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The Effect of Voids in Mineral Aggregate (VMA) on Hot Mix Asphalt Pavements

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16. Abstract (Limit: 200 words)

This research project investigates the level of Voids in the Mineral Aggregate (VMA) in Minnesota paving projects. Researchers analyzed 10 paving projects from 1996 to determine if a VMA decrease occurred, the magnitude of the de =crease, and the potential causes of the decrease. Potential causes include the generation of fines, high-production temperatures, and long storage or cure times.

Three of the 10 projects had a VMA decrease of 1.9 or more. These three projects also had the highest plant temperatures and fairly long storage times, which makes increased asphalt absorption a likely cause of the VMA decrease. Five projects showed a moderate drop in VMA. Most had some increases in fines, and some had moderately high plant temperatures and storage times. The two projects with little or no changes in VMA had very little change in gradation, and moderate to low plant temperatures and storage times.

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The Effect of Voids in Mineral Aggregate (VMA) on Hot-Mix Asphalt Pavements

Final Report

Prepared by:

Bruce A. Chadbourn

Eugene L. Skok, Jr.

Benita L. Crow

Samantha Spindler

University of Minnesota
Department of Civil Engineering
500 Pillsbury Dr. SE
Minneapolis, MN 55455-0116

David E. Newcomb
National Asphalt Pavement Association
NAPA Building
5100 Forbes Blvd.
Lanham, MD 20706-4413

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Prepared for

Minnesota Department of Transportation
Office of Research Services
First Floor
395 John Ireland Boulevard, MS 330
St. Paul, MN 55155

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Executive Summary

Improving the ability of asphalt pavement to survive the weather cycles in Minnesota has been the subject of research for many years. Asphalt durability is often linked to the thickness of the asphalt coating on the aggregate particles. In order for a pavement to have adequate film thickness, there must be sufficient space between the aggregate particles in the compacted pavement. This void space is referred to as Voids in the Mineral Aggregate (VMA). It must be sufficient to allow adequate effective asphalt (that which is not absorbed into the aggregate particles) and air voids.

The study described in this report was conducted to investigate the level of VMA in Minnesota paving projects. A problem develops when a mix with sufficient VMA is produced in the mix design procedure, yet after construction the measured VMA is significantly lower. This phenomenon is referred to as "VMA collapse" and is the suspected cause of many durability related failures.

Ten Minnesota paving projects from 1996 were analyzed to determine if a VMA decrease occurred, the magnitude of the decrease, and the potential causes of the decrease. Potential causes include the generation of fines, high production temperatures, and long storage or cure times. The generation of fines may be due to construction-related aggregate degradation, and can decrease VMA by increasing the surface area of the aggregate blend. High temperatures and long storage times can result in increased asphalt absorption into the aggregate relative to the mix design, making less asphalt available to coat the particles.

Three of the ten projects had a VMA decrease of 1.9 or more. These three projects also had the highest plant temperatures and had fairly long storage times, which makes increased asphalt absorption a likely cause of the VMA decrease. There were five projects with a moderate drop in VMA. Most had some increase in fines, and some had moderately high plant temperatures and storage times. The two projects with little or no change in VMA had very little change in gradation, and moderate to low plant temperatures and storage times.

The Lottman moisture sensitivity test was also performed on samples from the ten projects, but no correlation was found between the test results and change in VMA. A precision analysis was

also conducted for the Lottman moisture sensitivity test. Asphalt samples were sent to several laboratories in Minnesota for compaction and moisture sensitivity testing. The between-laboratory variability was considered unacceptable according to statistical tests recommended by the American Society for Testing and Materials (ASTM). Among the assumed causes of this variability were the use of assumed rather than measured specimen heights and variations in saturation procedures.

CHAPTER 1
INTRODUCTION

BACKGROUND

Durability of Hot-Mix Asphalt

Durability of an asphalt mixture refers to the ability of the mixture to retain the original properties. These include the resistance to load and abrasion. Resistance to load can be impaired when:

1. The asphalt becomes hard and brittle and thus cannot withstand strains without fracturing.
2. The asphalt debonds from the aggregate (truly strips) causing the surface to lose strength and subsequently crack and disintegrate.

Durability also refers to the ability of the mixture to resist abrasion of the surface due to the scraping action of tires combined with water. The surface is more susceptible to abrasion if:

1. The void content is high allowing air and water to prematurely harden the asphalt.
2. The asphalt and aggregate are not chemically compatible, making it easier to "strip" the asphalt from the aggregate.
3. The asphalt film thickness is not sufficient to protect the mix from the abrasive action of tires and water.

For a given asphalt and aggregate mixture, the durability is enhanced if adequate film thickness is attained. For a given effective asphalt content, the film thickness will be greater if the aggregate gradation is coarser. This can most effectively be accomplished by decreasing or minimizing the percentage of fines. Establishing an adequate Voids in Mineral Aggregate (VMA) during mix design and in the field will help establish adequate film thickness without excessive asphalt bleeding or flushing.

Krugler, et al [1] described durability as one of the most important factors in the performance of hot-mix asphalt (HMA). Durability refers to the ability of an HMA pavement to resist changes in properties as the pavement weathers and ages.

Voids in Mineral Aggregate (VMA)

VMA is the volume of intergranular void space between the aggregate particles of a compacted paving mixture. It includes the air voids and the volume of the asphalt not absorbed into the aggregate (Roberts, et al [2]). Stated another way, VMA describes the portion of space in a compacted asphalt pavement or specimen which is not occupied by the aggregate. VMA is expressed as a percentage of the total volume of the mix.

When aggregate particles are coated with asphalt binder, a portion of the asphalt binder is absorbed into the aggregate, whereas the remainder of the asphalt binder forms a film on the outside of the individual aggregate particles. Since the aggregate particles do not consolidate to form a solid mass, air pockets also appear within the asphalt-aggregate mixture. Therefore, as Figure 1.1 illustrates, the four general components of HMA are: aggregate, absorbed asphalt, asphalt not absorbed into the aggregate (effective asphalt), and air. Air and effective asphalt, when combined, are defined as VMA.

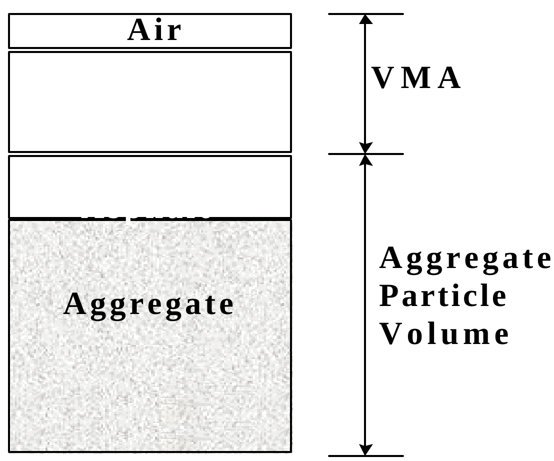

Figure 1.1 Illustration of VMA

VMA is calculated according to the following relationship:

$$\mathrm{VMA} = 100 - \frac{P_s \times G_{mb}}{G_{sb}} \tag{1.1}$$

Where:

P_s = Aggregate content, percent by total mass of mixture

G_{sb} = Bulk specific gravity of total aggregate

G_{mb} = Bulk specific gravity of compacted mixture

Figure 1.1 illustrates how VMA is derived in terms of volume and mass. VMA is the percentage of the compacted HMA mixture that is made up of asphalt not absorbed into the aggregate and air voids. Asphalt not absorbed into the aggregate is referred to as the effective asphalt binder. According to Krugler , et al [1], the absorbed asphalt is excluded because it does not contribute significantly to the durability or strength properties of the mixture.

The importance of designing VMA into an HMA mix has been recognized for many years. It was first discussed and used by McLeod in 1956 [3]. For many years, the Asphalt Institute mix design procedures have used a minimum VMA criteria that is dependent upon maximum aggregate size. If the VMA is too low, it can be increased by modifying the gradation, asphalt content, or particle angularity. Table 1.1 shows typical minimum VMA values recommended by the Asphalt Institute [4].

Table 1.1 Minimum VMA (after Asphalt Institute [4]).

Nominal Maximum Particle Size [1, 2]		Minimum VMA, percent		
		Design Air Voids, percent [3]		
mm	**in.**	**3.0**	**4.0**	**5.0**
1.18	No. 16	21.5	22.5	23.5
2.36	No. 8	19.0	20.0	21.0
4.75	No. 4	16.0	17.0	18.0
9.5	3/8	14.0	15.0	16.0
12.5	1/2	13.0	14.0	15.0
19.0	3/4	12.0	13.0	14.0
25.0	1.0	11.0	12.0	13.0
37.5	1.5	10.0	11.0	12.0
50	2.0	9.5	10.5	11.5
63	2.5	9.0	10.0	11.0

1 - Standard Specification for Wire Cloth Sieves for Testing Purposes, ASTM E11 (AASHTO M92)
2 - The nominal maximum particle size is one size larger than the firs sieve to retain more than 10 percent.
3 - Interpolate minimum voids in the mineral aggregate (VMA) for design air void values between those listed.

Asphalt Film Thickness

One of the key elements in the durability and moisture susceptibility of an asphalt mixture is asphalt film thickness. Asphalt film thickness describes the dimension of the asphalt binder coating of the aggregate particles. A thin asphalt coating on aggregate particles is one of the primary causes of premature aging of the asphalt binder, and is one definition of lack of durability.

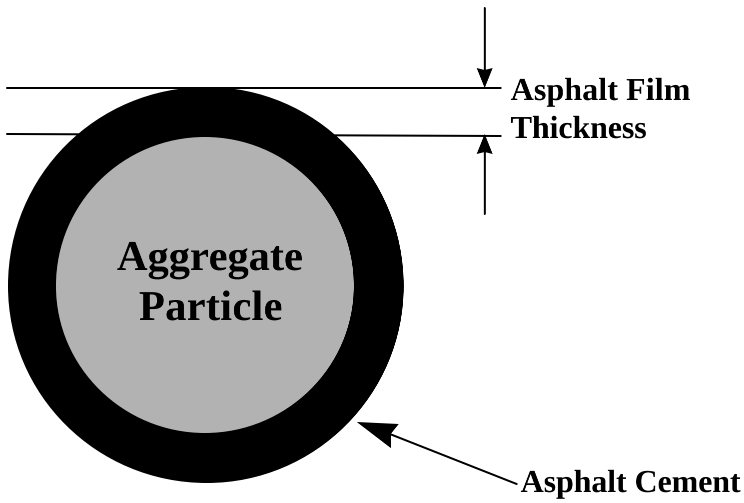

Figure 1.2 Illustration of Asphalt Film Thickness

According to Roberts, et al [2], inadequate film thickness can create a lack of cohesion between aggregate particles and create a "dry" mix. Also, if the asphalt film is too thin, air which enters the compacted HMA can more rapidly oxidize the asphalt, causing the pavement to become brittle. Additionally, if the aggregates are hydrophilic, thin asphalt films are more easily and rapidly penetrated by water than thick ones, causing stripping or debonding of the asphalt binder from the aggregate.

When film thickness is being calculated (estimated) it should be based on the quantity (volume) of asphalt on the surface of the aggregate and not include that which is absorbed into the aggregate particles. The quantity of asphalt on the aggregate surface is defined as the effective asphalt content which can be calculated by subtracting the percent asphalt absorption from the total asphalt content.

Asphalt film thickness is measured in microns as shown in Figure 1.2. It can be calculated by dividing the effective volume of asphalt binder by the total estimated surface area of the aggregate particles.

Film thickness has been shown to be a function of size distribution, particle shape, and the amount of asphalt binder in the mix.

Kandhal and Chakraborty [5] noted difficulties in defining the concept of an “average film thickness”. The validity of assigning a film thickness calculated simply by dividing the total surface area of the aggregate, obtained from its gradation, by the volume of effective asphalt binder is questionable. It is unlikely that all the particles in a mix will have the same average asphalt film thickness. Coarser aggregate particles may or may not have the film thickness that fine aggregate particles have, and the extremely fine portions of the aggregate may become embedded in the asphalt binder completely.

Aljassar and Haas [6] described a simple method of estimating aggregate surface area. This method involves assuming a smooth, spherical shape for aggregate particles, with the particle diameter equal to the size of the opening of a mesh screen on which the aggregate particle would be retained during a sieve analysis. The formula stated below can be used to determine the approximate surface area of an aggregate gradation:

$$A_T = a_s \times N = \frac{6W_s}{G_{sb} \times \rho_w \times d} \tag{1.2}$$

Where:

A_T = Total surface area of all aggregate particles (m^2)

a_s = Surface area of an aggregate particle (m^2)

N = Number of aggregate particles

W_s = Mass of aggregate (kg)

G_{sb} = Specific gravity of the aggregate

ρ_w = Density of water (assume 1000 kg/m^3)

d = Diameter of aggregate particle (m)

When the surface area is calculated using Equation 1.2, the film thickness can be estimated using Equation 1.3 developed by Hveem at the California DOT (Roberts, et al [2]). The formula provides only an estimation of the average film thickness on an aggregate particle:

$$T_F = \frac{V_{be}}{A_T \times W_s} \times 304\,800 \tag{1.3}$$

Where:

T_F = Average film thickness, microns

V_{be} = Volume of effective asphalt binder (ft^3)

A_T = Surface area of the aggregate (ft^2 / lb of aggregate)

W_s = Mass of aggregate (lb)

(304 800 is a conversion factor used to express T_F in microns.)

The calculated film thickness should be considered an estimate, but it does give a measure of how film thickness will vary with gradation, absorption, and other parameters.

Generally, it has been found that adequate film thickness can only be maintained without a tendency to bleeding by providing adequate volume between the aggregate particles. This can be accomplished by selecting an aggregate gradation and mix design with appropriate VMA. The definition of and mix design for appropriate levels of VMA are covered in the next section.

In order to ensure durable flexible pavements, it is necessary to design mixes with adequate film thickness. The mixes must be able to maintain the design asphalt film thickness in the produced pavement following construction.

Moisture Susceptibility Testing

The susceptibility of asphalt mixtures to moisture is another measure of the durability of an asphalt mixture. Moisture susceptibility of a mix is determined by first testing the mixture dry and then after soaking and/or freezing for specified periods. One standard method used to evaluate moisture susceptibility and stripping is the Lottman Test (ASTM D 4867) [7]. For this test, six specimens are compacted to 7 ± 1 % air voids. The six specimens are divided into two groups of three so that the average air void content of the groups are approximately equal. One group is tested dry and the other is tested after a period of moisture conditioning. The conditioning consists of vacuum saturating the specimens to between 55 and 80 % saturation. They are then placed in a 60 °C (140 °F) water bath for 24 hours followed by a 25 °C (77 °F) bath for 1 hour. Indirect tensile strength is then determined for the dry and wet samples.

Moisture susceptibility is reported as a tensile strength ratio (TSR) which is calculated using Equation 1.4:

$$TSR = \frac{S_{tm}}{S_{td}} \times 100 \quad (1.4)$$

Where:

TSR = Tensile Strength Ratio

S_{tm} = Average tensile strength of the moisture-conditioned samples, kPa

S_{td} = Average tensile strength of the dry samples, kPa

In a study by Stroup-Gardiner, et al [8], three Minnesota Department of Transportation (Mn/DOT) projects were selected to analyze stripping, cohesion problems, and mix design problems. It was concluded that moisture-related pavement distress in Minnesota could be associated with one of three mechanisms:

1. True stripping
2. Loss of binder cohesion
3. Low initial mixture tensile strength

Stroup-Gardiner, et al [8] concluded that mixture properties can be improved by increasing film thickness, reducing air voids, and minimizing the use of marginal aggregate sources.

VMA Change

During the mix design phase, asphalt-aggregate mixtures are designed to achieve a set of minimum standards with regard to volumetrics. A minimum VMA requirement is indicative of this type of standard. In the production of HMA pavements, a phenomenon referred to as *VMA collapse* may occur. VMA collapse describes the situation in which the VMA in a produced HMA is lower than the VMA determined during mix design. Due to the ramifications of producing a pavement that does not achieve desired VMA specifications, a need exists to research the causes of the collapse and determine ways to adjust for it. If the causes of VMA collapse cannot be sufficiently controlled, then the design must be adjusted; VMA collapse must be predicted to some extent so that the design VMA is equal to the production VMA. Also, investigation into the effect of VMA collapse on asphalt film thickness is needed.

OBJECTIVES

The main objective of this research was to determine if a VMA collapse had occurred in any of ten paving sites across the state of Minnesota in 1996, investigate the possible causes of the VMA collapse if it had occurred, and recommend solutions that might aid in resolving the problem. The investigation focused on the factors that contribute to the variability in VMA, including material properties and plant production characteristics, with an emphasis placed on the factors that might decrease VMA. Variability in the determination of VMA was also explored. Incorporation of the concept of asphalt film thickness and its relationship to VMA was also investigated.

SCOPE

This research program investigated the phenomenon of VMA collapse and the parameters which influence VMA. The emphasis of the study was to analyze a paving project mix design, the physical properties of the project aggregate before production, and the material properties of the mix after it was obtained from behind the paver. Possible correlations between the analysis of the materials both before and after production with the preliminary results from the mix design phase obtained from Mn/DOT were explored.

Ten paving projects from around Minnesota were included. Supplies of all aggregate used in the mix during construction, as well as any recycled asphalt pavement (RAP), were sampled. Behind-the-paver samples were obtained during construction for each project for analysis. Detailed information regarding plant production procedures was recorded.

The following tests were performed on the aggregates [9]:

1. ASTM C 127 Specific Gravity and Absorption of Coarse Aggregate
2. ASTM C 128 Specific Gravity and Absorption of Fine Aggregate
3. ASTM C 136 Sieve Analysis of Fine and Coarse Aggregates
4. ASTM D 5821 Percentage of Fractured Particles in Coarse Aggregate (Coarse Aggregate Angularity)
5. ASTM D 4791 Flat or Elongated Particles in Coarse Aggregate
6. ASTM C 1252 Uncompacted Void Content of Fine Aggregate (Fine Aggregate Angularity)
7. ASTM D 2419 Sand Equivalent Value Fine Aggregate

In addition, the behind-the-paver samples were compacted according to mix design specifications, and the air void content, VMA, and film thickness were determined and compared to original specifications. Testing on the compacted specimens included dry indirect tensile strengths and retained tensile strengths after moisture conditioning. An ignition oven was used, following the Mn/DOT standard test method described in Chapter 3, to determine asphalt content of the behind-the-paver samples. The aggregate remaining after ignition was analyzed to determine the extent of aggregate degradation occurring between mix design and construction. Comparisons were made between mix design gradations and aggregate gradations following ignition.

Final conclusions and recommendations are based on a summary of the data accumulated during the research.

CHAPTER 2
LITERATURE REVIEW

FACTORS THAT CONTRIBUTE TO VARIABILITY IN VMA

The importance of adequate VMA in an asphalt mixture was presented in Chapter 1. To analyze the contribution of VMA to pavement durability, it is important to understand the parameters of an HMA that relate to the determination of VMA. Certain characteristics of an HMA mixture and its components can change the VMA and film thickness. These characteristics are summarized in Table 2.1.

