of the soil and on the nature of the compactive effort”.[46] In addition, some degree of particle crushing occurs.

Granular base material, when placed as part of a pavement section beneath the asphalt layer, undergoes compaction to obtain a densification of the base. A well compacted base will minimize the potential of any soft, yielding areas and provide a solid granular structure capable of distributing the highway loading.

A modified proctor test was performed on the stockpile sample to observe the degree of crushing that occurs in the particles during a typical compactive effort. Roadway base material is generally compacted using a steel drum roller undergoing several passes. Both compaction and crushing occurs during this process and the Modified Proctor Test will simulate the compaction process.

8.2 TEST PROCEDURE

A representative sample of the stockpile was weighed and sieved to determine a baseline gradation of the material. A 6-inch diameter mold is used and a 10.0 pound hammer is used for the compactive effort in a series of 56 blows of the rammer for each layer. The compactive effort was applied to the sample until the top of the mold was reached. At this point, the sample was then removed, sieved and weighed to determine the gradation.

8.3 RESULTS

The following is the resulting gradation of the material.

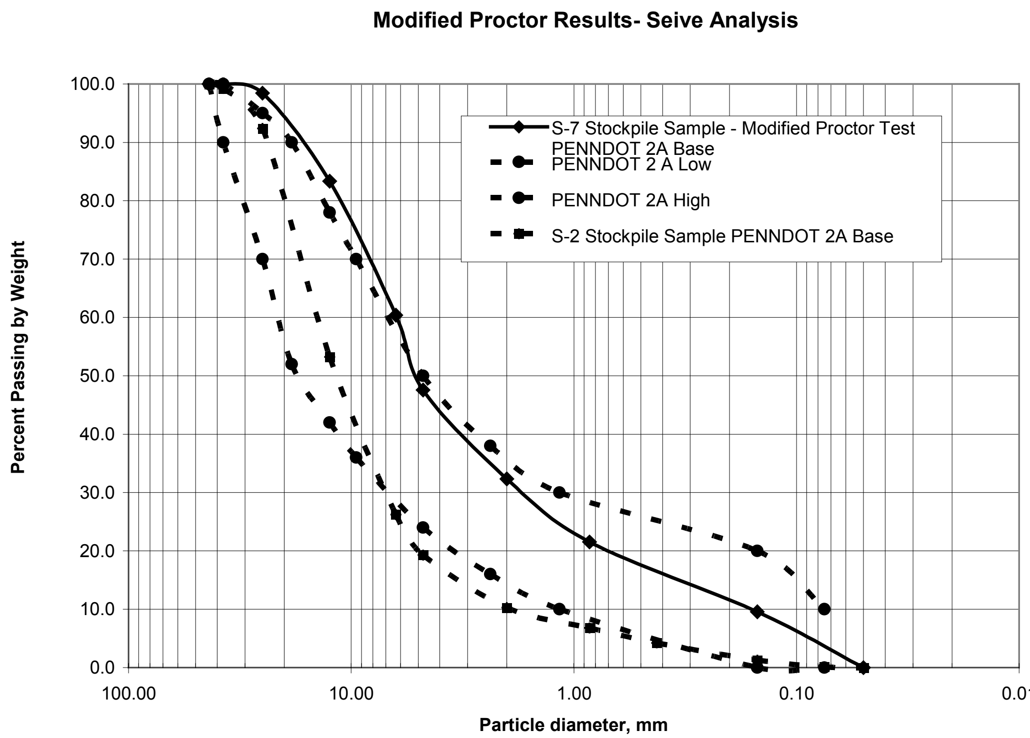

Figure 32 Gradation Comparison From Modified Proctor Testing

The results indicate a significant amount of crushing has occurred during compaction using the modified proctor test. Although this may not be a true indication of the crushing that actually occurs in the field, it provides an indication of the breakdown potential.