Table 2.1 Factors that Affect the VMA of an HMA

Factor	**Effect on VMA**
Aggregate Gradation	Dense gradations decrease VMA
Aggregate Shape	More rounded aggregates decrease VMA
Aggregate Texture	Smooth or polished aggregates decrease VMA
Asphalt Absorption	Increased asphalt absorption results in lower effective asphalt content and lower VMA (for the same level of compaction)
Dust Content	Higher dust contents increase surface area, decrease film thickness, and tend to lower VMA
Baghouse Fines/Generation of Dust	Increased fines and dust increase surface area, decrease film thickness, and tend to lower VMA
Plant Production Temperature	Higher plant production temperatures decrease asphalt binder viscosity, which results in more asphalt absorption, lower effective asphalt binder and lower VMA
Temperature of HMA during Paving	Higher temperatures during paving create soft mixtures, lower air voids, and lower VMA
Hauling Time	Longer hauling times allow for increased asphalt absorption, lower effective asphalt content and lower VMA
Aggregate Handling	More steps in aggregate handling increases potential for aggregate degradation, resulting in an increase in fines, and lower VMA

Material Properties

The extent to which an HMA mixture can be compacted is related to aggregate gradation, aggregate surface characteristics, amount of asphalt, and asphalt absorption by the aggregate. Aggregate gradation is the size distribution of the aggregate particles, including the amount of material passing the 75-µm sieve (dust content). Aggregate surface characteristics include the shape, angularity, and surface texture. Aggregate absorption of asphalt binder is dependent on the aggregate porosity, and pore size, as well as the viscosity of the asphalt binder.

Aggregate Gradation

When selecting an aggregate for an HMA mixture, the initial focus is on the aggregate gradation. The surface characteristics of the aggregate are analyzed after the gradation has been proven to be suitable. Two factors relating to aggregate gradation having the most influence on VMA are density, or the ability of the aggregate particles to pack together, and the aggregate surface area.

Density

Figure 2.1 illustrates a 0.45 power plot of an aggregate gradation developed by the Federal Highway Administration (FHWA) and reported by Goode and Lufsey [10]. It is used to estimate how densely a given aggregate mixture will compact. It consists of the particle size raised to the 0.45 power on the x-axis and the percent passing each sieve size plotted on an arithmetic y-axis. A line drawn from the origin of this plot through the nominal maximum aggregate size is estimated as the maximum density line for any given aggregate (The nominal maximum aggregate size is defined as the first sieve to retain between 0 and 10% of the aggregate). Goode and Lufsey [10] demonstrated that an aggregate having a gradation that produces a straight line on a 0.45 power gradation graph will have the maximum achievable density, and subsequently (as shown by Nijboer [11]) the lowest air void content and the lowest VMA in an HMA mixture. Deviating from the maximum density line in either the fine or the coarse direction will tend to increase the VMA of the compacted mixture (see Figure 2.1).

However, Huber and Shuler [12] note that significant confusion exists concerning different methods used to draw aggregate gradation "maximum" density lines. Closely related to maximum density lines, and also in debate, is the definition of nominal aggregate maximum size.

For the purposes of this paper, the definitions used for the maximum density line and the aggregate nominal maximum size are stated above.

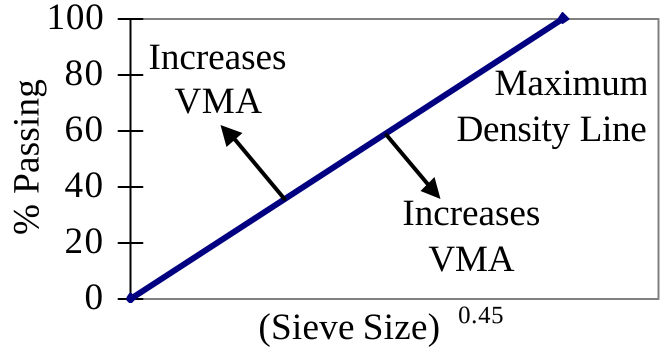

Figure 2.1 Maximum Density Line Related to VMA

Aschenbrener and MacKean [13] defined "distance" as the absolute value of the difference in percent passing between the actual gradation and the maximum density line at a given sieve size. This value characterizes the actual deviation from the maximum density line. Increasing the sum of the distances between a gradation and the maximum density line will tend to increase the VMA. Aschenbrener and MacKean [13] showed that the distances for the 2.36-mm and smaller sieve sizes had the greatest effect on the VMA of the compacted mixture. Kandhal, et al [14] determined that the percent passing the 475-mm (No. 4), 2.36-mm (No. 8), 1.18-mm (No. 16), 0.600-mm (No. 30), 0.300-mm (No. 50), 0.150-mm (No. 100), and 0.075-mm (No. 200) sieves were the most practical predictive variables for VMA. Lefebvre [15] described attempts to correlate Aschenbrener and MacKean's "distance" to VMA. These two variables did not correlate well due to the many other factors that affect VMA. Consequently, the only way to be certain of the VMA of a mix is to produce a sample and measure the parameters from which VMA is calculated.

Surface Area

The total surface area of an aggregate mixture is dependent primarily on the gradation. Aggregate gradations that deviate above the maximum density line are defined as "fine" while those that deviate below the maximum density line are "coarse". An analysis of Equation 1.2 indicates that as particle diameter decreases, the surface area per unit mass increases. Using this equation, a simple estimate of surface area can be determined for the purpose of comparing different aggregate gradations. It should be noted that the results of this equation are approximate.

The calculated surface area of a given aggregate gradation is related to the asphalt film thickness. As stated previously, deviating from the gradation maximum density line in either the fine or coarse direction will tend to increase the VMA of the compacted mixture for a given asphalt content. However, increasing the fine portions of an aggregate also increases the overall surface area of the aggregate. This causes the asphalt film thickness to decrease, possibly leading to durability problems, as the same amount of asphalt binder is spread over a larger surface.

Material Passing the 75 -mm Sieve (Dust Content)

The amount of material passing the 75-μm sieve has a significant effect on HMA properties. Anderson [16] listed ways the dust or fine fraction can affect HMA:

1. Stiffening the asphalt binder
2. Extending the asphalt binder
3. Altering the moisture resistance of the mix
4. Affecting the aging characteristics of the mix
5. Affecting the workability and compaction characteristics of the mix

Fines that are less than 2 μm in diameter may become part of the asphalt, causing further hardening.

An increase in the dust proportion will generally decrease the VMA. Due to the relationship between particle diameter and surface area, increasing the amount of material passing the 75-μm sieve will result in a greater total surface area of the aggregate blend. This results in a thinner average film thickness, lower effective asphalt content, and could lower the VMA.

Aggregate Surface Characteristics

Two primary aggregate surface characteristics are the shape of the aggregate particles and the aggregate surface texture. Aggregate shape is related to angularity (the number of crushed faces on the aggregate particles) and the extent to which the particle is flat or elongated. Aggregate surface texture is a qualitative description of the degree of roughness on the particle surface.

Aggregate Shape

Generally, it is desirable to have aggregate particles with a somewhat angular shape in asphalt mixtures. Flat and elongated particle shapes are undesirable. In compacted mixtures, particles that are cubic in shape exhibit greater interlock and internal friction, resulting in greater mechanical stability than flat and elongated particles. According to Roberts, et al [2], mixtures with flat and elongated particles tend to densify under traffic, ultimately leading to rutting due to low voids and plastic flow. Higher quantities of crushed aggregates and more angular crushed aggregates will generally produce a higher VMA. The increase in VMA results from the angular aggregates creating more void space during compaction due to the increased number of sharp edges and fractured faces. Since VMA includes air voids and the effective asphalt content, increasing the air voids in the compacted mixture will increase the VMA and allow more asphalt into the mix.

Aschenbrener and MacKean [13] determined that higher quantities of rounded, natural sands and more rounded aggregates will generally result in a lower VMA. Round particles have the potential to fit very densely together because the smoothness of the surface and the lack of angular edges, which together reduce the internal friction. The decrease in internal friction and the ability of uncrushed aggregates to compact more easily into a dense arrangement reduces void space, which ultimately leads to a reduction in VMA.

Aggregate Surface Texture

Aggregate surface texture is qualitatively described by the degree to which the aggregate is polished or dull. Polished aggregate particles have smooth surfaces. As with rounded aggregate particles, this contributes to a lack of internal friction, the ability to compact in a dense arrangement, and a decrease in void space and VMA. Aggregates with rough surface textures have a high level of internal friction, higher air void contents, and higher VMA. In addition, rougher aggregates also have the potential for improved adhesion of the asphalt binder to the aggregate due to the jagged surface texture.

Another aspect of aggregate surface texture is the amount of surface area. Generally, the rougher the surface is, the greater the surface area. Therefore, rougher aggregates tend to require more asphalt binder to coat the individual particles. This decreases the overall film thickness without necessarily decreasing the VMA as the effective asphalt content remains the same. However, durability problems can arise from the reduction of film thickness, regardless of the VMA of the mixture.

Absorption

The amount of asphalt binder absorbed by an aggregate is dependent on the porosity, void volume, and pore size of the aggregate, as well as the viscosity of the asphalt binder. Porosity is directly influenced by the void volume and pore size. Aggregates with larger pore sizes allow for increased asphalt binder absorption. However, aggregates with small pores have the potential for selective absorption of the lighter asphalt binder fractions. This accelerates premature aging and can create a lack of durability.

Asphalt binders that are more viscous tend to limit absorption by aggregates due to a lack of fluidity and an inability to fill aggregate pores. Alternatively, asphalt binders that are not as viscous have a greater ability to fill aggregate pores.

Aggregate Porosity

All mineral aggregates have some porosity and have the potential to absorb asphalt binder. The absorption may occur continually or at any point during mixing at the HMA plant, storage in a silo, hauling time in trucks, or in service. Kandhal and Maqbool [17] stated that although some absorption may lead to improved strength in a compacted mixture through particle adhesion, the portion of the asphalt that is absorbed is no longer available as binder. Therefore, aggregates with a large void volume and/or pore size will have a reduced effective asphalt content. This will lead to a decrease in VMA provided the air voids remain constant.

In the past, Mn/DOT assumed that all aggregates had an asphalt absorption of one percent (measured as percent of asphalt binder absorbed by weight of aggregate), regardless of the aggregate source. Actual aggregate porosity was not taken into consideration in this assumption. Porous aggregates were assumed to absorb the same amount of asphalt binder as relatively non-porous aggregates. As a result, aggregates with absorption values greater than one percent caused the VMA to be underestimated. This resulted in a lower film thickness and a subsequent lack of durability. Conversely, aggregates with absorption values less than one percent resulted in over-asphalted and potentially unstable mixtures.

Asphalt Binder Viscosity

Asphalt binder absorption is also dependent on viscosity. Viscous asphalt binders are not able to penetrate aggregate pores as readily as more fluid asphalt binders. As temperatures increase, the viscosity of asphalt decreases. Consequently, at higher temperatures, asphalt is more readily absorbed by aggregates.

Kandhal and Maqbool [17] list the following HMA problems that may result from the use of incorrect asphalt absorption values:

1. Incorrect computation of percent air voids, VMA, or voids filled with asphalt, which may lead to mixtures lacking durability or stability.
2. Low effective binder content may lead to raveling, cracking, or stripping.
3. Possible premature age hardening and low temperature cracking as a result of changes in asphalt properties due to selective absorption.
4. Construction problems such as segregation and tender mixes.

It has also been shown that the amount of asphalt absorbed by aggregates in the field may be substantially higher than in the laboratory. This may be due to prolonged mixing, storing, or hauling times, or higher temperatures during construction. Anderson, et al [18] cautioned that the potential absorption of asphalt in the field should be accounted for if realistic volumetric parameters are to be calculated for the completed mat. In view of all the consequences above, and especially in the determination of VMA and film thickness, asphalt absorption needs to be accurately estimated.

Plant Production Characteristics

HMA plant production characteristics that can influence both the VMA and the film thickness of a completed mat are: the use of baghouse fines and the generation of fines, plant production temperatures, and hauling time.

Use of Baghouse Fines / Generation of Fines (-75-mm)

In the past decade, stricter air pollution controls have created a need for dust collection systems, such as baghouse filters, during the production of asphalt paving mixtures. Kandhal [19] stated that asphalt plants frequently use the collected baghouse fines as a partial or total replacement for mineral filler in paving mixtures in order to conform with the pollution control regulations and avoid accumulation of excess fines.

When the dust content is increased in the mixture, the potential exists for problems similar to those previously reported under the "dust content" of the Material Properties section. If the amount of asphalt binder in the mix remains constant, the addition of baghouse fines will create more surface area and subsequently, a lower film thickness on the aggregate particles.

Fines are also generated during the handling of the aggregate before it reaches the plant. At the beginning of the production process, aggregate is loaded into a truck from a quarry or pit and hauled to the production plant site. During this period, the potential exists for fines to be generated due to abrasion that occurs during handling. As the individual particles rub against one another or against equipment, fines are able to break off of the larger particles.

The same thing can occur at the plant site as the aggregate is separated and recombined. Abrasion occurs at many points during this process and more fines can be created. As the aggregate is combined with the asphalt binder, again the potential exists for abrasion as the aggregate is still being handled and particles are in contact.

Abrasion can cause an increase in fines and surface area and a reduction in film thickness. The aggregate particles can also change shape. Abrasion tends to round more angular particles. As mentioned previously, rounded aggregate particles reduce the internal friction and result in a dense arrangement, consequently lowering the air void content and VMA.

Plant Production Temperatures

HMA plants produce mixes at temperatures ranging from 120 to 165 °C (250 to 325 °F). As the temperature of the mixture increases, the asphalt binder viscosity decreases and the potential for the asphalt binder to be absorbed into the aggregate increases. Therefore, if a plant produces an

HMA at temperatures that are higher than needed for compaction, the aggregate will absorb more asphalt binder, resulting in a lower effective asphalt content.

Hauling Time

After a mix is produced at an HMA plant, the mix is transported to the paving site. At this point, the mix is unloaded into the paver or on the roadway in front of the paver. The paver forms the mix into a paved mat. The temperature of the mix at the end of this process is important in determining if adequate compaction can be obtained.

Compaction has a great effect on the strength and durability of an HMA pavement. The main objective of pavement compaction is to achieve density so that the pavement will gain the desired strength and durability. The compactibility of HMA is related to the viscosity of the asphalt binder, which changes with temperature. McLeod [3] described a 1,000-fold increase in asphalt viscosity as the temperature drops from 135 to 57 °C (275 to 135 °F) and a ten-fold increase in resistance to compaction as the mix temperature dropped from 135 to 63 °C (275 to 145 °F). If the temperature behind the paver is too cool, the mat will not achieve adequate density, resulting in poor stability and durability.

Hauling time is the interval between the loading of the HMA from the plant into the truck until the time the mix is run through the paver and compacted. A long hauling time can result in a mix that is too cool for adequate placement and compaction.

VARIABILITY IN THE DETERMINATION OF VMA

As can be deduced from the Equation 1.1, the overall precision of the VMA calculation depends on the precision of the bulk specific gravity and asphalt absorption calculations for the aggregate, the bulk specific gravity of the compacted mixture, and the effective asphalt content. All laboratory tests performed on similar materials will have some variability due to inherent random testing errors. Other causes of variability are sampling procedures, operator experience, equipment, and environmental factors such as temperature and humidity. Due to the importance of repeatable estimates of VMA, it is essential to determine the variability of various test procedures used in standard specifications and to set acceptable specification limits for these test properties. The procedure is described in ASTM C 802 (Practice for Conducting an

Interlaboratory Test Program to Determine the Precision of Test Methods for Construction Materials) [9].

Specific Gravity and Absorption of Coarse Aggregate

ASTM C 127 [9] is used for measuring the specific gravity and absorption of the portion of the aggregate that is retained on the 4.75-mm (No. 4) sieve. The greatest source of variation in this test method may be the difficulty in determining the saturated-surface-dry (SSD) condition. The SSD condition is defined as the moisture content at which the aggregate surface pores are filled with water and the surface is dry. The aggregate sample is to be dried until "all visible films of water are removed". At this point, the sample is considered to have reached a SSD condition. This is a subjective evaluation based on the opinion of the person who is performing the test. In order to reduce the variation of the results of this test, there is a need to reduce the subjectivity of estimating this parameter.

A frequently used rule-of-thumb is to dry the aggregate until it has a dull finish. This means that the aggregate no longer has a shiny surface, which indicates the presence of surface moisture. However, there is still some subjectivity in determining the SSD condition. In particular, this concept may apply to gravel, but may not be as suitable for a crushed limestone which does not achieve a shiny finish when wet. Some aggregates change color as they dry and it may be possible to determine the saturated-surface dry condition of the aggregate by examining color changes. However, it is difficult to define what the color should be when the aggregate is saturated-surface-dry, as different aggregate sources produce different colors as they dry. Krugler, et al [1] described a method of defining the SSD condition color that is consistent for all aggregate types. This method involves comparing a drying test sample to an oven-dry sample of the same material. The SSD condition is estimated as the point when the test sample has the same color as the oven-dry comparison sample.

Absorption capacity also contributes to the ability to achieve repeatable test results. Krugler, et al [1] showed that absorptive aggregates tend to have increased variability compared to nonabsorptive aggregates. The mineral composition of the aggregate also can play a role in test result variability. Some mineral aggregates, such as limestone, tend to have increased variability compared to gravel or granite. Aggregate surface area can also contribute to the ability to achieve repeatable test results. Finer aggregate gradations or angular aggregate both have increased surface area, as discussed previously.

According to ASTM C 127 [9], the acceptable variability, in terms of standard deviation, for coarse aggregate bulk specific gravity is 0.009 for within-lab precision and 0.013 for between-lab precision.

Specific Gravity and Absorption of Fine Aggregate

ASTM Test Method C 128 [9] is used for measuring the specific gravity and absorption of fine aggregate or the portion of the aggregate passing the 4.75-mm (No. 4) sieve. As with coarse aggregates, the greatest source of variation is in determining when an aggregate has reached the SSD condition. The variability is greater than that for coarse aggregate due to the greater complexity in determining the SSD condition.

To determine the specific gravity of fine aggregate, the material is oven dried overnight, and then saturated with water for 24 hours. Following saturation, the aggregate is dried to an approximately SSD condition, weighed, and then oven-dried and weighed again. ASTM C128 [9] prescribes the cone test for the determination of the SSD condition. This is a procedure in which the aggregate is placed into a metal mold in the form of a frustum of a cone. The mold is placed on a firm, nonabsorbent surface with the large diameter down, and is filled to overflowing with a portion of the fine aggregate. The aggregate is lightly tamped with a metal tamper 25 times, with each drop starting about 5 mm above the surface of the aggregate. The mold is then removed from the sample. The SSD condition is defined as the point at which approximately 25 percent of the top surface of the cone falls.

The determination of the SSD condition is subjective; based upon the opinion of the person performing the test. It is difficult to accurately determine at which point 25 percent of the top cone surface falls. Variability will result from uneven tamping, overcompacting the sample, and allowing portions of the sample that have dried to be recombined with the portion of the sample that is still either wet or SSD. Also, the test method itself states that "some angular fine aggregate or material with a high proportion of fines may not slump in the cone test upon reaching a surface-dry condition." Since many bituminous mixture aggregates are angular in nature, Kandhal and Lee [20] and Root [21] have stated that this test procedure needs improvement to consider all fine aggregates.

As mentioned above with ASTM Test Method C 127, other factors can contribute to the variability in the determination of specific gravity and absorption of fine aggregate. In general, the same factors that contribute to the variability of coarse aggregate test results will also contribute to the variability of fine aggregate test results. Krugler, et al [1] identified absorptive aggregates, aggregate composed of limestone, and aggregates with large surface areas as those most likely to increase the variability of test results.

According to ASTM Test Method C 128, the acceptable variability for fine aggregate specific gravity is a standard deviation of 0.011 for within-lab precision and 0.023 for between-lab precision.

Specific Gravity of Compacted Specimens

The bulk specific gravity of a compacted asphalt mixture is usually determined using ASTM C 2726 [9]. Variations from the procedure can introduce significant variability in the specific gravity calculations. The procedure requires that the compacted mix be weighed in air, submerged, and SSD after being submerged. The SSD condition is obtained by blotting the surface with a slightly damp towel after the submerged weight is measured. Equation 2.1 is used to calculate the bulk specific gravity.

$$G_{mb} = \frac{A}{B - C} \tag{2.1}$$

Where:

G_{mb} = Bulk specific gravity of compacted specimen

A = Oven dry mass (g)

B = SSD mass (g)

C = Submerged mass (g)

ASTM C 2726 specifies water temperature, time for compaction ,dimensions of the compacting hammer, and other conditions. According to ASTM C 2726, the acceptable variability for bulk specific gravity is a standard deviation of 0.0269 for within-lab precision and 0.124 for between-lab precision. To attain these levels of precision it is necessary to follow the procedures outlined in the test method.

A source of variation in density is aggregate degradation during compaction. Degradation is dependent on the type of aggregate. The amount of degradation may vary from the laboratory to field compaction.

Maximum Density of Hot-Mix Asphalt

The theoretical maximum specific gravity of an asphalt mixture can be measured using ASTM D2041-95 Standard Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures [7], commonly known as the Rice test. For this test, a sample of the mix is broken up into a voidless mass of particles less than 4.75 mm (0.25 in.) in size. The sample is placed in a calibrated container. The weight and volume of the voidless mix are then used to calculate the maximum specific gravity as shown in Equation 2.2.

$$G_{mm} = \frac{A}{A - C} \tag{2.2}$$

Where:

G_{mm} = Theoretical maximum specific gravity

A = Mass of oven dry sample in air, g

C = Mass of water displaced by sample at 25 °C (77 °F), g

The test can be run either by weighing the sample in a container that can be submerged or by placing the sample in a pycnometer (container with calibrated volume) filled with water.

Knowing the bulk specific gravity of the compacted mixture and the maximum specific gravity, the air void content of the compacted mixture can be calculated using Equation 2.3.

$$V_a = \left(1 - \frac{G_{mb}}{G_{mm}}\right) \times 100 \tag{2.3}$$

Where:

V_a = Air voids in the compacted mix, percent

G_{mb} = Bulk specific gravity of the compacted mix

G_{mm} = Theoretical maximum specific gravity of the mix

Two other important properties of a mix can also be calculated if the composite aggregate specific gravity and asphalt binder specific gravity are known. These are:

1. The percent asphalt absorbed by the aggregate
2. The effective asphalt content of the mix. The effective asphalt content is the value used to estimate the asphalt film thickness and VMA.

The percent of asphalt binder absorbed can be estimated from the theoretical maximum specific gravity (G_{mm}) of the HMA. The following three equations were provided by the Asphalt Institute [22]. In Equation 2.4, the effective specific gravity (G_{se}) of the aggregate is calculated. Equation 2.5 is then used to calculate the absorbed asphalt (P_{ba}). The effective asphalt content can then be calculated using Equation 2.6.

$$G_{se} = \frac{100 - P_b}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}} \qquad (2.4)$$

Where:

G_{se} = Effective specific gravity of aggregate

G_{mm} = Maximum specific gravity of paving mixture (no air voids)

P_b = Asphalt content, percent by total mass of mixture

G_b = Specific gravity of asphalt

$$P_{ba} = 100 \times \frac{G_{se} - G_{sb}}{G_{sb} G_{se}} \times G_b \qquad (2.5)$$

Where:

P_{ba} = Absorbed asphalt, percent by mass of aggregate

G_{se} = Effective specific gravity of aggregate

G_{sb} = Bulk specific gravity of aggregate

$$P_{be} = P_b - \frac{P_{ba}}{100} \times P_s \qquad (2.6)$$

Where:

P_{be} = Effective asphalt binder content, percent by weight of aggregate

P_b = Asphalt binder content, percent by total weight of mix

P_{ba} = Percent of asphalt absorbed by the aggregate, percent by mass of aggregate

P_s = Total aggregate content, percent by total weight of mix

Factors such as mixing and placing temperature and time of storage can have an effect on the percent asphalt absorbed for a given mixture.

Asphalt Content

The asphalt content of an asphalt mix can be determined by one of three methods:

1. Mix design
2. Solvent extraction
3. Ignition oven

For the purposes of this research, the design asphalt content was used to determine the mix design VMA, and the asphalt content following ignition was used to determine the VMA of the sampled mixture. Variability of asphalt content using an ignition oven is discussed in greater detail in the following chapter.

EFFECT OF VARIABILITY ON VMA

To calculate VMA, it is necessary to determine the bulk specific gravity of the aggregate and the compacted mixture, the theoretical maximum specific gravity of the mixture, and the asphalt content. The greater the variability among these three properties, the greater the variability of the VMA. Krugler, et al [1] stated that the variability of measured bulk specific gravity of the aggregate and compacted specimens can significantly affect the variability of VMA, regardless of whether the same asphalt content is used

VMA CHANGE

Kandhal, et al [14] described factors that may cause a decrease in VMA. These include:

1. Aggregate gradation
2. Amount of rounded aggregate particles
3. Aggregate absorption of asphalt binder
4. Dust content
5. Plant production temperature
6. Hauling distance

A greater emphasis was placed on aggregate gradation, aggregate absorption, plant production temperatures, and hauling distances. More limited research was conducted on the amount of rounded aggregate particles and dust content.

In order to control VMA, factors contributing to the change in VMA must be identified and minimized to establish realistic expectations. VMA collapse is defined as the decrease in VMA realized from a field mix during construction compared to the VMA measured in the laboratory during the initial design of the mixture. Therefore, all of the above parameters need to be quantified to study the problem of VMA variability. Once the parameters are quantified, steps can be taken to reconcile the differences between mix design and production.

SUPERPAVE CONSENSUS AGGREGATE PROPERTIES

The Asphalt Institute [22] developed a manual for the Superpave (Superior Performing Asphalt Pavements) under contract with the FHWA. Superpave is an asphalt mix design procedure developed as part of the Strategic Highway Research Program (SHRP). Although the projects evaluated in this study were not Superpave projects, aggregate tests specified in the Superpave mix design procedure were conducted on the stockpile aggregates to further evaluate the performance of the mixtures. Mn/DOT [23] has adapted this procedure in the 2360 Specification for HMA pavements. The results of the aggregate tests must meet criteria that are based on the design traffic level for the pavement. The Mn/DOT criteria are shown in Table 2.2.

Coarse Aggregate Angularity (CAA)

The Asphalt Institute [22] defines this test as a means of evaluating a mixture's internal friction and rut resistance. CAA is measured using ASTM D 5821 [9]. The results are presented as two values separated by a slash (/). The first number represents the percent by mass of particles that have one or more fracture faces, and the second number represents the percent that have two or more fractured faces. A fractured face must have an area of at least 25 percent of the cross-sectional area of the particle in order to be considered.

Table 2.2 Mn/DOT 2360 Aggregate Criteria for Wear Courses (Depth £ 100 mm)

Traffic Level (Millions of ESALs)		Level 1 (under 0.3)	Level 2 (0.3 to 1)	Level 3 (1 to 3)	Level 4 (3 to 10)	Level 5 (10 to 30)	Levels 6&7 (over 30)
CA Angularity (% Particles with Fractured Faces)	At least 1 FF	55 or higher	75 or higher	75 or higher	85 or higher	95 or higher	100
	At least 2 FF	-----	-----	-----	80 or higher	90 or higher	100
CA Flat or Elongated %		-----	-----	10 or lower	10 or lower	10 or lower	10 or lower
FA Angularity (% Uncompacted Voids)		-----	40 or higher	40 or higher	45 or higher	45 or higher	45 or higher
FA Sand Equivalent		40 or higher	40 or higher	40 or higher	45 or higher	45 or higher	50 or higher

Fine Aggregate Angularity (FAA)

This test is also used to evaluate a mixture's internal friction. FAA is measured using ASTM C 1252 [9]. An oven dried sample of the aggregate is sieved and re-batched into a specified gradation before testing . The sample is then placed in a funnel and allowed to fall from a specified height into a calibrated cylinder. The top is leveled with a straight edge, and the mass of the aggregate is determined. Using the known mass, volume and bulk specific gravity of the aggregate, the volume of voids can be calculated. It is assumed that an aggregate composed of rounded particles will have a lower internal friction, and therefore a lower volume of voids.

Flat or Elongated Particles in Coarse Aggregate

The Asphalt Institute [22] stated that flat or elongated particles are undesirable in an HMA mix because they tend to break during compaction. The procedure is described in ASTM D 4791 [9]. This value indicates the percent by mass of aggregate particles that fail a length-to-thickness test. Mn/DOT [23] uses a 3:1 criteria (the particle fails if the length is more than three times the thickness). A proportional caliper device is used in which the larger gap is three times as wide as the small gap. The large gap is set to the maximum dimension of the particle, and the smaller gap is used to evaluate the minimum dimension of the particle.

Sand Equivalent Value of Fine Aggregate

This parameter is measured using ASTM D 2419 [9]. It is a measure of the ratio of sand to clay in a fine aggregate. The test is conducted by suspending the aggregate in a fluid and allowing it to settle in a graduated cylinder. The sand settles quickly, and the clay settles more slowly, forming a separate layer above the sand. The Sand Equivalent value is the ratio of the sand column height to the total height of the column (sand + clay), expressed as a percentage.

MOISTURE SENSITIVITY TESTING

A test that is used to identify potential durability problems in HMA is ASTM D 4867 Standard Test Method for Effect of Moisture on Asphalt Content Paving Mixtures [7], commonly known as the Lottman test. An even number (at least six) of specimens are compacted to 7 $\pm$ 1 % air voids. The specimens are then sorted into two subsets so that the average air voids of two subsets are approximately equal. A subset to be tested dry was stored while the other subset underwent moisture conditioning.

The subset to be moisture conditioned is partially saturated with distilled water at room temperature using a vacuum chamber. The volume of absorbed water is obtained by subtracting the air dry mass of the specimen from the saturated surface-dry mass of the partially saturated specimen. The degree of saturation is calculated by dividing the volume of the absorbed water by the volume of the air voids and express as a percentage. The percent saturation must be between 55 and 80. If this value is less than 55, the specimen is resaturated. If the value is greater than 80, the specimen must be discarded.

The partially saturated specimens are conditioned by soaking them in distilled water at 60 $\pm$ 1.0 °C (140 $\pm$ 2.0 °F) for 24 hours. The temperature of the moisture-conditioned and the dry subsets is then adjusted by soaking the specimens in a 25 $\pm$ 1.0 °C (77 $\pm$ 2.0 °F) water bath.

Each specimen is placed in an apparatus capable of applying a diametral load at a rate of 50 mm/min. while measuring the vertical load. The load is applied until a drop in load is detected (specimen failure). The tensile strength is defined as the maximum stress measured in the specimen prior to failure.

The tensile strength is calculated using Equation 2.7.

$$S_t = \frac{2P}{\pi t D} \tag{2.7}$$

Where:

S_t = Tensile strength, psi

P = Maximum load, lb.

t = Specimen height immediately before tensile test, in.

D = Specimen diameter, in.

The tensile strength ratio is a ratio of tensile strength of the wet subset to the tensile strength of the dry subset. It is calculated using Equation 1.4.

CHAPTER 3
FIELD DETERMINATION OF VMA

INTRODUCTION

As discussed in Chapters 1 and 2, VMA has been determined during the laboratory phase of HMA design for a number of years. The Asphalt Institute [4] defined criteria relating minimum VMA to maximum size of the aggregate have been used to assure appropriate film thickness for design voids in the total mix.

There has been some concern that the VMA attained in the field can be somewhat different than the laboratory measured VMA for a given mix.

Ten 1996 paving projects across Minnesota were used to study the variation between laboratory and field asphalt mixes. These projects are summarized in Table 3.1. They were designated according to the location of the Mn/DOT district office to provide a geographical reference. A map illustrating the approximate locations of the paving projects is shown in Figure 3.1. Detailed plant information for each project is provided in Appendix A.

Samples were collected by filling two to four large buckets with behind-the-paver material before compaction. The buckets were reheated for approximately one hour and separated into 1300-g samples at the Pavement Materials Lab at the University of Minnesota.

In addition to the behind-the-paver, or production samples, contractors sent the project aggregates used in the mixes to the University of Minnesota for analysis and provided plant information on the projects. From this and the production samples, three major sources of information from each project were used to determine possible changes in VMA. The three primary factors are:

1. Aggregate properties
2. Field mix properties
3. HMA plant production characteristics

Table 3.1 Summary of Paving Projects

Project No.	Name	District	Hwy	Description	Mix Type
1506-30	Bemidji I	2	92	Bituminous Overlay	31 Level
6007-10	Bemidji II	2	32	Bituminous Overlay	31 Level
7505-18	Detroit Lakes I	4	59	Regrades	32 Base
8408-48	Detroit Lakes II	4	75	Overlay/Widening	41 Binder
1906-40	Metro	Metro	52	Mill & Overlay	48 Level
2514-89	Rochester I	6	61	Mill & Overlay	48 Level
5503-28	Rochester II	6	14	Mill & Overlay	42 Wear
5209-58	Mankato	7	169	Bituminous Overlay	47 Binder
4201-36	Willmar I	8	14	Bituminous Overlay	41 Wear
4303-36	Willmar II	8	15	Bituminous Overlay	41 Wear

AGGREGATE PROPERTIES

Aggregates used for nine of the ten projects were provided to the University of Minnesota. Aggregate from Project 7505-18, Detroit Lakes I was not available. ASTM and the American Association of State Highway and Transportation Officials (AASHTO) standard tests performed on the aggregate were:

1. Bulk specific gravity and absorption of coarse aggregate (ASTM C 127/AASHTO T85)
2. Bulk specific gravity and absorption of fine aggregate (ASTM C 128/AASHTO T84)
3. Sieve analysis (ASTM C 136/AASHTO T27)
4. General aggregate mineralogy
5. Ignition calibration factor (Mn/DOT standard test method)

Bulk Specific Gravity and Absorption (ASTM C 127 and C 128)

Bulk specific gravity tests were performed on the aggregate and the recycled asphalt pavement (RAP) portions of the HMA mixture mainly for the bulk specific gravity (G_{sb}), used in Equation 1.1. RAP portions were heated in the ignition oven to remove the asphalt binder and were then treated as virgin aggregates. Once the bulk specific gravity of all the project aggregates and RAP were determined, a combined G_{sb} value was calculated using the mix design information

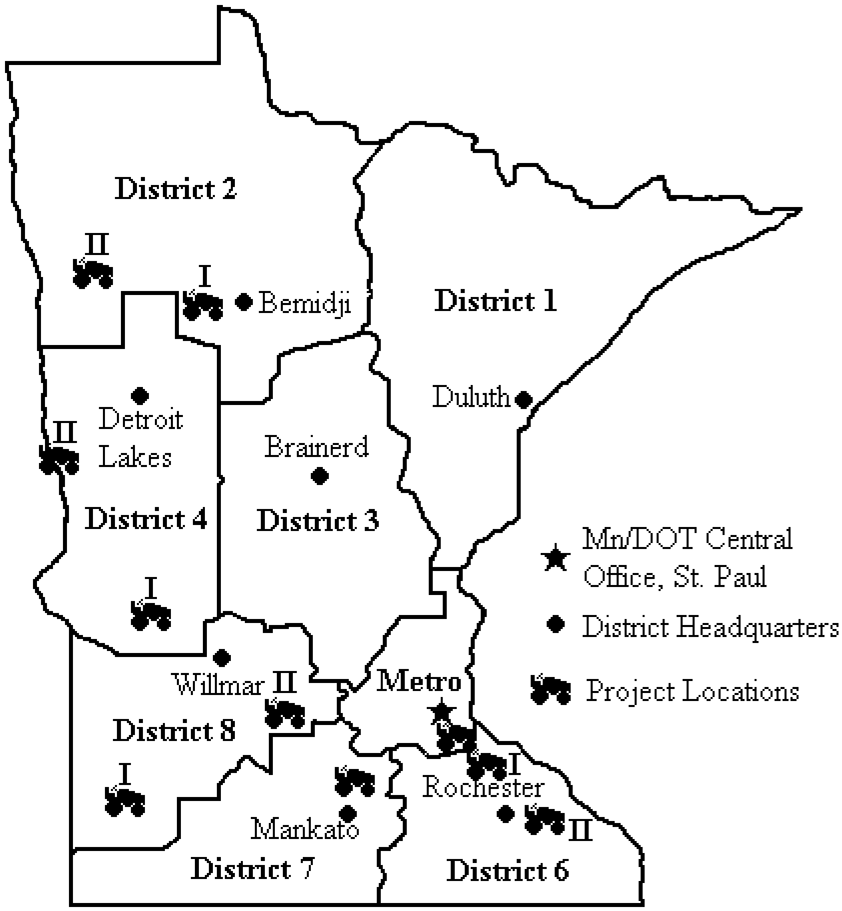

Figure 3.1 Map of 1996 Minnesota VMA Projects

for the project regarding aggregate proportions. This combined G_{sb} value was calculated according to the Equation 3.1 and used to determine the VMA from Equation 1.1:

$$G_{sb} = \frac{100}{\frac{P_1}{G_{sb(1)}} + \frac{P_2}{G_{sb(2)}} + \cdots + \frac{P_n}{G_{sb(n)}}} \tag{3.1}$$

Where:

G_{sb} = Combined aggregate bulk specific gravity

P_1, P_2,… = Percentage of aggregate (or RAP aggregate) 1, 2,… in the mixture

$G_{sb(1)}$, $G_{sb(2)}$,… = Bulk specific gravity of aggregate (or RAP aggregate) 1, 2,…

For the RAP aggregate, the standard Mn/DOT test procedure calls for initial extraction of the asphalt binder by solvent. After extraction, the remaining aggregate is treated as a virgin aggregate and placed in the ignition oven for determination of a calibration factor. However, asphalt binder extraction by solvent was not available for this research. Therefore, the RAP material was placed in the ignition oven for extraction and then the remaining aggregate was placed in the ignition oven again for determination of a calibration factor.

A source of error in determining a calibration factor for RAP material is encountered when performing initial extraction with the NCAT ignition oven. It may be assumed that there will be some fine aggregate material that will escape from the sample, rise with the heat, and be transported out of the oven along with the hot fumes and the asphalt binder. Therefore, when the aggregate is put into the ignition oven for calibration factor determination, the aggregate is no longer a complete representative sample, as a portion of the fine material is absent.

The Mn/DOT standard test procedure [24] calls for a 0.15% subtraction from the calibration factor determined for RAP aggregate if initial extraction is done by a solvent. This is intended to compensate for any retained asphalt binder. However, for this research, since the ignition oven was used for initial extraction, it was assumed that there was no retained asphalt binder and the 0.15% was not subtracted from the calibration factor.

Sieve Analysis (ASTM C 136)

Since many project aggregates required both a coarse and fine bulk specific gravity test, a basic, unwashed sieve analysis was performed on the aggregate to separate the aggregate into its coarse and fine portions. The bulk specific gravity of the separated portions was then determined. The gradation was also recorded during this sieve analysis to use as a comparison to the gradations determined by the contractor and Mn/DOT, if those data were available.