9.0 QUANTIFICATION OF CRUSHING IN THE SAMPLES

9.1 GRAIN SIZE ANALYSIS

9.1.1 Breakage Factor

The samples were subject to grain size distribution testing as outlined in the previous section. From the grain size distribution curves, two methods will be used to quantify crushing in the samples. The first method is that developed by Lade.[47] Lade extended the correlation between permeability and grain size by factoring in the particle crushing component B_{10}, based on the D_{10} particle size:

$$B_{10}=1-D_{10f}/D_{10i}$$

in which B_{10}=particle breakage factor; D_{10f}=effective grain size of the final gradation and D_{10i}=effective grain size of the initial gradation.

An analysis of the above particle breakage factor equation indicates that for no breakage, B_{10}=0; for infinite particle breakage B_{10}=1.

Table 6 Breakage Factor

Sample	$\mathbf{D_{10i}}$	$\mathbf{D_{10f}}$	$\mathbf{B_{10}}$
Campbells Run S-1	1.625 mm	0.4 mm	0.75
Worthington Avenue S-3	1.625 mm	0.85 mm	0.47
Worthington Avenue S-4	1.625 mm	1.75 mm	0
Campbells Run S-5	1.625 mm	1.0 mm	0.38

The particle breakage factor provides a quantification which indicates the degree of which a particular sample crushes. When the breakage factor approaches one, there is more breakage in the sample. Using the breakage factor, Lade further quantified breakage by relating it to total input energy such as that from triaxial compression and extension tests to evaluate particle crushing as it affects permeability during the crushing process. The next section discusses the calculation of permeability using the sample grain size distribution.

9.2 PERMEABILITY

One of the most widely used correlations relating grain size with permeability is Hazen's (1911) formula:[48]

$$K=100x\ (D_{10})^2$$

In which k=permeability coefficient (cm/sec) and D_{10}=grain diameter, (cm) corresponding to 10% of the material being smaller by weight and is often referred to as the effective grain size. This formula provides a rough estimate of the permeability of a soil sample using the grain size distribution curve.

Table 7 Permeability Results

Sample	**D_{10} (cm)**	**K (cm/sec)**
Campbells Run S-1	0.04	0.16
Stockpile Sample S-2	0.20	4.00
Worthington Avenue S-3	0.085	0.722
Worthington Avenue S-4	0.175	3.061
Campbells Run S-5	0.04	0.16
Stockpile Sample S-6	0.05	0.25

The results of the permeability calculation indicate a significant decrease in permeability as expected since the D_{10} particle size decreased. Permeability of base courses is important since drainage of water is required to minimize expansion of the pavement due to the freeze-thaw cycle.

9.3 FRACTAL ANALYSIS

9.3.1 Fractal Dimensions

Although soil classification methods are an effective way to quantify grain-size distribution of a particular sample, more specific methods can be used to further quantify grain particles such as fractal fragmentation theory. Two types of fractal dimensioning are used to evaluate the roughness of particles, Dr and D_F. The D_R and D_F dimensions are measures of the roughness of the particle. The first method being the parallel line method which measures the roughness by using a unit length of measurement as a function of the total length of the particle perimeter as discussed by Vallejo and Hyslip.[49]