Aggregate Mineralogy

A determination of general mineralogy was made for each aggregate. This was done by a visual inspection of a representative sample. Limestone aggregate was identified using the HCl acid test. Basic descriptions of igneous, limestone, granite or quartz were used to classify the material. These descriptions were important in determining the type of aggregate available for use in different areas of Minnesota.

Ignition Oven Calibration Factor

In addition to the common aggregate properties listed above, a calibration factor was determined for each project aggregate using the Mn/DOT test method for calibrating virgin aggregate sources using an ignition oven. The calibration factor is a means of accounting for the organic components of the aggregate that are lost due to ignition as well as fine mineral material that is drawn through the exhaust pipe. Calibration factors are determined by placing an aggregate sample of a known mass in an ignition oven set at 575 °C (1067 °F) for 60 minutes. After the aggregate has cooled, the after-ignition mass is recorded and the calibration factor (C_n) is calculated from the following formula:

$$C_n = \frac{C - E}{C} \times 100 \tag{3.2}$$

Where:

C_n = Calibration factor, %

C = Initial weight of sample, g

E = Final weight of sample, g

If a project contained RAP, a sample of the RAP was placed in the ignition oven and the asphalt binder was burned off the aggregate. After allowing the aggregate to cool to room temperature, it was treated as a virgin aggregate source and the standard Mn/DOT test method for determining a calibration factor for virgin aggregate was used.

A composite aggregate mixture calibration factor (C_m) was determined for each project using the following formula:

$$C_m = \frac{100}{\frac{P_1}{C_1} + \frac{P_2}{C_2} + \cdots + \frac{P_n}{C_n}} \tag{3.3}$$

Where:

C_m = Combined aggregate calibration factor

P_1, P_2, …= Percentage of aggregates 1, 2, … in mixture

C_1, C_2, …= Calibration factor of aggregates 1, 2, …

One factor considered which affects VMA is aggregate gradation. This report illustrates how the aggregate might break down and the gradation might change from initial production at the plant to paving. To analyze the gradation after construction, the asphalt binder must be removed from the HMA mixture. This was done by using the Mn/DOT ignition oven test procedure for determining asphalt content from asphalt paving mixtures. Accurately determining the asphalt content calls for calibration factors to be determined for each aggregate source. This allows for the distinction between asphalt binder lost during ignition and fine aggregate material lost during ignition. The ignition test procedure is explained in greater detail in the following section.

FIELD MIX PROPERTIES

Each of the ten 1996 paving sites was visited during construction, and behind-the-paver samples were obtained. These samples were collected to provide information about the material characteristics of the HMA once it had been mixed, hauled, and run through the paver. These samples were collected from the paved mat behind the paver to give the best representation of the final mixture as it exists in the pavement.

Procedures and tests performed on the field samples were:

1. Marshall compaction (ASTM D 1559)
2. Bulk specific gravity of compacted specimens (ASTM D 2726)
3. Theoretical maximum specific gravity of mixture (ASTM D 2041)
4. VMA from Equation 1.1 and air void computation
5. Modified Lottman test (ASTM D 4867)
6. Asphalt binder extraction in an ignition oven (Mn/DOT Standard Test Procedure)

Marshall Compaction (ASTM D 1559)

For each project, the Mn/DOT mix design information was obtained. The behind-the-paver samples for each project were compacted according to the mix design that was used for that project and lift. Every project was compacted using the 50-blow Marshall Mix Design procedure.

The procedure consisted of heating the individual 1300-g samples in a 140 °C (285 °F) oven. The samples remained in the oven until they reached a temperature of 135 °C (275 °F), which was considered equivalent to a curing time of two to three hours. The standard Marshall Mix Design procedure was followed and a minimum of 18 samples were compacted for each project.

Bulk Specific Gravity of the Compacted Specimens

On the day following compaction, the bulk specific gravity of each sample was determined using ASTM D 2726 and Equation 2.1. Four height measurements were recorded and the average of these four was used as the height of the specimen.

Theoretical Maximum Specific Gravity / Air Void Computation

In order to determine the air void content of the compacted specimens, the theoretical maximum specific gravity of the mixture was determined using ASTM D2041 and Equation 2.2. A minimum of two maximum specific gravity tests were performed using 1300-g samples representative of the mixture used in compacting the 18 samples. If the two results were within the single operator precision of ASTM Test Method D2726 (0.035), the average of the two values was used. If the two results were not within the precision, a third test was run. When two of three results were within the precision, the average was taken of those two and used as the theoretical maximum specific gravity. The air void content (VTM) of each compacted specimen was determined according to the Equation 2.3.

VMA Calculation

VMA was computed for each compacted specimen using Equation 1.1. The combined aggregate bulk specific gravity determined from the project aggregates was used. The asphalt binder content was determined using the Mn/DOT ignition oven method.

Moisture Sensitivity Testing

A set of 18 compacted samples was tested to determine the TSR using ASTM D 4867 [7]. The samples used for this portion of the study were compacted at a reduced effort, with a target air void range of 7 ± 1 percent. The bulk specific gravity and air void content of each sample was determined. The 18 samples were separated and regrouped according to similar air void contents. The samples were then tested following the procedure outlined in Chapter 2.

Asphalt Binder Extraction Using Ignition Oven

The final test procedure performed on the behind-the-paver samples was extraction of the asphalt binder using an ignition oven, as presented briefly in the previous sections. Two individual 1300-g samples of the mixture were recombined and a 2000-g representative sample was placed in the ignition oven and tested at 538 °C (1000 °F). The composite aggregate calibration factor, as determined prior to the test, was input into the ignition oven controls to allow for an accurate determination of the asphalt content without incorporating fine material lost from the aggregate during ignition. The sample remained in the oven until its mass had stabilized, or for a maximum of 60 minutes.

Determination of Asphalt Binder Content

Using Equation 3.4, the asphalt binder content was determined and compared with the specified mix design asphalt binder content.

$$P_b = \frac{C - E}{C} \times 100 - C_m \tag{3.4}$$

Where:

P_b = Asphalt content, percent

C = Initial weight of sample, g

E = Final weight of sample, g

C_m = Aggregate mixture calibration factor, percent (from Equation 3.3)

Gradation Analysis

The aggregate that remained after the ignition oven test was tested to determine the change in aggregate properties from initial plant production to behind-the-paver sample. A sieve analysis was performed using ASTM C 136 to determine the change in gradation. This gradation was compared with the mix design gradation to determine if degradation had occurred.

Surface Area Analysis

Two aggregate surface area calculations using Equation 1.2 were made for each project. The surface area was determined using both the mix design gradation and the gradation determined after the field sample had been tested in the ignition oven. These two surface area values were compared.

The portion of material passing the 75-µm sieve was not included in the surface area analysis for the following reasons:

1. It is difficult to accurately determine the size distribution of the - 75-µm material.
2. An unknown proportion of the fines was lost during the ignition test.
3. The uncertainty regarding the quantity of fines combined with the large contribution of very small particles to the surface area calculation (see Equation 1.2) would result in erroneous surface area values.

Asphalt Film Thickness

Once the two surface areas were calculated, the asphalt film thickness was estimated using Equation 1.3, both for the mix design gradation and the post-construction gradation after ignition. The asphalt film thickness estimated in the preliminary mix design was compared to that on the produced HMA aggregate.

Since the material passing the 75-µm sieve was not included in the surface area estimation, it can be assumed that the estimated film thickness estimation was greater than the actual film thickness.

HMA PRODUCTION CHARACTERISTICS

Construction information and data which could affect the field mix VMA and other in-place HMA properties were documented Details are included in Appendix A. Eight of the ten project contractors provided information on the following factors:

1. Plant Characteristics
    a. Type
    b. Make (model no.)
    c. Capacity
    d. Dust collection system
2. Project Production Statistics
    a. Rate

b. Angle and speed of drum

c. Stockpile moisture content

d. Temperature

3. Transport

   a. Storage time

   b. Storage temperature

   c. Haul distance (maximum, minimum, average)

Plant Characteristics

All of the plants used for the ten projects were drum mix plants with baghouse dust collection systems. The baghouse fines were reintroduced into all the mixes. The first four plant characteristics (production rate, capacity, make of plant, and angle/speed of drum) were acquired to provide a general knowledge of the size and type of each paving project. The fifth plant characteristic, type of dust system, was the same for all ten paving projects: all projects used baghouse collection systems and all the fines were reintroduced into the mixture.

Project Production Statistics

The production rates and angle and speed of the drum were noted. The angle and speed should relate to the stockpile moisture content as adjustments are necessary if the moisture changes significantly. Aggregate moisture content variations may also adversely affect the results of asphalt contents measured using the ignition oven.

Plant and paving temperatures were recorded because of their potential effect on VMA. High plant production temperatures, as stated previously, can decrease asphalt binder viscosity and cause more asphalt absorption and a subsequent decrease in VMA. Lower paving temperatures can make the mix less workable and result in low density and higher VMA. However, the increase in VMA is simply due to the increase in air voids which can result in low stability and durability.

The final two factors were recorded to give an idea of how long the mixture cured before it was put through the paver. Storing mixtures for longer times at elevated temperatures can cause increased asphalt absorption, lower effective asphalt content, and lower film thickness. Plants that were located farther away from the paving site had more potential for VMA collapse caused by prolonged storage and increased asphalt absorption.

CHAPTER 4
RESULTS AND ANALYSIS

INTRODUCTION

This chapter provides the results and findings of the mixture durability research program. The ten projects are summarized in Table 3.1. Results of aggregate testing are listed in Appendix B, and aggregate gradations before and after construction are listed in Appendix C. Each project was analyzed for changes in mix parameters during design and construction for aggregate gradation, VMA, and asphalt film thickness. The projects were first separated according to the magnitude of the change in VMA and analyzed as three distinct groups. The potential causes of VMA changes were then reviewed to determine which contributed to the decrease in VMA.

Following the VMA analysis, asphalt film thickness was estimated to determine its relationship to the variability of VMA and to determine which parameters had the most effect on the film thickness. The impact of change between design and production aggregate gradations was also examined through the use of the 0.45 power gradation chart. Distances were measured between the project aggregate maximum density line and the aggregate gradation determined in mix design and following construction. The distances were measured by determining the difference between the percent passing of two gradations and adding the distances over a standard set of sieves.

The moisture sensitivity testing results are included to determine if low TSR values were caused by low film thickness or other parameters.

PROJECT DESCRIPTIONS

The ten projects used for this study are summarized in Table 3.1. The mixes were designated by layer (base, binder, level, or wear). The number in front of the mix type described the classification system used for the HMA mixtures in Minnesota and is related to the largest aggregate size, presence of recycled materials, and degree of fracture. Three mix types were used:

1. Type 31: Low volume, low aggregate angularity mixture

2. Type 32: Type 31 mix with RAP
3. Type 41: Medium volume and aggregate angularity mixture
4. Type 42: Type 41 mix with RAP
5. Type 47: High volume and aggregate angularity mixture
6. Type 48: Type 47 mix with RAP

The general aggregate mineralogy consisted of identifying the largest one or two components of the aggregate. If RAP was included, the percentage of RAP in the mix as a total percent of the aggregate was specified. Finally, the asphalt binder penetration was recorded.

VMA CHANGE

In order to determine if there had been a decrease in the VMA between mix design and construction, a design VMA was calculated using Mn/DOT mix design data and material properties. For post-construction VMA, the bulk specific gravity of the combined project aggregate was determined by testing the individual project aggregate samples and then using the aggregate proportions to calculate the combined aggregate bulk specific gravity. The material

Table 4.1 Post-Construction Results from Contractor, Mn/DOT, and the University of Minnesota

Project	Cumulative Aggregate Bulk specific gravity				VMA			
	Cont.	Mn/ DOT	U of MN	Std. Dev.	Cont.	Mn/ DOT	U of MN	Std. Dev.
Bemidji I	2.646	2.604	2.598	0.030	n/a	15.1	14.1	0.7
Bemidji II	2.603	2.594	2.612	0.006	n/a	15.0	15.0	0
Detroit Lakes I	2.548	2.486	n/a	0.044	n/a	n/a	14.4	--
Detroit Lakes II	2.588	2.550	2.553	0.027	n/a	13.9	13.6	0.2
Metro	2.692	2.661	2.662	0.022	n/a	14.5	15.4	0.6
Rochester I	2.643	2.634	2.613	0.006	n/a	14.1	13.2	0.6
Rochester II	2.612	2.598	2.584	0.010	n/a	13.4	15.1	1.2
Mankato	2.577	2.501	2.529	0.054	n/a	11.3	12.6	0.9
Willmar I	2.606	2.608	2.599	0.005	n/a	n/a	15.5	--
Willmar II	2.595	2.609	2.559	0.026	n/a	16.3	13.9	1.7

obtained from behind the paver was compacted according to mix design specifications and the bulk specific gravity of the compacted samples was determined.

Table 4.1 is a summary of the aggregate bulk specific gravity used in the VMA calculation, as well as the VMA results from Mn/DOT and the University of Minnesota. (Contractor data for VMA were not available.) All VMA values are post-construction. A standard deviation was included as part of the table to indicate the difference between material and mix parameters.

VMA following construction was determined by averaging the resulting VMA for all the compacted samples. Table 4.2 summarizes both the average mix design and average post-construction VMA values for each project, as well as the difference between the two.

In general, the greater the standard deviation for the combined aggregate bulk specific gravity, the greater the standard deviation of the three VMA values. The table represents only differences between MnDOT and University of Minnesota data because contractor VMA data were not available.

Table 4.2 Difference in VMA

Group	Project	Mix Design VMA (%)	Post-Construction VMA (%)	Difference in VMA (%)
Large	Rochester I	15.8	13.2	- 2.6
	Willmar II	16.2	13.9	- 2.3
	Mankato	14.5	12.6	- 1.9
Moderate	Detroit Lakes II	14.7	13.6	- 1.1
	Rochester II	16.1	15.1	- 1.0
	Metro	16.1	15.4	- 0.7
	Bemidji I	14.7	14.1	- 0.6
	Willmar I	16.1	15.5	- 0.6
Small	Bemidji II	15.0	15.0	0
	Detroit Lakes I	14.2	14.4	+ 0.2

Table 4.2 shows that the only two projects where the VMA did not decrease were Bemidji II and Detroit Lakes I. There were three projects that had a slight decrease in VMA of approximately one-half percent: Bemidji I, Metro, and Willmar I. There were two projects with a decrease in VMA of approximately one percent: Detroit Lakes II and Rochester II. The other three projects had a VMA decrease of two percent or more: Rochester I (- 2.6%), Mankato (- 1.9%), and Willmar II (- 2.3%). The projects were then grouped into three categories:

1. Large decrease: VMA decrease of approximately 2% or more.
2. Moderate decrease: VMA decrease of approximately 0.5 - 1.0%
3. Small decrease: Increase or no change in VMA

Table 4.3 contains a summary of Superpave aggregate test results for the stockpile aggregates. The Asphalt Institute refers to these tests as "Consensus Properties" in its Superpave mix design manual [22]. Chapter 2 contains descriptions of these tests.

Stockpile samples were tested at the University of Minnesota and summarized in a weighted average according to the percentages in the mix. Results are summarized in Appendix B.

Table 4.3 Superpave Aggregate Test Results.

Group	Project	Change in VMA	Superpave Aggregate Test Results (weighted average of all stockpiles)			
			CAA	Flat/ Elongated	FAA	Sand Equivalent
Large	Rochester I*	- 2.6	82/75	15	40.8	81
	Willmar II	- 2.3	99/96	12	40.0	84
	Mankato	- 1.9	99/90	10	40.6	59
Moderate	Detroit Lakes II	- 1.1	66/54	8	39.3	63
	Rochester II*	- 1.0	96/89	13	41.0	84
	Metro*	- 0.7	91/83	12	41.6	83
	Bemidji I	- 0.6	97/88	17	41.2	76
	Willmar I	- 0.6	90/83	10	39.0	64
Small	Bemidji II	0	63/47	13	40.5	75
	Detroit Lakes I	+ 0.2	n/a	n/a	n/a	n/a

* Mix contained RAP which was not included in Superpave aggregate calculations.

Large Decrease in VMA

Rochester I, Willmar II, and Mankato all had a decrease in VMA of approximately two percent or more. In order to determine what caused the drop in VMA, four of the five potential causes were explored:

1. Generation of fines
2. Change in aggregate shape
3. Mix production temperatures
4. Storage and hauling times

The use of baghouse fines was excluded because all projects used baghouse fines collection systems and all fines were fed back into the mix. Since some projects exhibited a decrease in VMA and some did not, it was assumed that there were other factors contributing to the decrease in VMA. The contribution of baghouse fines to the variability of VMA between mix design and post-construction was assumed to be similar for all projects.

Generation of Fines

In order to determine if fines were generated during construction, the aggregate gradations specified in mix design were compared to the aggregate gradation following construction. For all three projects, there did not appear to be any significant increase in fines from mix design to post-construction. In fact, two of the projects showed that the post-construction gradation was coarser than the mix design gradation. As a result, a significant change in total surface area is not expected. The three gradation comparisons are shown in Figure 4.1, Figure 4.2, and Figure 4.3. The difference in surface area between mix design and following construction for all three projects is summarized in Table 4.4

Since the material passing the 75-µm sieve is collected in a pan, there is no direct measurement of particle diameter. Without particle diameter, the contribution of this material to the surface area of the aggregate cannot be accurately estimated and was not included in the surface area analysis. Therefore, the percentage of the aggregate gradation that this material makes up is included in Table 4.4. It can be assumed that the larger this percentage is, the greater the actual surface area. Therefore, in the case of Rochester I, since both gradations resulted in 5 percent minus 75-µm material, the difference in surface area recorded at the bottom of the table is

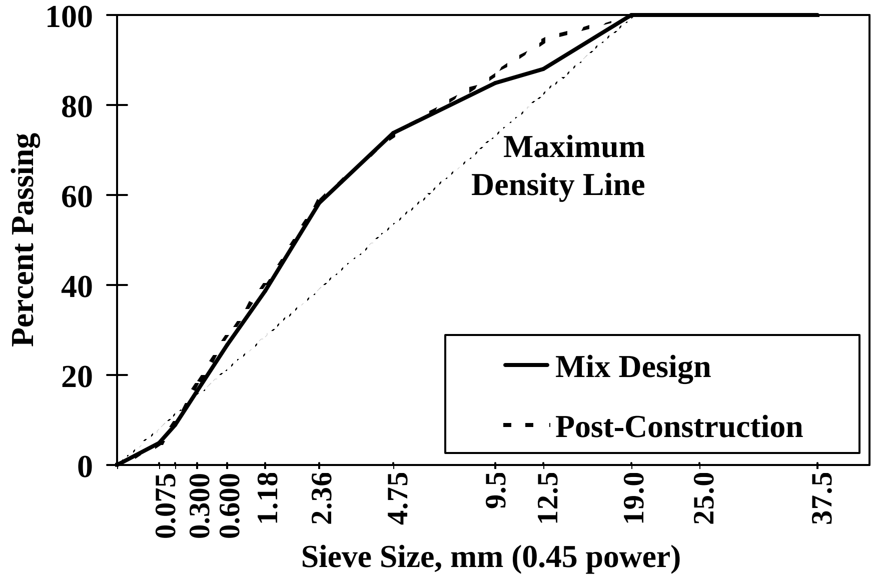

Figure 4.1 Gradation Comparison for Rochester I

appropriate. However, in the cases of both Willmar II and Mankato, the percentage of minus 75-μm material determined for the mix design gradation is greater than that determined for the post-construction gradation. In those two cases, it would be appropriate to assume that if this material was incorporated into the surface area estimation, the difference in the Willmar II gradations would be larger, whereas the difference in the Mankato gradations would be smaller.

Since there was no notable change in the fineness of the aggregate gradations or a noticeably

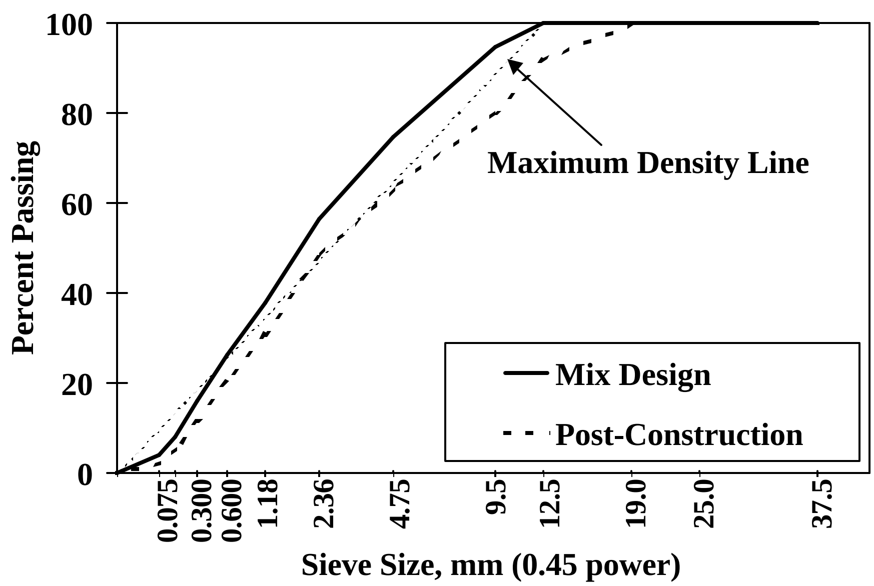

Figure 4.2 Gradation Comparison for Willmar II

large increase in surface area, it can be deduced that the generation of fines did not contribute significantly to the VMA decrease these projects experienced.

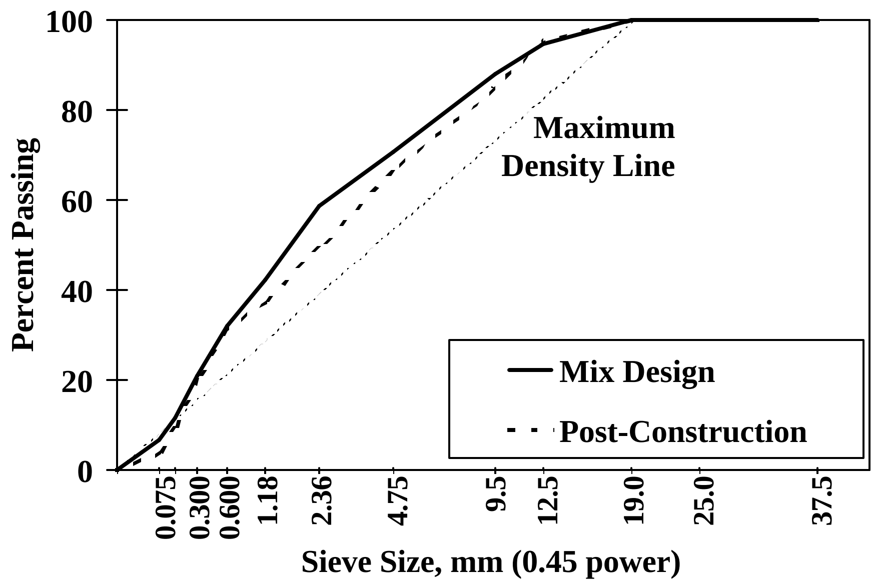

Figure 4.3 Gradation Comparison for Mankato

Change in Aggregate Shape

A brief visual inspection of the mixture aggregate following the ignition test indicated that for all three projects, there was not a significant change in aggregate shape compared to the virgin aggregate. Also, since the aggregate gradation was not finer after construction, it can be

Table 4.4 Summary of surface area estimations

Project	**Rochester I**	**Willmar II**	**Mankato**
Mix Design Surface Area (m^2/kg)	7.5	7.9	8.7
- 75-µm Material Not Included (% of total aggregate)	5	4	7
Post-Construction Surface Area (m^2/kg)	8.3	6.5	9.3
- 75-µm Material Not Included (% of total aggregate)	5	2.3	4
Surface Area Difference (m^2/kg)	+ 0.8	- 1.4	+ 0.6

assumed that the aggregate degradation was negligible, and not a significant contributor to the decrease in VMA. However, in order to accurately prove this assumption, a quantitative measurement of aggregate shape needs to be determined for both the virgin aggregates and the post-construction mix aggregate. The Fine Aggregate Angularity (FAA) test is a candidate for this type of comparison. Results of FAA testing on the original stockpile aggregates are summarized in Table 4.3. Samples of post-construction aggregate were not available.

Mix Production Temperatures / Storage and Hauling Times

The plant mix production temperatures for the three projects were 160, 154, and 149 °C (320, 310, and 300 °F) for Mankato, Willmar II, and Rochester I, respectively. These three production temperatures were the highest of the ten projects. The high production temperatures have the potential to increase the asphalt absorption due to the decrease in viscosity of the asphalt binder. Table 4.5 summarizes the asphalt binder absorption for mix design and post-construction for all ten projects. The absorption values were calculated using the procedure outlined in Chapter 3. Plant production temperatures, hauling distances and storage times are also included. It should be noted that high plant temperatures are generally correlated with long haul distances and curing times.

Table 4.5 indicates that three of the four greatest increases in asphalt absorption between mix design and post-construction are Rochester I, Willmar II, and Mankato, the three projects with the greatest decrease in VMA. From the data, it can be deduced that the increase in asphalt binder absorption could be attributed to three major causes:

1. High plant mix production temperatures
2. Haul distance or storage time
3. Aggregate mineralogy

High plant mix production temperatures

All three projects with greater decreases in VMA had plant production temperatures in excess of 150 °C (300 °F). Plant production temperatures become a factor when they exceed the mixing temperatures stated for a specific grade of asphalt binder. Both Rochester I and Willmar II used a 120/150 penetration asphalt binder, whereas the Mankato project used an 85/100 penetration asphalt binder.

Table 4.5 Asphalt binder Absorption

Project	VMA Difference	Mix Design Asphalt Absorption, P_b (%)	Post-Construction Asphalt Absorption, P_b (%)	P_b Difference	Plant Temperature (°C)	Average Distance to Paving Site (km)	Average Cure Time (Hours)
Rochester I	- 2.6	0.37	1.35	0.98	149	5	0.5
Willmar II	- 2.3	1.00	2.11	1.11	154	61	1.25
Mankato	- 1.9	0.80	2.45	1.65	160	16	0.75
Detroit Lakes II	- 1.1	1.12	1.29	0.17	n/a	n/a	n/a
Rochester II	- 1.0	0.69	1.51	0.82	143	24	1.0
Metro	- 0.7	0.13	1.50	1.37	138	8	0.5
Bemidji I	- 0.6	1.36	1.84	0.48	148	8	1.0
Willmar I	- 0.6	1.10	1.27	0.17	127	3	0.25
Bemidji II	0	1.29	1.08	- 0.21	141	11	0.5
Detroit Lakes I	0.2	1.62	1.65	0.03	n/a	n/a	n/a

Typical mixing temperatures according to Mn/DOT asphalt binder data for most 120/150 penetration asphalt binders used in Minnesota range from 135 to 146 °C (275 to 295 °F); mixing temperatures for 85/100 penetration asphalt binders range from 146 to 152 °C (295 to 305 °F) [25]. Therefore, all three projects had mixing temperatures that were greater than the recommended temperature range. Figure 4.4 illustrates how high temperatures can affect the viscosity of the asphalt binder. High mixing temperatures can decrease asphalt binder viscosity, lead to increased asphalt absorption. If the mix is compacted to the same air void content, this will result in a lower VMA.

Long Hauling Distances and/or Storage Times

From Table 4.5 it appears that a long hauling distance may have played a role in the VMA decrease of the Willmar II project. The average distance from the production plant to the paving site was 38 miles. This amounted to an average asphalt mix storage time of about 1.25 hours.

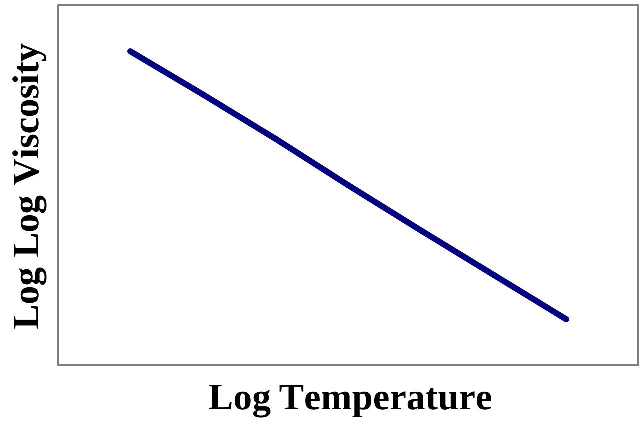

Figure 4.4 Relationship Between Asphalt Binder Viscosity and Temperature

The longer HMA mixes are stored at elevated temperatures, the greater the opportunity for asphalt absorption.

The remaining two projects, Rochester II and Mankato, did not have long hauling distances or storage times. Therefore, the majority of the absorption that occurred in those two projects could be assumed to be from the high plant mixing temperatures.

Aggregate Mineralogy

It is important to note that all three of these projects had limestone aggregate. Roberts, et al [2] stated that asphalt binder normally bonds well with carbonate aggregates, when compared to siliceous aggregates such as gravel. Porous limestone may absorb more asphalt binder.

Moderate Decrease in VMA

Five projects had a moderate decrease in VMA of 1.1 to 0.6 percent. The causes of VMA collapse for these projects are more difficult to define as there does not tend to be a single factor contributing to the decrease. There are several factors involved with varying effects on VMA. In order to determine which are playing roles in the decrease, the four potential causes of VMA drop were reviewed.

Generation of Fines

Of the five projects making up this group, only Detroit Lakes II showed a significant difference in gradation from mix design to post-construction (see Figure 4.5). Bemidji I, Metro, Rochester

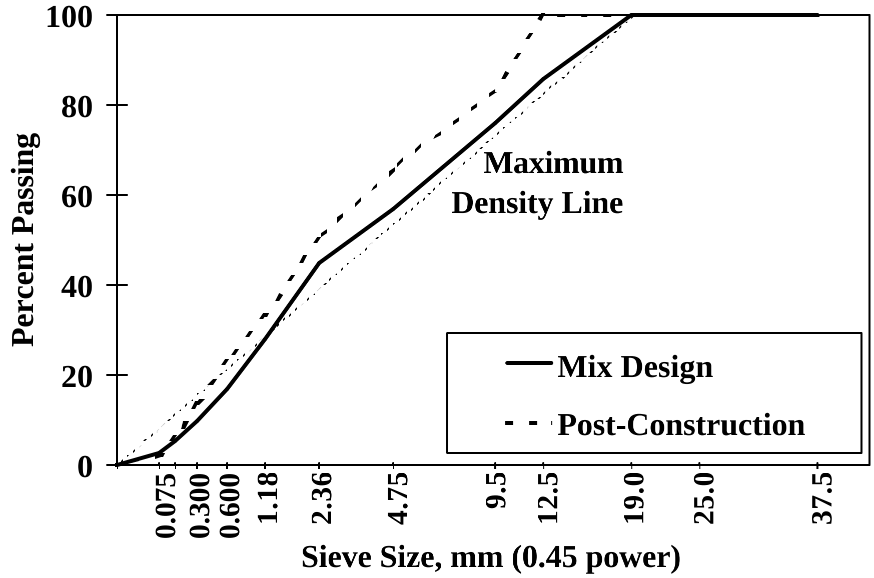

Figure 4.5 Gradation Comparison for Detroit Lakes II

II, and Willmar I, experienced little or no difference in gradation before and after construction. Figure 4.6 illustrates the Rochester II gradation, which is representative of the gradation differences in this group. The pre- and post-construction gradations are very similar for Rochester II.

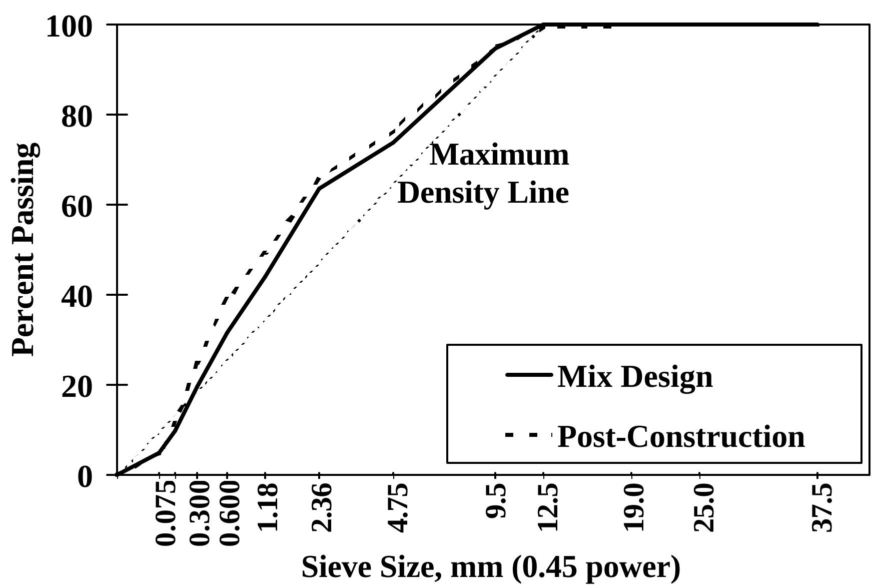

Figure 4.6 Gradation Comparison for Rochester II

Change in Aggregate Shape

By visual inspection, there was not an obvious change in aggregate particle shape indicating little aggregate degradation. In order to accurately determine if any degradation had occurred, a comparison of FAA results for the aggregate blend sampled prior to construction and the aggregate remaining after the ignition oven test should be made.

Mix Production Temperatures / Storage and Hauling Times

Of the five projects experiencing a moderate decrease in VMA, aggregate degradation resulting in a finer gradation can be assumed to be the main factor only for Detroit Lakes II. The other projects do not appear to have a large enough difference in aggregate gradation to explain the decrease in VMA it experienced. Therefore, other factors must have contributed to the decrease.

As shown in Table 4.5, the Metro project had a moderate plant mix temperature of 138 °C (280 °F). The average distance from the plant to the paving site was five miles. Both the temperature and hauling distance are in the medium range when compared to the other nine projects. However, the difference in asphalt absorption between mix design and following construction was the second highest (1.37 percent), behind the Mankato project. Therefore, the decrease in VMA can be assumed to have resulted primarily from the increase in asphalt absorption. But, since the temperature and hauling distance alone do not seem to adequately account for such a large difference in asphalt absorption, the aggregate mineralogy was examined.

Three of the remaining four projects, Detroit Lakes II, Bemidji I, and Willmar I, did not experience a large difference in asphalt absorption. The fourth project, Rochester II, experienced nearly a one percent asphalt absorption (0.82 percent). The plant mix temperature of 143 °C (290 °F) is moderately high and the hauling distance is above average when compared to the other nine projects. Therefore, the increase in asphalt absorption could be accounted for by the mix temperature and hauling distance.

Aggregate Mineralogy

The five projects exhibiting a moderate decrease in VMA all contained aggregate that was made up of both limestone and granite, except for the Rochester II project, which consisted entirely of limestone. As stated previously, aggregate composed of limestone has the tendency to absorb more asphalt binder due to its porosity. Thus, the increase in asphalt absorption for the Rochester II project can be attributed to the aggregate mineralogy, combined with a slightly high plant mix temperature and hauling distance.

The Metro project can be only partially explained. The increase in asphalt absorption cannot be entirely attributed to either the plant mix temperature or hauling time. Neither parameter was high compared to the other projects. The asphalt absorption can be entirely attributed to the aggregate mineralogy, as the project contained approximately an equal amount of granite and limestone. The increase in asphalt absorption between mix design and construction can be caused by the combined effect of the three parameters produced a cumulative increase in asphalt absorption and a subsequent decrease in VMA.

Low Decrease in VMA

The two projects that experienced little or no drop in VMA were Detroit Lakes I and Bemidji II. By looking at the same four causes of VMA collapse, it is possible to explain why these two projects did not experience a decrease in VMA.

Generation of Fines

It has been hypothesized that aggregate degradation and the resulting change in aggregate gradation can contribute to a drop in VMA for a particular project. Therefore, it can be assumed that hard aggregates that do not break down have the potential to maintain the mix design gradation throughout production.

Detroit Lakes I and Bemidji II exhibited a negligible drop in VMA between mix design and production. Figure 4.7 and Figure 4.8 show little change in aggregate gradation between mix design and following construction.

For Detroit Lakes I, the mix design and post-construction gradations are similar for both the larger and smaller sieve sizes, with some deviation in the middle. For Bemidji II, the two gradations actually cross, with the post-construction gradation finer than the mix design gradation over the larger sieve sizes and coarser over the smaller sieve sizes. The difference between mix design and post-construction gradations for the two projects is minimal.

The Bemidji II project, with only a 0.2 m^2/kg difference between mix design and production, had the smallest change in surface area of the ten projects. It appears that the minimal change in gradation and surface area helped minimize the VMA decrease.

Change in Aggregate Shape

Through visual inspection there was no noticeable degradation of the aggregate particles after production. The aggregate remaining following the ignition oven test was nearly identical with the virgin aggregate for each project. It was impossible to detect any change in aggregate shape. This statement is supported by the similar gradations between mix design and post-construction. However, as stated previously, this can be more reliably supported by comparing the results of the virgin aggregate and the mix aggregate using a quantitative result from the FAA Test.