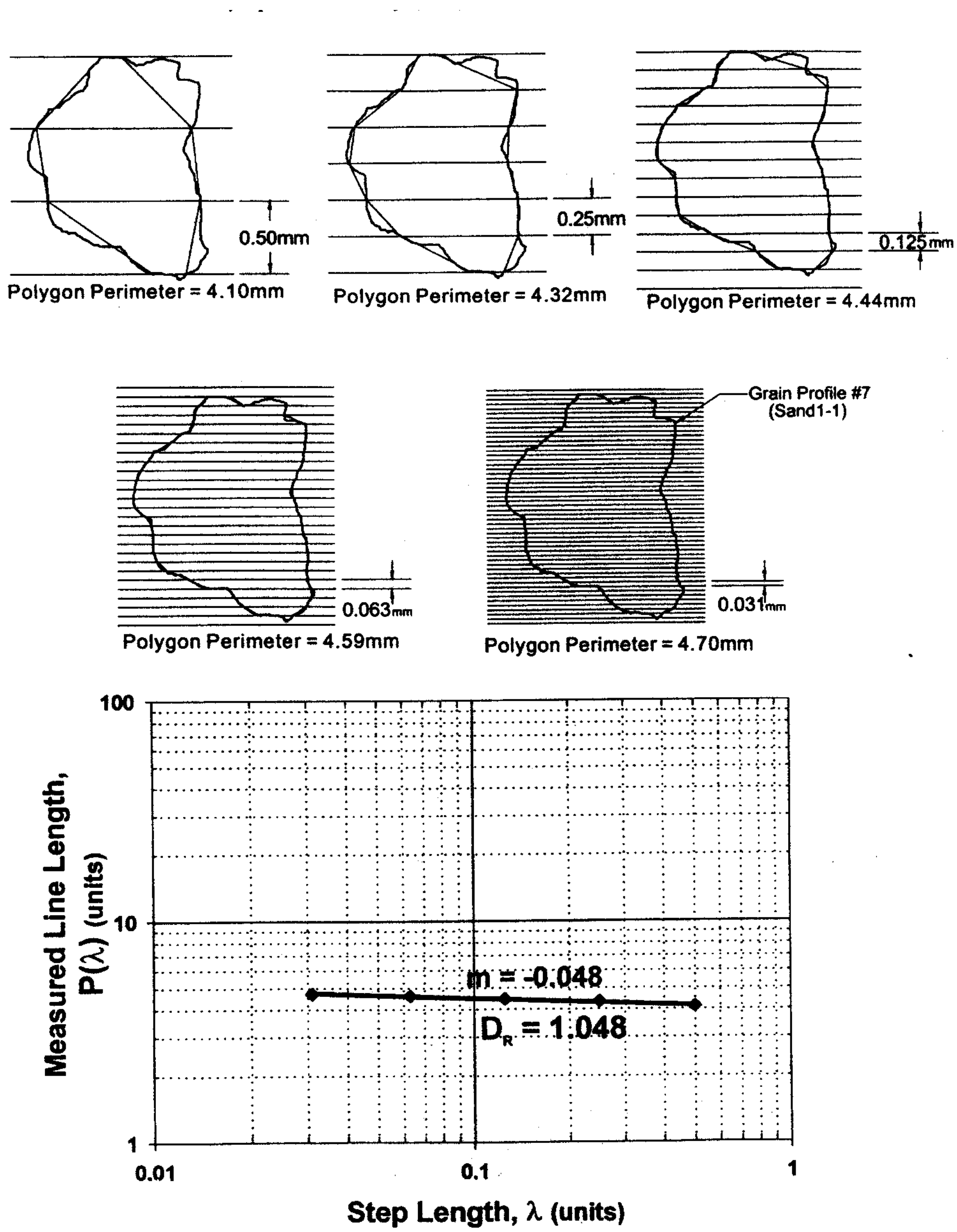

Figure 33 Parallel Line Method Example[50]

The roughness fractal dimension can be measured in two ways. *Mandelbrot* proposed using the ratio of linear extents within a given population of geometrically similar fractal shapes. The area-perimeter method evaluates and entire population of related shapes as opposed to individual particles using the divider method. However, both methods to measure D_R provide a measure of particle roughness.

9.3.1 Fractal Dimension Using the Area-Perimeter Method

The second analysis performed to determine particle roughness by means of fractal dimensioning was the area-perimeter method. Digital photographs of individual particles were obtained from each sample. The particles were grouped together in relative similar sizes using various sizes within the particle gradation.