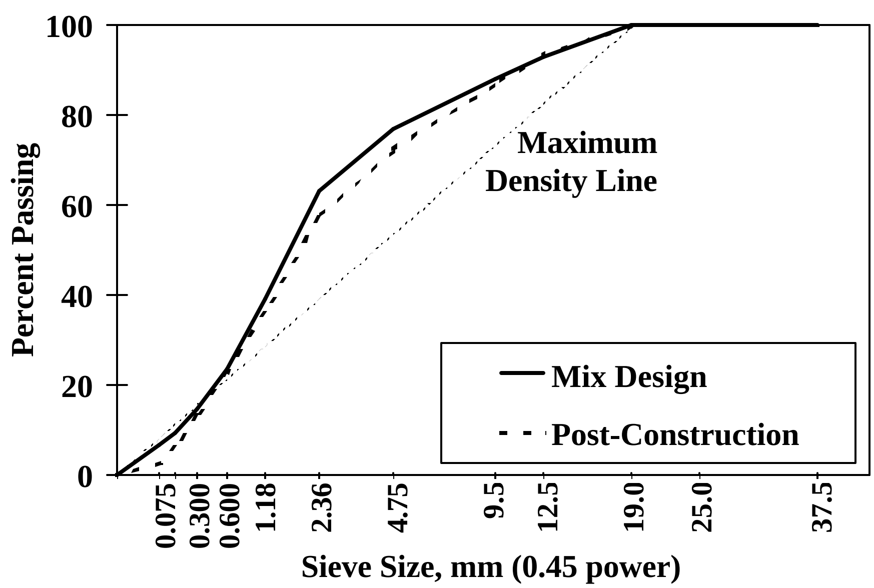

Figure 4.7 Gradation Comparison for Detroit Lakes I

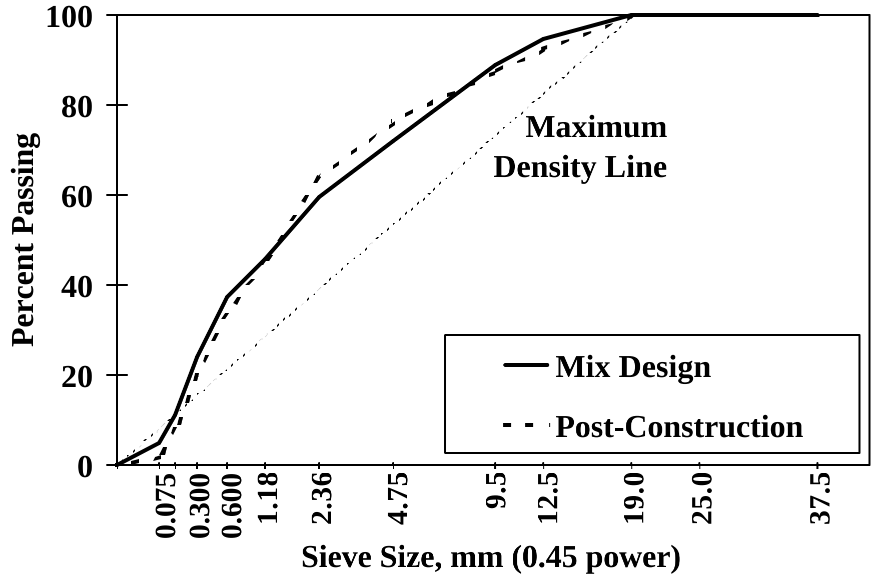

Figure 4.8 Gradation Comparison for Bemidji II

Mix Production Temperatures / Storage and Hauling Times

The Detroit Lakes I project did not provide any data regarding mix production temperatures or hauling times. However, from Table 4.5, the Bemidji II project had a moderate plant mix temperature of 141 °C (285 °F) and a relatively short hauling distance of approximately seven miles. According to Table 4.5, the production asphalt absorption was 0.2 percent less than the mix design absorption. In other words, there was no sign of additional absorption during construction, which explains why this project did not experience a VMA collapse.

Both mix design and post-construction asphalt absorption were determined for Detroit Lakes I. Table 4.5 indicates there was an increase in asphalt absorption of 0.03 percent between mix design and post-construction. Again, it can be assumed that the minimal additional asphalt absorption contributed to the low VMA difference.

Aggregate Mineralogy

The Detroit Lakes I project aggregate was never submitted to the University for analysis. Consequently, there was no record of the aggregate mineralogy for that project, so it cannot be considered.

The Bemidji II project aggregate was composed mainly of limestone which tends to have a high absorption. Table 4.5 shows that the asphalt absorption for both mix design and post-construction was over one percent. However, due to the moderate plant mix temperature and short hauling distance, there was little additional absorption by the limestone aggregate during production, and consequently, minimal change in VMA.

Asphalt Film Thickness Variability

Because of the importance of asphalt film thickness to HMA durability, the asphalt film thickness was estimated for both mix design and following construction. The film thickness estimates, along with the corresponding VMA change, are summarized in Table 4.6. It is important to note that the material passing the 75-μm sieve was not included in the surface area computation, and thus, the average asphalt film thickness calculation. Since this material has the potential to increase the surface area by a fair amount, the estimate of average film thickness for each project contained in Table 4.6 can be assumed to be greater than what was calculated.

Table 4.6 indicates that nine of the ten projects experienced a loss in average film thickness. The only project that did not was the Bemidji II project, which also did not experience a drop in

Table 4.6 Change in Asphalt Film Thickness

Project	VMA Difference (%)	Estimated Average Film Thickness (μm)		
		Mix Design	Post-Construction	Difference
Rochester I	-2.6	7.1	6.2	- 0.9
Willmar II	- 2.3	7.5	6.3	- 1.2
Mankato	-1.9	5.6	3.8	- 1.8
Detroit Lakes II	-1.1	9.3	6.2	- 3.1
Rochester II	-1.0	6.2	4.4	- 1.8
Metro	-0.7	7.4	6.0	- 1.4
Bemidji I	-0.6	6.7	4.5	- 2.2
Willmar I	-0.6	9.4	7.1	- 2.3
Bemidji II	0	4.8	5.1	+ 0.3
Detroit Lakes I	0.2	7.8	6.7	- 1.1

VMA. The other project that did not experience a drop in VMA, Detroit Lakes I, had the smallest drop in asphalt film thickness when compared to the other nine projects.

Asphalt film thickness is directly related to aggregate gradation. In general, a fine-graded mix has a greater total surface area and therefore a lower film thickness than a coarse-graded mix. Figure 4.5 shows the change in aggregate gradation for Detroit Lakes II which experienced the greatest change in asphalt film thickness. Figure 4.8 illustrates the gradations for Bemidji II, the only project without a loss in film thickness. It did not experience any significant change in aggregate gradation between mix design and construction.

Aggregate Gradation Distances

As mentioned in Chapter 3, aggregate gradations can be evaluated using the 0.45 power gradation chart. Both the mix design and post-construction aggregate gradations were compared to the maximum density line for each project. This was done by determining the sum of the differences in percent passing between the maximum density line and the gradations over a standard set of sieves. These results, along with VMA and asphalt film thickness comparisons, are listed in Table 4.7.

Table 4.7 Aggregate Gradation Differences from the Maximum Density Line

Project	VMA Difference (%)	Sum of Distances from Maximum Density Line			Change in Asphalt Film Thickness (μm)
		Mix Design	Post-Construction	Absolute Difference	
Rochester I	- 2.6	59	74	15	- 0.9
Willmar II	- 2.3	20	46	26	- 1.2
Mankato	- 1.9	38	24	14	- 1.8
Detroit Lakes II	- 1.1	17	48	31	- 3.1
Rochester II	- 1.0	29	42	13	- 1.8
Metro	- 0.7	51	63	12	- 1.4
Bemidji I	- 0.6	61	81	20	- 2.2
Willmar I	- 0.6	19	30	11	- 2.3
Bemidji II	0	72	76	4	+ 0.3
Detroit Lakes I	+ 0.2	56	50	6	- 1.1

Eight of the ten projects had post-construction aggregate gradations that deviated farther from the maximum density line than the mix design aggregate gradation. In all but one of these cases, the deviation was caused by the aggregate gradation becoming finer. It was suggested in Chapter 2 that deviating from the maximum density line in either the coarse or fine direction tends to cause an increase in VMA. However, it is also stated that the generation of fines increases surface area and tends to decrease VMA for the same level of compaction. The results of this project show that deviating from the maximum density line in the fine direction did not necessarily result in a mix with a higher VMA

The two projects that did not have a drop in VMA, Bemidji II and Detroit Lakes I, had very little change when comparing the mix design and post-construction gradations to the maximum density line (distances of 4 and 6, respectively). These were the smallest changes of the ten projects. These minimal changes also resulted in the smallest decrease in asphalt film thickness from mix design to post-construction.

The project experiencing the greatest change in gradation from mix design to construction, Detroit Lakes II, also had the greatest decrease in asphalt film thickness. Figure 4.5 indicates that the cause can be attributed to a much finer post-construction aggregate gradation.

Moisture Sensitivity

Each of the ten projects were subjected to a Lottman test as detailed in Chapter 2. Table 4.8 shows that the majority of the projects had a tensile strength ratio (TSR) greater than 70 percent. The two projects that had lower values were the Metro project with a borderline TSR of 68.2 percent and Willmar I, with a TSR of 59.5 percent. The Willmar I mix would not meet current design specifications.

The average dry tensile strengths for these two projects were also relatively low. The two Bemidji projects exhibited the lowest dry tensile strengths, considerably lower than the other six projects tested. These two projects were also the only two 31- type mixes, which are mixes used on low volume roadways. All other projects were either designed for medium or high volume or contained RAP material. Eliminating these two projects and Metro and Willmar I, the two projects with the lowest TSR, are also the two projects with the lowest average dry tensile strength.

Table 4.8 Lottman Test Results

Project	Average Air Void Content (%)	Average Dry Tensile Strength (MPa)	Average Wet Tensile Strength (MPa)	Tensile Strength Ratio (%)
Bemidji I	7.0	0.717	0.552	77.2
Bemidji II	6.5	0.738	0.593	79.6
Detroit Lakes I	6.6	1.90	1.45	75.2
Detroit Lakes II	N/A	N/A	N/A	N/A
Metro	6.7	1.40	0.951	68.2
Rochester I	5.1	1.96	1.54	78.5
Rochester II	8.1	1.97	1.56	79.4
Mankato	8.2	2.15	1.80	81.8
Willmar I	8.0	1.52	0.903	59.5
Willmar II	N/A	N/A	N/A	N/A

The Willmar I project, with the lowest TSR, also had one of the greatest decreases in asphalt film thickness. Detroit Lakes II, which had over a 3-μm decrease in asphalt film thickness, had an insufficient supply of behind-the-paver material to do moisture sensitivity testing. The other project with a large decrease in asphalt film thickness, Bemidji I, had a TSR of 77 percent. This project had one of the highest overall film thickness values.

SUMMARY

Of the possible causes of decrease in VMA, none stand out as a significant factor in all of the projects. In the three projects with the largest decrease in VMA (Rochester I, Willmar II, and Mankato), increased asphalt absorption due to a combination of high plant temperatures, long haul times, and porous aggregates appears to be the main cause. The relationship between plant temperature and change in VMA is shown in Figure 4.9, and the relationship between cure time and change in VMA is shown in Figure 4.10.

Of the five projects with a moderate decrease in VMA, Rochester II and Bemidji I had moderately high plant temperatures and cure times of about 1 hour. Detroit Lakes II had a significant change in gradation (see Figure 4.5) which indicates a generation of fines. Detroit

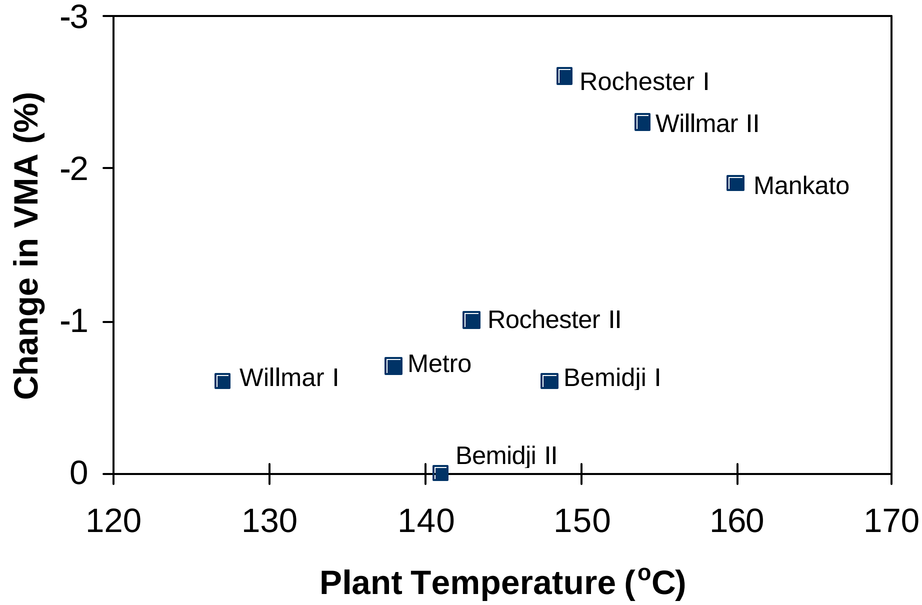

Figure 4.9 Relationship Between Plant Temperature and Change in VMA

Lakes II, Bemidji I, and Willmar I all had an estimated drop in film thickness in excess of 2 µm, while the decrease for Rochester II was nearly 2 µm (see Table 4.6).

The two projects with no VMA decrease (Bemidji II and Detroit Lakes I) had little or no decrease in film thickness, and very little change in gradation.

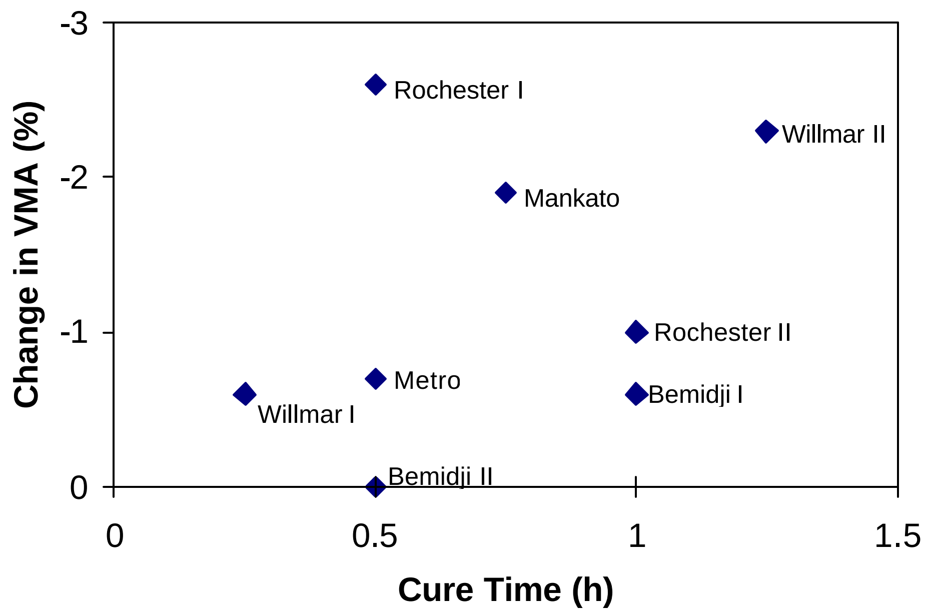

Figure 4.10 Relationship Between Cure Time and Change in VMA

CHAPTER 5
PRECISION OF THE LOTTMAN TEST

BACKGROUND

Test specimens of hot mix asphalt were proportioned and mixed in the Mn/DOT, Asphalt Engineering Laboratory according to Standard Test Method for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus (ASTM D 1559). Six specimens were prepared per participating laboratory. Three were tested dry and three were tested after partial saturation and moisture conditioning. Specimens typically were 102 mm in diameter and 64 mm high.

The samples were randomly divided and shipped to several Minnesota laboratories. The laboratories were requested to compact the specimens in accordance with ASTM D 1559. Lottman tests on the compacted specimens were preformed in accordance to ASTM D 4867. The specimens were compacted to 7 ± 1 percent air voids. The Lottman test procedure was performed within 24 hours after compaction. The theoretical maximum specific gravity was determined in accordance to ASTM D 2041. The specimen height was determined in accordance to ASTM D 3549. The bulk specific gravity was determined in accordance to ASTM D 2726. Calculations for the air voids content were performed in accordance to ASTM D 3203. The specimens were sorted into two subsets so that the average air voids of two subsets are approximately equal. A subset to be tested dry was stored while the other subset underwent moisture conditioning.

The subset to be moisture conditioned was partially saturated to between 55 and 80 percent with distilled water at room temperature using a vacuum chamber. The partial saturation was performed using a partial vacuum of approximately 508 mm Hg for approximately five minutes.

The partially saturated specimens were conditioned by soaking them in distilled water at 60 ± 1.0 °C (140 ± 2.0 °F) for 24 hours. The swell of the partially saturated specimens was determined by dividing the change in specimen volume by the original specimen volume.

The temperature of the moisture-conditioned and dry subsets were adjusted by soaking the specimens in a water bath at 25 ± 1.0 °C (77 ± 2.0 °F). The specimens were placed in the loading frame and the loading strips were placed so that they were parallel and were centered on the vertical diametral plane. A diametral load rate of 50 mm/min was applied until maximum load was reached. Loading continued until the specimen fractured. The specimen was broken open to observe any moisture damage. The dry and wet tensile strength and TSR were calculated.

The results were sent to the Pavement Materials Laboratory at the University of Minnesota for analysis. The calculations and associated values presented in this report are based solely on the data received from these laboratories.

CALCULATIONS

Tensile Strength Ratio

The tensile strength and TSR were calculated using Equations 2.7 and 1.4, respectively.

The data were analyzed statistically to determine if there was a significant difference in TSR values within the 14 laboratories and between laboratories. The variability was determined by first calculating the average value and standard deviation for each of the mixes tested by the individual laboratories. The cell (laboratory) averages were then averaged to give a grand average of all TSR values. The variation of individual laboratories from the overall average was then determined.

Between laboratory consistency is defined using the h-value for each lab defined in Equation 4.10. The h-value is compared to the t-statistic to establish if the laboratory results are in the same population defined by the overall average and standard deviation.

The within laboratory variation is defined using the k-value statistic defined by Equation 4.11. The k-value for each laboratory is used to determine if the laboratory is indicating a material in the same population defined by the total testing program.

Cell Average

The cell average is the average of all the data obtained for an individual laboratory.

$$\bar{x} = \frac{\sum_{1}^{n} x}{n} \qquad 4.1$$

Where:

$\bar{x}$ = Cell average

x = Individual test results in one cell

n = Number of test results reported per laboratory

Cell Standard Deviation

The cell standard deviation is a measure of the within-laboratory variability of each individual laboratory.

$$s = \sqrt{\frac{\sum_{1}^{n} \left(x - \bar{x}\right)^2}{n - 1}} \qquad 4.2$$

Where:

s = Cell standard deviation

Average of the Cell Averages

The Average of the Cell Averages is the mean value for the all of the samples produced.

$$\dot{x} = \sum_{1}^{p} \frac{\bar{x}}{p} \qquad 4.3$$

Where:

$\dot{x}$ = Average of the cell averages for one material

p = Number of laboratories participating in the study

Cell Deviation

The Cell Deviation is the deviation a particular laboratory average is from the overall average.

$$d = \bar{x} - \dot{x} \qquad 4.4$$

Where:

d = Cell deviation

Standard Deviation of the Cell Averages

The Standard Deviation of the cell averages is a measure of the variability of the cell averages.

$$s_{\overline{x}} = \sqrt{\sum_{1}^{p} \frac{d^2}{(p-1)}} \qquad 4.5$$

Where:

$s_{\overline{x}}$ = Standard deviation of the cell averages

Repeatability Standard Deviation

The repeatability standard deviation is a measure of inner-laboratory variability.

$$s_r = \sqrt{\sum_{1}^{p} \frac{s^2}{p}} \qquad 4.6$$

Where:

s_r = Repeatability standard deviation

Reproducibility Standard Deviation

The reproducibility standard deviation is a measure of inter-laboratory variability.

$$s_R^* = \sqrt{s_{\overline{x}}^2 + \frac{s_r^2 (n-1)}{n}} \qquad 4.7$$

Where:

s_R^* = Reproducibility standard deviation

n = Number of test results reported per laboratory

* indicates a provisional value

Repeatability and Reproducibility Limits

The repeatability and reproducibility limits are proportional the repeatability standard deviation and reproducibility standard deviation, respectively.

$$\text{Repeatability limit} = 2.8 \times s_r \qquad 4.8$$

$$\text{Reproducibility limit} = 2.8 \times s_R^* \qquad 4.9$$

Consistency Statistics

The h-value is defined as the between laboratory consistency statistic. The h-value is an indicator of how one laboratory cell average compares with the other laboratory cell averages. Critical h-values are calculated using a relationship with the t-value. The k-value is the within laboratory consistency statistic. The k-value is an indicator of how one laboratory's with-in lab variability compares with the combined variability of all the participating laboratories. Critical k-values are calculated using a relationship with the F-value.

$$h = \frac{d}{s_{\bar{x}}} \tag{4.10}$$

$$k = \frac{s}{s_r} \tag{4.11}$$

Where:

h = Between laboratory consistency statistic

k = Within laboratory consistency statistic

F-value

The F-value is a ratio of variabilities, it indicates whether the data from the different laboratories are from the same population.

$$\text{F-value} = n \times \frac{s_{\bar{x}}^2}{s_r^2} \tag{4.12}$$

RESULTS

ASTM D 4867 [7] states that the repeatability limit of the tensile strength should be 159 kPa (23 psi). This index is also known as the 95 percent limit on the difference between two test results, meaning that similar data between specimens can be expected to differ in absolute value by less than 2.8 times the repeatability standard deviation. The repeatability standard deviation is a measure of the inner-laboratory variability. The wet and dry tensile strength had values for the repeatability limit of 149 and 147 kPa (22 and 21 psi), respectively. The repeatability limits for tensile strength in this study fall within the limit to be expected as reported by ASTM D 4867.