Figure 34 Coarse grained particles tested using the Area-Perimeter

The results were plotted on logarithmic paper so that the fractal dimension, D_R could be analyzed. Results of the Fragmentation Fractal Dimension analysis, indicate that the field samples obtained from the roadway sample locations with the exception of the Worthington Avenue Sample, S-4, showed an increase in roughness as compared to the stockpile sample.

The results are summarized in Table 8.

Table 8 Roughness Fractal Dimension, D_R by the Area-Perimeter-Method

Sample	No. Of Particles	Dr	Average Dr
Field Sample S-1	12 14	1.000 1.058	1.029
Stockpile Sample S-2 (baseline)	21 42	**1.000** **1.087**	**1.043**
Field Sample S-3	42 34	1.063 1.001	1.032
Field Sample S-4	34 13	1.069 1.058	1.063

9.3.2 Fragmentation Fractal Dimension, D_F

The second method to quantify crushing from the grain size distribution uses the fractal dimension D_F of the size distribution before and after the crushing tests.[51] Since grain size distribution plots are a series of individual line segments, the fractal dimensioning theory can be applied. Tyler and Wheatcraft[52] developed an equation that relates mass and corresponding sieve diameters. This fractal distribution, D_F, of the distribution of particles can be obtained from the following relationship.

$$\frac{M(R < r)}{M_T} = \left(\frac{r}{r_L}\right)^{3-}$$

where M(R<r) is the cumulative mass of particles with size R smaller than a comparative size r; M_T is the total mass of particles; r is the size of the opening of the sieves, and r_L is the maximum particle size defined by the largest sieve size opening. The larger the value of D_F, the more well-graded is the size distribution of the particles.

The Figures below present the Fragmentation Fractal Dimension for each sample along with the "baseline" sample so that the relative comparison can be applied.

In general, Turcotte[53] determined that the Fragmentation Fractal Dimension, D_F is typically about 2.5 if pure crushing has occurred. Turcotte proposed some samples such as broken coal, DF=2.5, granite fragments from an underground explosion, D_F=2.5 and a Basalt sample from a projectile impact having a D_F =2.56. He concluded that materials in which the fractal dimension differed significantly from 2.5 generally were not subject to pure crushing and is typically part of sorting. Glacial till for example has a fractal dimension, D_F=.2.88. Further test were completed using a block of ice which showed that if the crushing force on a block of ice is determined equally by the fracture of fragments of all sizes, all particle sizes make an equal contribution so therefore, it results in a dimension D_F=2.5.

Therefore, the Fractal laboratory testing is a key test in determining if some degree of crushing has occurred in the field samples. Refer to Figure 35 below for Fractal comparison.

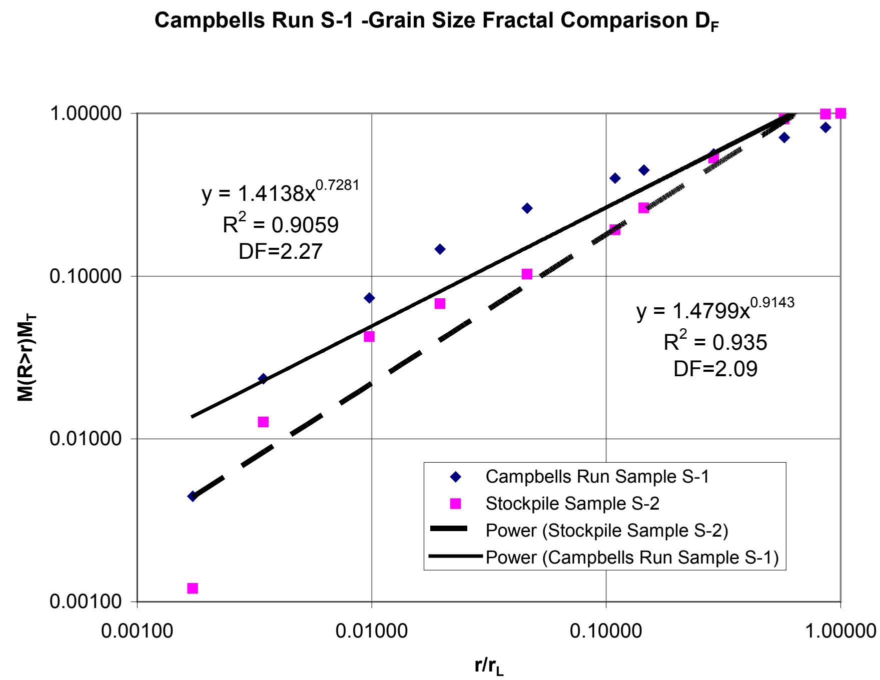

Figure 35 Campbells Run Rd Sample S-1, Grain Size Fractal Comparison

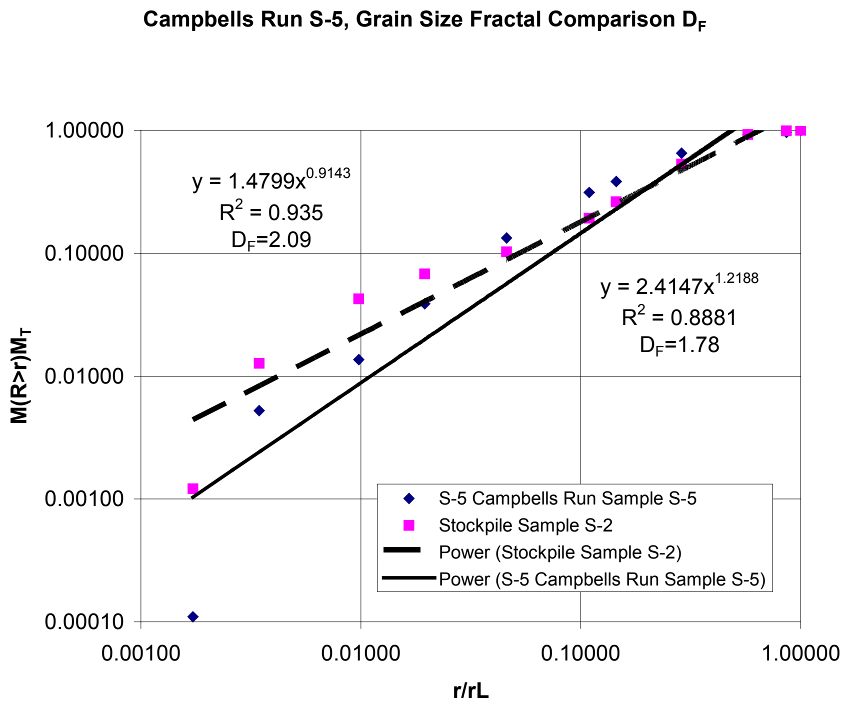

Figure 36 Campbells Run Road Sample S-5 – Grain Size Fractal Comparison

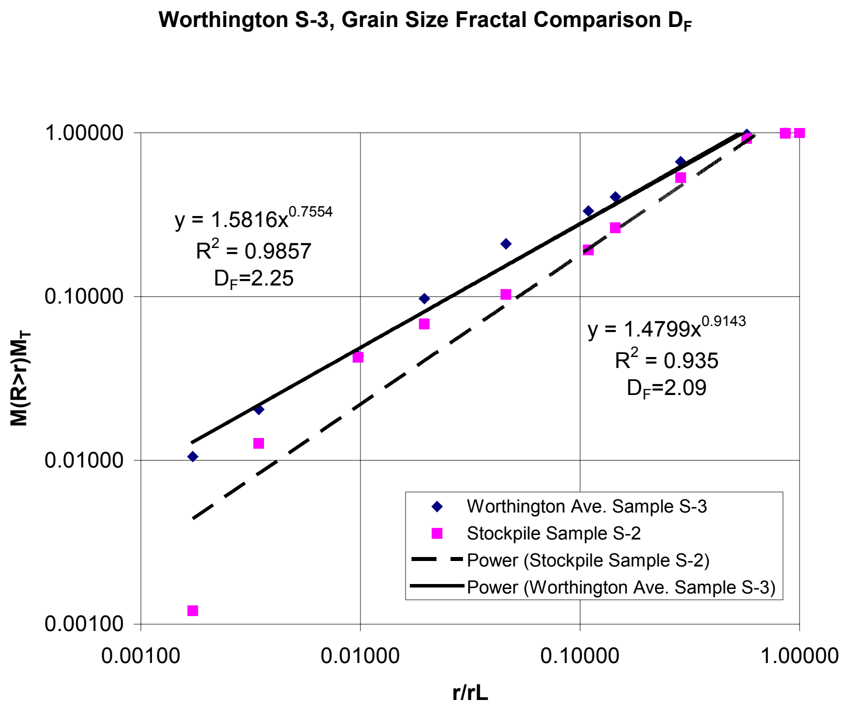

Figure 37 Worthington Avenue Sample S-3 – Grain Size Fractal Comparison

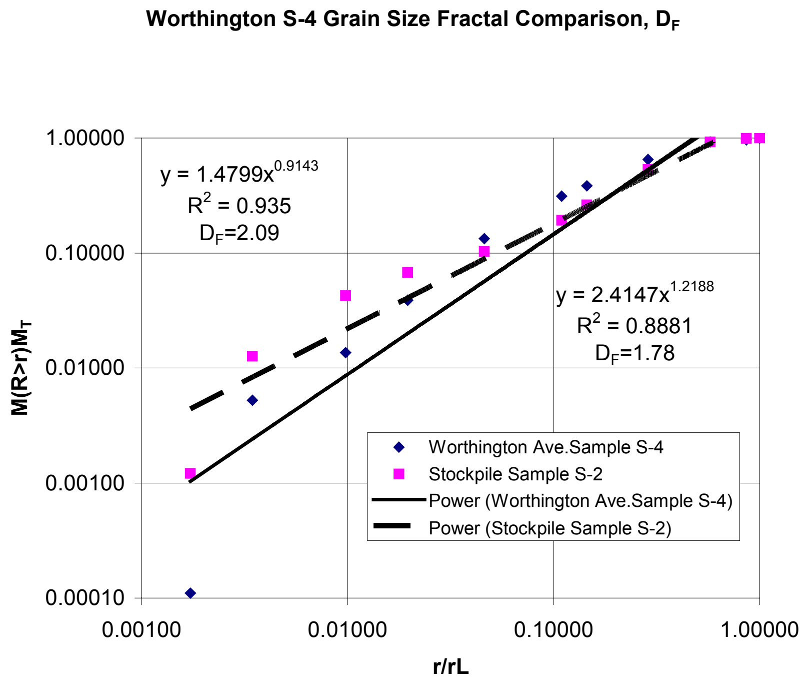

Figure 38 Worthington Avenue Sample S-4 – Grain Size Fractal Comparison

Their equation resulted in a linear relationship using a logarithmic transformation. The fractal dimension was determined using the slope measurement of the plotted line. Results of the Fragmentation Fractal Dimension analysis, indicate that the field samples obtained from the roadway sample locations with the exception of the Worthington Avenue Sample, S-4, showed an increase in roughness as compared to the stockpile sample. In addition, a review of the results of D_F indicates that all of the samples with the exception of the two stockpile samples and Worthington Avenue Sample, S-4 resulted in a fractal dimension close to 2.5. The fractal dimension of D_F=2.5 was used by Turcotte as a basis for crushing.

Based on the results of the Fractal Fragmentation Dimension, it could be concluded that some degree of crushing occurred in the field samples. The results are shown below in Table 9.