ASTM D 4867 reports that the reproducibility limit for the tensile strength ratio should be 23 percent. This index is also known as the 95 percent limit on the difference between two test results, meaning that similar data between laboratories can be expected to differ in absolute value by less than 2.8 times the reproducibility standard deviation. The reproducibility standard deviation is a measure of the between-laboratory variability. The tensile strength ratio had a value for the reproducibility limit of 138.1 percent. This value exceeds the expected value reported by ASTM D 4867.

The consistency statistics are a measure of within- and between-laboratory variability. The h-value is the between laboratory consistency statistic. It is an indicator of how one laboratory cell average compares with another laboratory cell average. Critical h-values are calculated using a relationship with the t-value. The k-value is the within laboratory consistency statistic. It is an indicator of how one laboratory's inner-laboratory variability compares with the combined variability of all the participating laboratories. Critical k-values are calculated using a relationship with the F-value. A table of critical h- and k-values can be found in ASTM E 691 [26].

The critical h- and k-values are 2.64 and 2.24, respectively. The laboratory 14 consistency statistics exceed the critical values in the wet and dry tensile strengths and in the tensile strength ratio. Laboratory 14 data were excluded from all of the calculations because of the large variability. Laboratory 56 also exceeded the critical k-value for the dry and wet tensile strengths. The reported dry tensile strengths vary from 1160 to 1425 kPa (168 to 207 psi). The reported wet tensile strengths vary from 969 to1156 kPa (141 to 168 psi). Detailed tables of the statistical results are contained in Appendix D.

The F-value is a ratio of between-laboratory variabilities to within-laboratory variabilities. It is an indicator of whether the data reported by all the laboratories are from the same population. Critical F-values are found in most probability and statistic books. The critical F-value for a 95 percent confidence level is 1.67. None of the laboratories met the criteria. The percent air voids was the closest with a F-value of 3.8 followed by tensile strength ratio having a F-value of 4.6. The other material characteristics also resulted in large F-values.

The high variability of the data in this study can be attributed to deviations from ASTM specified test procedure. Three laboratories assumed a specimen thickness of 63.6 mm. The laboratories used these assumed values to calculate the tensile strengths for the specimens. ASTM specifies that a thickness of 63.6 mm should be targeted during compaction and measurement of the exact height should be made. The thickness of each specimen was calculated using the measured volume and diameter. The thickness was then used to calculate the tensile strength. Investigation into the difference in tensile strengths calculated using the assumed and actual values show a measurable difference.

Another deviation from the ASTM procedure is the method used to saturate or moisture-condition the specimens. The time and pressure used to saturate the specimens was highly variable. ASTM D 4867 specifies applying a partial pressure of 508 mm (29 in.) Hg for 5 min. targeting a percent saturation between 55 and 80 percent.

CONCLUSIONS

Deviations from the ASTM test procedure by the participating laboratories resulted in highly varied results. It is recommended that ASTM procedures be followed closely. On-going training of laboratory personnel and regular equipment calibration are recommended to improve on the repeatability of the Lottman test, especially if it is to be used for project mix design control.

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The following conclusions can be made based on the findings of this research program:

1. VMA collapse usually cannot be explained by one cause. In general, VMA collapse is caused by a combination of two elements: (a) generation of fines and (b) asphalt absorption.
2. The generation of fines can be caused by aggregate degradation due to aggregate handling during loading, hauling, mixing, and construction. Aggregate degradation can cause particles to lose their shape or texture. Degradation can cause more fines, significantly changing the gradation.
3. If asphalt absorption is greater in the field than in the design mix and the mix is compacted to the same level of air voids, the field VMA will be lower. Increased asphalt absorption into the aggregate can be caused by high plant mix temperatures, long hauling distances, and aggregate porosity. Plant mix temperatures in excess of 150 °C (300 °F) decrease the viscosity of the asphalt binder, increasing the potential for absorption by the aggregate. Long hauling distances result in longer mix storage time. The longer HMA mixes are maintained at elevated temperatures, the greater the potential for asphalt absorption. Aggregates composed primarily of limestone have a potential for high absorption. This shows the need to evaluate the mix as close to field conditions as possible.
4. Asphalt film thickness can give insight into changes occurring between mix design and production. However, because it is a general estimate, it is better analyzed in conjunction with other parameters, such as VMA or aggregate gradation distances from the maximum density line.
5. In general, if an HMA has about the same asphalt film thickness from mix design to production, there will be little or no change in VMA.
6. HMA mixes experiencing aggregate degradation during production tend to have increased aggregate surface areas and decreased asphalt film thickness.

7. If the difference in the sum of the distances in percent passing between the maximum density line and the two gradations, mix design and post-construction, is minimal, the HMA will experience little or no change in asphalt film thickness. This also increases the potential for the HMA to maintain the mix design level of VMA.
8. In general, aggregate gradations that deviate from the maximum density line in the fine direction did not increase the VMA. Rather, finer aggregate gradations tended to result in a decrease in VMA due to the additional fine material and a lower film thickness.
9. HMA mixes with significant decreases in asphalt film thickness between mix design and post-construction have more potential for low TSR values.
10. Due to the limited amount of moisture sensitivity testing, acceptable correlations between either low asphalt film thickness or low VMA cannot be shown.
11. The repeatability of the Lottman test was not acceptable because standard procedures were not followed; especially height measurements and saturation procedures.

RECOMMENDATIONS

Based on the conclusions above, the following recommendations have been determined:

1. The properties and proportions of the materials in a hot-mix asphalt need to be measured carefully and consistently using standard ASTM or AASHTO procedures so that their effect on performance can be evaluated.
2. The aggregate in an HMA is subject to crushing and degradation when stockpiled, loaded, fed through a hot-mix plant, hauled to the job site, placed through the paver, and compacted by the rollers. There needs to be a test performed during mix design to determine the level of toughness and resistance to abrasion of the project aggregate. If an aggregate is deemed suitable for an HMA mixture, except that it has a tendency to generate some fines during handling, this can be accounted for in the preliminary designs and adjusted so that the volumetric properties in the field are within specification.
3. The absorption capacity of the aggregates must also be accurately measured in the design phase. Absorptive aggregates must be acknowledged and designed for

appropriately by accounting for the additional asphalt binder that will be absorbed and no longer available to coat the aggregate particles.

4. The bulk specific gravity of the aggregate must be accurately determined in the design phase. It is apparent that the lack of precision in determining aggregate bulk specific gravity and absorption will directly cause poor precision in determining VMA. Standard ASTM and/or AASHTO procedures must be followed carefully and consistently throughout MnDOT and by contractors.
5. The tendency for increased absorption due to higher temperatures and/or increased storage time should be determined for specific aggregate sources. This can be accomplished by running bulk specific gravity and maximum theoretical specific gravity tests on mixes using the same aggregate with various temperatures and storage times as a special study on some projects around the state.
6. Plant mix temperatures must stay within the specified range for the asphalt binder grade being used in the HMA. Plant mix temperatures that exceed this create the potential for increased asphalt absorption and lower VMA, as well as possible "drain down."
7. While asphalt film thickness is a very important parameter with regard to pavement durability, it has to be understood that because of variations in aggregate shape and surface characteristics, it is only an estimate and should be viewed with a level of caution. Asphalt film thickness is best analyzed when it can be correlated and compared with other volumetric parameters.
8. Continued research in this area, especially with regard to particle size and shape changes between mix design and post-construction, as well as potential correlations with decreases in VMA or asphalt film thickness and pavement durability. This should include:
   a. Correlation of TSR with performance of the jobs being monitored.
   b. Correlation between VMA and TSR.
   c. Correlation between film thickness and TSR.
   d. Correlation between VMA collapse and durability problems.
9. The performance of these projects should be monitored to determine how well the in-service pavement durability relates to the results of the VMA and moisture

susceptibility tests run on the mixtures used during the design and construction of the projects.

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APPENDIX A

ASPHALT PLANT INFORMATION

VMA
Bituminous Pilot Project
(Plant Information)

Project: District 2 (Bemidji I) 1506-30
Highway: 92
Description: Bituminous Overlay

Plant Information:

Production Rate: 360 - 370 ton/hr (on average)

Capacity: 600 ton/hr

Make of Plant/Model Number: Drum plant / Boeing

Angle of Drum: 3.5 in / 4 ft

Speed of Drum: 7.25 rev. / min. (10 ft. diameter)

Dust System: Baghouse Fines
At What Rate: 100% back into mix

Stockpile Moisture Contents: 4 %

Temperature of Plant Mix During Production: 298 ^{o} F

Temperature of Plant Mix Behind Paver: 275 - 285 ^{o} F

Distance from Plant to Paving Site: 5 miles

Average Curing Time of Mix: 1 hour, depending on paving location within job.

Other Notes:

VMA
Bituminous Pilot Project
(Plant Information)

Project: District 2 (Bemidji II) 6007-10
Highway: 32
Description: Bituminous Overlay

Plant Information:

Production Rate: 340 - 350 ton/hr (on average)

Capacity: 450 ton/hr

Make of Plant/Model Number: Drum plant / Boeing

Angle of Drum: 3.5 in / 4 ft

Speed of Drum: 7.25 rev. / min. (10 ft. diameter)

Dust System: Baghouse Fines
At What Rate: 100% back into mix

Stockpile Moisture Contents: 3.8 %

Temperature of Plant Mix During Production: 285 ^{o} F

Temperature of Plant Mix Behind Paver: 275 ^{o} F

Distance from Plant to Paving Site: 7 miles

Average Curing Time of Mix: 30 - 45 minutes, depending on paving location within job.

Other Notes:

VMA
Bituminous Pilot Project
(Plant Information)

Project: District 4 (Detroit Lakes I) 7505-18
Highway: 59
Description: Regrades (Morris to south county line)

Plant Information:

Production Rate: 360 - 370 ton/hr (on average)

Capacity: 600 ton/hr

Make of Plant/Model Number: Drum plant / Boeing

Angle of Drum: 3.5 in / 4 ft

Speed of Drum: 7.25 rev. / min. (10 ft. diameter)

Dust System: Baghouse Fines
At What Rate: 100% back into mix

Stockpile Moisture Contents: 4 %

Temperature of Plant Mix During Production: 298 ^{o} F

Temperature of Plant Mix Behind Paver: 275 - 285 ^{o} F

Distance from Plant to Paving Site: 5 miles

Average Curing Time of Mix: 1 hour, depending on paving location within job.

Other Notes:

VMA
Bituminous Pilot Project
(Plant Information)

Project: District 4 (Detroit Lakes II) 8408-41
Highway: 75
Description: Bituminous Overlay; Widening

Plant Information:

Production Rate: 350 - 400 ton/hr (on average)

Capacity: 600 ton/hr

Make of Plant/Model Number: Drum plant / DM 70 Barber Green

Angle of Drum: 3.5 in / 4 ft

Speed of Drum: 7.25 rev. / min. (10 ft. diameter)

Dust System: Baghouse Fines
At What Rate: 100% back into mix

Stockpile Moisture Contents: 4 %

Temperature of Plant Mix During Production: 298 ^{o} F

Temperature of Plant Mix Behind Paver: 275 - 285 ^{o} F

Distance from Plant to Paving Site: 22-23 miles

Average Curing Time of Mix: 1 hour, depending on paving location within job.

Other Notes:

VMA
Bituminous Pilot Project
(Plant Information)

Project: District 7 (Mankato) 5209-58
Highway: 169
Description: Bituminous Overlay, St. Peter to LeSeuer

<u>Plant Information</u>:

Production Rate: 300 ton/hr (on average)

Capacity: 400 ton/hr

Make of Plant/Model Number: Drum plant / Gencorp (SP?)

Angle of Drum:

Speed of Drum:

Dust System: Baghouse Fines
At What Rate: 100% back into mix

Stockpile Moisture Contents: 7 %

Temperature of Plant Mix During Production: 300 - 320 ^{o} F

Temperature of Plant Mix Behind Paver: 285 - 295 ^{o} F

Distance from Plant to Paving Site: 10 miles

Average Curing Time of Mix: 40 minutes, depending on paving location within job.

Other Notes:

VMA
Bituminous Pilot Project
(Plant Information)

Project: Metro 1906-40
Highway: 52
Description: Mill and Overlay, south junction of highway 55 to highway 50

Plant Information:

Production Rate: 560 ton/hr (on average)

Capacity: 600 ton/hr

Make of Plant/Model Number: Drum plant / Barber Green

Angle of Drum: 4 in

Speed of Drum: 6 - 7 rev. / min.

Dust System: Baghouse Fines
At What Rate: Assumed a gain of 1% of -200 material
(It actually ran about 0.8%)

Stockpile Moisture Contents: RAP: 3.5 % ;
Cold feed combined virgin aggregate: 2.6%

Temperature of Plant Mix During Production: 275 - 285 ^{o} F

Temperature of Plant Mix Behind Paver: 240 ^{o} F

Distance from Plant to Paving Site: 0.5 miles from end of job; 5 miles to middle of job.

Average Curing Time of Mix: 15 - 20 minutes; 10 minute minimum; 30 - 45 min. maximum, depending on traffic and location within job.

Other Notes:

VMA
Bituminous Pilot Project
(Plant Information)

Project: District 6 (Rochester I)   2514-89
Highway: 61
Description: Grade and Surface, North of Red Wing to Highway 316

Plant Information:

Production Rate: 350 ton/hr (on average)

Capacity: 400 ton/hr

Make of Plant/Model Number: Drum plant / DM 70 Barber Green

Angle of Drum: 1 in / 1 ft

Speed of Drum: 7 rev. / min. (10 ft. diameter)

Dust System: Baghouse Fines
At What Rate: 100% back into mix

Stockpile Moisture Contents: 4 %

Temperature of Plant Mix During Production: 300 ^{o} F

Temperature of Plant Mix Behind Paver: 280 ^{o} F

Distance from Plant to Paving Site: 3 miles

Average Curing Time of Mix: 30 minutes, depending on paving location within job.

Other Notes:

VMA
Bituminous Pilot Project
(Plant Information)

Project: District 6 (Rochester II)　　5503-28
Highway: 14
Description: Mill and Overlay, Chester to St. Charles

Plant Information:

Production Rate: 425 ton/hr　(on average)

Capacity: 450 ton/hr

Make of Plant/Model Number: Drum plant / Magnum Series 500

Angle of Drum: 1 in / 1 ft

Speed of Drum: 7 rev. / min.　(10 ft. diameter)

Dust System: Baghouse Fines
At What Rate: 100% back into mix
0.5 - 1.0 % of total mix

Stockpile Moisture Contents: 4 - 4.5 %

Temperature of Plant Mix During Production: 290 ^{o} F

Temperature of Plant Mix Behind Paver: 256 ^{o} F

Distance from Plant to Paving Site: 15 miles

Average Curing Time of Mix: 15 minutes to 1.5 hours, depending on paving location within job.

Other Notes: Mix Type: 42 Binder mix, 42 Wearing Coarse

VMA
Bituminous Pilot Project
(Plant Information)

Project: District 8 (Willmar I) 4201-36
Highway: 14
Description: Bituminous Overlay, Florence to Balletin

Plant Information:

Production Rate: 330 ton/hr (on average)

Capacity: 250 - 500 ton/hr

Make of Plant/Model Number: Drum plant / DM 70 Barber Green

Angle of Drum: 3.5 ^{o}

Speed of Drum: 6 rpm

Dust System: Baghouse Fines
At What Rate: 100% back into mix

Stockpile Moisture Contents: 5 % Washed Sand was 7%

Temperature of Plant Mix During Production: 250 - 260 ^{o} F

Temperature of Plant Mix Behind Paver: 250 ^{o} F

Distance from Plant to Paving Site: 1.5 miles

Average Curing Time of Mix: 10 - 15 minutes, depending on paving location within job.

Other Notes:

VMA
Bituminous Pilot Project
(Plant Information)

Project: District 8 (Willmar II) 4303-22
Highway: 15
Description: Bituminous Overlay, Winthrop to 212

Plant Information:

Production Rate: 325 ton/hr (on average)

Capacity: 400 ton/hr

Make of Plant/Model Number: Drum plant / Cedar Rapids 10040 HMS PR

Angle of Drum: 5/8" to 12 in

Speed of Drum: 9 rev. / min.