Table 9 Sample Classification and Fractal Dimension, D_F

Sample	**USCS Classification**	**Fractal Dimension, D_F**
Campbell's Run Sample S-1	GW	2.247
Stockpile Field Sample S-2	GP	2.085
Worthington Ave, Sample S-3	GW	2.245
Worthington Ave, Sample S-4	GP	1.728
Campbells Run Sample S-5	GW	2.245
Stockpile Field Sample S-6	GW	1.728

According to the USCS Classification method, the samples are grouped within the GP-GW range and using the uniformity coefficient, Cu, the soils are GP since the Cu for each sample is less than 4.

10.0 SUMMARY AND CONCLUSIONS

The data obtained from collection, subsequent laboratory testing, quantification and analysis of the field crushing of granular samples was presented in the forgoing sections. Based on a review of the results of this analysis, the following summary and conclusions have been made:

The gradation of the field samples obtained from existing roadway base courses followed closely to the Pennsylvania Department of Transportation Specification for roadway base courses. The PADOT specification provides for a range of particle sizes that can be used for bases courses to give the contractor some flexibility, but at the same time specify a coarse material that can achieve the requirements of base materials such as strength and permeability. The gradation of the stockpile samples obtained for comparison followed closely to the PADOT specification range of particle sizes and also, the gradation of both stockpile samples were very similar. .Since the samples obtained from the field locations were compared to "baseline" samples obtained from soil stockpiles, analysis of the actual crushing of the materials based strictly on grain size distribution curves was somewhat limited. However, evaluation of the crushing parameters such as breakage factor, fractal dimension, abrasion testing and proctor testing indicate that crushing can and does occur in the granular base.

From a review of the results of grain size distribution curves for each field sample as it pertains to crushing, it appears that some degree of crushing has occurred. Although there were some anomalies observed in the grain size distribution curves for some samples above the 10 mm size, most samples exhibited some degree of apparent crushing as evidenced by the smaller particle sizes in the lower half of the sample. However, the crushing was not determined to be severe.

The point load test was used to observe the effects of crushing on individual particles when undergoing compressive loading and to observe the effects of strength on particle size. The samples were observed to break into 2 or 3 separate smaller pieces and a plot of the particle strength versus particle size indicated a decrease in particle tensile strength with increasing particle size. The results indicate that there was a decrease in particle tensile strength with increasing particle size, as theorized by *McDowell and Bolton.* However, the point load test results from two separate samples analyzed in both the dry and saturated states indicate conflicting results. It cannot be concluded with any degree of confidence that a saturated granular material is more susceptible to crushing than a dry granular material based on the results of the laboratory testing.

One baseline sample was tested for degradation using ASTM C131-03 using the Los Angeles Machine. The sample percent loss calculated was determined to be 11.4 % which indicates a competent and durable material. The required PENNDOT specification for this type of material is 55% so that the result is well within the required range for Pennsylvania Roadway base material. However, the testing showed that a break down and crushing of the particles does occur.

A particle breakage factor was calculated along with the sample permeability based on the resulting grain size distribution to provide for a quantification of crushing. The resulting breakage factors ranged from zero to 0.75. Further quantification of the crushing of the samples was performed using the Roughness Fractal Dimension measured for the samples based on the area-perimeter method. The results indicate that some degree of crushing has occurred in 3 of the 4 samples analyzed as evidenced by lower Fractal Dimensions as compared to the baseline.

The second fractal dimension analyzed was the Fragmentation Fractal Dimension, D_F which quantifies crushing of the particles by relating mass and corresponding sieve diameters. A review of the results of D_F indicates that all of the samples with the exception of the two stockpile samples and Worthington Avenue Sample, S-4 resulted in a fractal dimension close to 2.5. The fractal dimension of D_F=2.5 was used by Turcotte as a basis for crushing, therefore indicating that some degree of crushing has occurred rather than a sorting of the particles.

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