Dust System: Baghouse Fines
At What Rate: All fines reentered into mix

Stockpile Moisture Contents: 2.8 %

Temperature of Plant Mix During Production: 310 ^{o} F

Temperature of Plant Mix Behind Paver: 270 ^{o} F

Distance from Plant to Paving Site: 38 miles

Average Curing Time of Mix: 1 hour and ten minutes

Other Notes:

APPENDIX B

PROJECT INFORMATION

Bemidji I

TH-92

Project 1506-30

AC Content =	6.0	TSR Data:		VMA Data:	
Marshall Density (pcf) =	147.0	U of Mn:	79.6	Design	16.2
Design BSG =	2.367	Contractor:	71.0	U of Mn	15.5
Design VMA =	14.7	Mn/DOT:	72.4	Mn/DOT	14.9

VMA Required: 14.5 with 1.2% allowed for drop
Minimum VMA = 13.3

Aggregate Bulk Specific Gravity		Shevlin Fines +4	Shevlin Fines -4	Oein Fines +4	Oein Fines -4	Oein Coarse	**Blend**
	Percent	11.55	43.45	8.4	31.6	5	**100**
Design	C-mid	2.687	2.639	2.676	2.618	2.757	**2.646**
	Mn/DOT	2.690	2.587	2.671	2.556	2.764	**2.604**
	Univ. of MN	2.612	2.612	2.561	2.561	2.763	**2.598**
Prod - 1	C-mid	2.682	2.642	2.650	2.591	2.772	**2.637**
	Mn/DOT	2.687	2.628	2.650	2.501	2.748	**2.600**
	Maplewood	2.672	2.626	2.647	2.515	2.767	**2.603**
Prod - 2	C-mid	2.691	2.646	2.667	2.615	2.727	**2.647**
	Mn/DOT	2.682	2.596	2.657	2.572	2.772	**2.611**
	AVERAGE	**2.675**	**2.622**	**2.647**	**2.566**	**2.759**	**2.618**
						U of M Avg:	**2.598**
Superpave Tests (U of MN)							
Coarse Agg. Angularity (1 f.f./2 f.f.)						97/88	**97/88**
Flat/Elongated Particles						17	**17**
Fine Agg. Angularity			41.2		41.2		**41.2**
Sand Equivalent			80		70		**76**

Bemidji II

TH-32

Project 6007-10

VMA Required: 14.5 with 1.2% allowed for drop
Minimum VMA = 13.3

AC Content =	6.1	TSR Data:		VMA Data:	
Marshall Density (pcf) =	146.5	U of Mn:	86.0	Design	15.2
Design BSG =	2.352	Contractor:	73.0	U of Mn	15.0
Design VMA =	15.0	Mn/DOT:	78.3	Mn/DOT	15.4

Aggregate Bulk Specific Gravity		Deloron Fines +4	Deloron Fines -4	Deloron Rock	Reynolds Dust +8	Reynolds Dust -8	**Blend**
	Percent	13.6	66.4	13	2.66	4.34	**100**
Design	Itt	2.647	2.580	2.663	2.647	2.626	**2.603**
	C-mid	2.615	2.565	2.637			
	Mn/DOT	2.626	2.574	2.649	2.647	2.613	**2.594**
	Maplewood	2.625	2.638	2.646	2.650	2.650	**2.638**
	Univ. of MN	2.623	2.623	2.626	2.583	2.649	**2.623**
Prod - 1	C-Itt	2.615	2.568	2.634	2.624	2.585	**2.585**
	Mn/DOT	2.566	2.566	2.650	2.615	2.580	**2.579**
	Maplewood	2.626	2.623	2.648	2.632	2.627	**2.627**
	Univ. of MN	2.623	2.587	2.630	2.583	2.649	**2.600**
Prod - 2	C-Itt	2.628	2.587	2.635	2.622	2.630	**2.601**
	Mn/DOT	2.585	2.585	2.652	2.626	2.626	**2.596**
	Maplewood	2.626	2.606	2.650	2.614	2.629	**2.616**
	Univ. of MN	2.623	2.607	2.637	2.583	2.649	**2.614**
	AVERAGE	**2.618**	**2.593**	**2.643**	**2.619**	**2.626**	**2.606**
						U of M Avg:	**2.613**
Superpave Tests (U of MN)							
Coarse Agg. Angularity (1 f.f./2 f.f.)		47/31		80/63			**63/47**
Flat/Elongated Particles		15		11			**13**
Fine Agg. Angularity			39.9		46.7	46.7	**40.5**
Sand Equivalent			75		75	75	**75**

Detroit Lakes I

TH-59

Project 7505-18

	AC Content =	6.2	TSR Data:		VMA Data:	
Marshall	Density (pcf) =	145.3	U of Mn:	72.7	Design	14.5
	Design BSG =	2.332	Contractor:	71.0	U of Mn	14.3
	Design VMA =	14.2				

VMA Required:

Minimum VMA =

Aggregate Bulk Specific Gravity		Moser BA Sand +4	Moser BA Sand -4	Moser Rock Fines +4	Moser Rock Fines -4	Moser Rock	Johnson Rock	TH-59 RAP +4	TH-59 RAP -4	Blend
	Percent	3.3	29.7	0.6	4.4	6	6	7.5	42.5	100
Design	Contractor	2.566	2.566	2.607	2.588	2.634	2.640	2.535	2.509	2.548
	Mn/DOT	2.443	2.505	2.600	2.562	2.666	2.669	2.603	2.403	2.486
	Maplewood	2.618	2.466	2.490	2.567	2.655	2.664	2.610	2.407	2.481
	AVERAGE	**2.542**	**2.512**	**2.566**	**2.572**	**2.652**	**2.658**	**2.583**	**2.440**	**2.506**

Detroit Lakes II

TH-75

Project 8408-41

	AC Content =	5.7	TSR Data:	
	Marshall Density (pcf) =	145.8	U of Mn:	84.6
	Design BSG =	2.340	Contractor:	79.9
VMA Required:	Design VMA =	14.7	Mn/DOT:	85.3
Minimum VMA =				

Aggregate Bulk Specific Gravity		Schulstad Dirty +4	Schulstad Dirty -4	Schulstad Clean +4	Schulstad Clean -4	Kittleson 3/4" Rock	Kittleson 1/2" Rock	**Blend**
	Percent	7.48	26.52	9.75	29.25	20	7	**100**
Design	Contractor	2.569	2.573	2.572	2.574	2.642	2.593	**2.588**
	Mn/DOT	2.583	2.516	2.576	2.478	2.648	2.648	**2.550**
	Maplewood	2.581	2.585	2.571	2.600	2.680	2.627	**2.609**
	Midwest	2.583	2.578	2.576	2.553	2.646	2.666	**2.589**
	Univ. of MN	2.579	2.508	2.574	2.520	2.626	2.610	**2.553**
Prod - 1	Mn/DOT	2.627	2.531	2.573	2.526	2.645	2.549	**2.564**
Prod - 2	Mn/DOT	2.599	2.516	2.593	2.523	2.680	2.687	**2.575**
	AVERAGE	**2.589**	**2.544**	**2.576**	**2.539**	**2.652**	**2.626**	**2.575**
							U of M Avg:	**2.553**
Superpave Tests (U of MN)								
Coarse Agg. Angularity (1 f.f./2 f.f.)		61/42				63/54	82/67	**66/54**
Flat/Elongated Particles		7				9	5	**8**
Fine Agg. Angularity			39.2		39.3			**39.3**
Sand Equivalent			54		72			**63**

Mankato

TH-169

Project 5209-58

VMA Required: 14.5 with 1.2% allowed for drop
Minimum VMA = 13.3

AC Content =	5.3
Marshall Density (pcf) =	145.0
Design BSG =	2.327
Design VMA =	14.5

TSR Data:	
U of Mn:	81.7
Contractor:	
Mn/DOT:	

Aggregate Bulk Specific Gravity		Woelpern 1/2" BA	Kasota 3/4"Add	Kasota Man. Sand	Swenson 3/8" Screen	Blend
	Percent	46	24.0	14.0	16.0	100.0
Design	C - SMC	2.603	2.534	2.519	2.620	2.577
	S-D	2.483	2.513	2.433	2.598	2.501
	S-CO	2.586	2.507	2.615	2.600	2.573
	Univ. of MN	2.517	2.493	2.538	2.614	2.529
	Average	**2.547**	**2.512**	**2.526**	**2.608**	**2.545**
					U of M Avg:	**2.529**
Superpave Tests (U of MN)						
Coarse Agg. Angularity (1 f.f./2 f.f.)			99/90			**99/90**
Flat/Elongated Particles			10			**10**
Fine Agg. Angularity		37.9		46.8	43.1	**40.6**
Sand Equivalent		53		43	90	**59**

Metro

TH-52

Project 1906-40

	AC Content =	5.3	TSR Data:	
Marshall	Density (pcf) =	148.6	U of Mn:	72.0
	Design BSG =	2.386	Contractor:	71.5
	Design VMA =	16.1	Mn/DOT:	77.6

VMA Required: 14.5 with 1.2% allowed for drop
Minimum VMA = 13.3

Aggregate Bulk Specific Gravity		Solberg 3/8" BA	Kraemer 3/4" Rock	Ninnger Ag-Lime	Fischer Washed Sand	TH 52 Millings CA	TH 52 Millings FA	**Blend**	Moisture Test TSR
	Percent	20.0	20.0	17.0	13.0	4.5	25.5	**100**	
Design (Gsb)	Contractor	2.640	2.689	2.811	2.660	2.678	2.678	**2.692**	71.5
	Maplewood	2.621	2.691	2.692	2.631	2.668	2.663	**2.661**	
	Univ. of MN	2.640	2.656	2.753	2.619	2.650	2.650	**2.662**	
Prod - 1	Contractor	2.651	2.687	2.670	2.654	2.685	2.667	**2.667**	72.6
	Maplewood	2.641	2.674	2.685	2.580	2.698	2.643	**2.650**	77.6
Prod - 2	Contractor	2.659	2.679	2.680	2.661	2.686	2.676	**2.672**	77.4
	Maplewood	2.628	2.652	2.707	2.633	2.717	2.667	**2.661**	79.2
Prod -3	Contractor	2.663	2.685	2.683	2.654	2.684	2.680	**2.675**	74.8
	Maplewood	2.617	2.644	2.713					72.4
	AVERAGE	**2.640**	**2.673**	**2.710**	**2.637**	**2.683**	**2.666**	**2.667**	**75.1**
							U of M Avg:	**2.662**	
Superpave Tests (U of MN)									
Coarse Agg. Angularity (1 f.f./2 f.f.)			91/83					**91/83**	
Flat/Elongated Particles			12					**12**	
Fine Agg. Angularity		38.6		47.1	38.9			**41.6**	
Sand Equivalent		89		66	97			**83**	

Rochester I

TH-61

Project 2514-89

AC Content =	5.4	TSR Data:		
Marshall Density (pcf) =	146.6	U of Mn:	86.6	
Design BSG =	2.353	Contractor:	68.5	
Design VMA =	15.8	Mn/DOT:	85.5	

VMA Required: 14.5 with 1.2% allowed for drop

Minimum VMA = 13.3 Design VMA = 14.5

Aggregate Bulk Specific Gravity		RAP - C	RAP - F	Keller 3/4" lime	3/4" Bryan Crushed	RS & G Man san	5/16" R S & G	**Blend**	Moisture Test TSR
	Percent	5.2	14.8	15.0	47.0	13.0	5.0	**100.0**	TSR
Submitted		2.676	2.590	2.672	2.625	2.698	2.708	**2.643**	68.5
Design	C	2.681	2.609	2.635	2.598	2.576	2.708	**2.612**	92.8
	S-D	2.688	2.557	2.647	2.629	2.678	2.708	**2.634**	76.8
	S-CO	2.659	2.605	2.637	2.622	2.678	2.708	**2.635**	73.4
	Univ. of MN	2.676	2.590	2.569	2.622	2.619	2.660	**2.613**	
Prod - 1	C								73.4
	S-D	2.694	2.688	2.650	2.627	2.714			63.8
	S-CO	2.675	2.668	2.644	2.623	2.662			
Prod - 2	C			2.586	2.619		2.725		68.5
	S-D								95.3
	S-CO			2.618					75.4
Prod - 3	S-D			2.618					84.6
	S-CO								86.6
	AVERAGE	**2.678**	**2.615**	**2.628**	**2.621**	**2.661**	**2.703**	**2.627**	78.1
							U of M Avg:	**2.613**	
Superpave Tests (U of MN)									
Coarse Agg. Angularity (1 f.f./2 f.f.)				82/66	86/75		84/68	**85/72**	
Flat/Elongated Particles				11	15		23	**15**	
Fine Agg. Angularity						40.8		**40.8**	
Sand Equivalent						81		**81**	

Rochester II

TH-14 AC Content = 6.0 TSR Data:

Project 5503-28 Marshall Density (pcf) = 145.2 U of Mn: 84.8

Design BSG = 2.330 Contractor: 88.5

VMA Required: 14.5 with 1.2% allowed for drop Design VMA = 16.1 Mn/DOT: 86.4

Minimum VMA = 13.3 Design VMA = 15.9

Aggregate Bulk Specific Gravity		RS&G 5/16" Limestone	RS&G Man. Washed Sand	RS&G Natural Sand	RS&G 1/2" Limestone	3/4" RAP - C	3/4" RAP - F	3/4" Holst	**Blend**	Moisture Test
	Percent	5.0	18.0	28.0	19.0	15.0	15.0	0.0	**100.0**	TSR
Submitted (G_{sb})		2.691	2.678	2.626	2.540	2.596	2.596	2.688	**2.612**	
Design	C	2.588	2.523	2.544	2.504	2.564	2.593	2.658	**2.545**	86.4
	S-D	2.691	2.678	2.626	2.547	2.588	2.501	2.689	**2.598**	82.7
	S-CO	2.634	2.600	2.634	2.536	2.613	2.613	2.705	**2.602**	92.2
	Univ. of MN	2.665	2.584	2.629	2.504	2.583	2.583	2.674	**2.584**	
Prod - 1	C	2.703	2.650	2.624	2.505	2.358	2.419	2.680	**2.534**	89.6
	S-D	2.747	2.664	2.658	2.533	2.587	2.648	2.684	**2.626**	90.9
	S-CO	2.639	2.549	2.598	2.545	2.587	2.629	2.692	**2.584**	92.2
Prod - 2	C	2.740	2.677	2.630	2.516	2.326	2.402	2.680	**2.535**	
	S-D	2.530	2.686	2.661	2.532	2.584	2.638	2.688	**2.618**	
	S-CO	2.676	2.606	2.624	2.550	2.570	2.561	2.701	**2.591**	91.2
	AVERAGE	**2.664**	**2.627**	**2.623**	**2.528**	**2.541**	**2.562**	**2.685**	**2.585**	**89.3**
								U of M Avg:	**2.584**	
Superpave Tests (U of MN)										
Coarse Agg. Angularity (1 f.f./2 f.f.)					96/89			82/67	**96/89**	
Flat/Elongated Particles					13			11		
Fine Agg. Angularity		44	42.3	39.7					**41.0**	
Sand Equivalent		66	88	84					**84**	

Wilmar I

TH-14 AC Content = 6.3

Project 4201-36 Marshall Density (pcf) = 145.4

Design BSG = 2.334

VMA Required: Design VM A = 16.1

Minimum VMA = Design VMA = 14.8

Aggregate Bulk Specific Gravity		Mitzner Fines	Mitzner Coarse	Meridian GF CA 70	Marshall S&G Washed Sand	**Blend**
	Percent	50.0	20.0	15.0	15.0	**100**
Design	Contractor	2.577	2.614	2.677	2.626	**2.606**
	Mn/DOT	2.578	2.613	2.686	2.630	**2.608**
	Maplewood	2.608	2.607	2.658	2.610	**2.615**
	Univ. of MN	2.578	2.600	2.663	2.605	**2.599**
	AVERAGE	**2.585**	**2.609**	**2.671**	**2.618**	**2.607**
					U of M Avg:	**2.599**
Superpave Tests (U of MN)						
Coarse Agg. Angularity (1 f.f./2 f.f.)			82/71	100/100		**90/83**
Flat/Elongated Particles			7	14		**10**
Fine Agg. Angularity		38.8			39.8	**39.0**
Sand Equivalent		57			87	**64**

Wilmar II

TH-15

Project 4303-22

AC Content = 6.2

Marshall Density (pcf) = 144.8

Design BSG = 2.320

Design VMA = 16.2

VMA Required: 16.0 with 1.2% allowed for drop

Minimum VMA = 14.7

Aggregate Bulk Specific Gravity		Duffy - BA fines	Weckman 1/2" clear	Carver Washed sand	Duffy 3/4" rock	**Blend**	TSR No Antistrip	TSR With Anti strip
	Percent	72.0	20.0	8.0	0.0	**100.0**		
Design	C	2.583	2.624	2.638	2.611	**2.595**	64.4	74.4
	S-D	2.571	2.610					76.8
	S-CO	2.606	2.617	2.617	2.627	**2.609**		
	U of Mn	2.539	2.598	2.647	2.627	**2.559**		
Prod-1	C	2.565	2.585	2.652	2.614	**2.576**		
	S-D	2.573	2.610	2.646	2.625	**2.586**		
	S-CO			2.642	2.643			
Prod-2	C	2.581	2.592	2.642	2.620	**2.588**		
	S-D	2.569	2.599	2.655	2.620	**2.582**		
	S-CO	2.567			2.621			
Prod-3	C	2.569	2.598	2.640	2.625	**2.580**		
	S-D	2.575	2.616	2.676	2.627	**2.591**		
	S-CO			2.648	2.595			
	U of Mn							
Average		**2.573**	**2.605**	**2.646**	**2.621**	**2.585**	**64.4**	**75.6**
0					**U of M Avg.**	**2.559**		
Superpave Tests (U of MN)								
Coarse Agg. Angularity (1 f.f./2 f.f.)			99/96		71/57	**99/96**		
Flat/Elongated Particles			12		8	**12**		
Fine Agg. Angularity		40.3		38.8		**40.2**		
Sand Equivalent		81.0		100.0		**83**		

APPENDIX C

GRADATION ANALYSIS

Bemidji I
TH-92
Project 1506-30

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	99.5
5/8"	16.0	98.0	98.0
1/2"	12.5	94.0	93.9
3/8"	9.5	89.0	89.2
#4	4.75	75.0	77.8
#10	2.00	58.0	64.0
#40	0.425	22.0	30.0
#200	0.075	5.0	2.5

Bemidji II
TH-32
Project 6007-10

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	99.5
5/8"	16.0	98.0	98.0
1/2"	12.5	94.0	93.9
3/8"	9.5	89.0	89.2
#4	4.75	75.0	77.8
#10	2.00	58.0	64.0
#40	0.425	22.0	30.0
#200	0.075	5.0	2.5

Detroit Lakes I
TH-59
Project 7505-18

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	100.0
5/8"	16.0	99.0	100.0
1/2"	12.5	93.0	93.6
3/8"	9.5	88.0	86.8
#4	4.75	77.0	72.4
#10	2.00	61.0	55.0
#40	0.425	19.0	19.0
#200	0.075	7.0	2.8

Detroit Lakes II
TH-75
Project 8408-41

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	100.0
5/8"	16.0	97.0	100.0
1/2"	12.5	86.0	100.0
3/8"	9.5	76.0	83.4
#4	4.75	57.0	66.0
#10	2.00	43.0	48.0
#40	0.425	14.0	20.0
#200	0.075	3.0	2.5

Mankato
TH-169
Project 5209-58

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	100.0
5/8"	16.0	100.0	100.0
1/2"	12.5	95.0	95.5
3/8"	9.5	88.0	85.0
#4	4.75	71.0	66.3
#10	2.00	57.0	47.0
#40	0.425	29.0	29.0
#200	0.075	7.0	4.0

Metro
TH-52
Project 1906-40

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	100.0
5/8"	16.0	99.0	100.0
1/2"	12.5	93.0	95.7
3/8"	9.5	85.0	86.6
#4	4.75	72.0	74.6
#10	2.00	58.0	59.0
#40	0.425	24.0	26.0
#200	0.075	6.0	5.3

Rochester I
TH-61
Project 2514-89

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	100.0
5/8"	16.0	95.0	100.0
1/2"	12.5	88.0	94.5
3/8"	9.5	85.0	87.0
#4	4.75	74.0	73.6
#10	2.00	56.0	56.3
#40	0.425	23.0	25.0
#200	0.075	5.0	5.0

Rochester II
TH-14
Project 5503-28

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	100.0
5/8"	16.0	100.0	100.0
1/2"	12.5	100.0	99.7
3/8"	9.5	95.0	95.0
#4	4.75	74.0	76.7
#10	2.00	62.0	64.0
#40	0.425	28.0	36.0
#200	0.075	5.0	5.1

Wilmar I
TH-14
Project 4201-36

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	100.0
5/8"	16.0	100.0	100.0
1/2"	12.5	100.0	100.0
3/8"	9.5	95.0	96.5
#4	4.75	69.0	75.3
#10	2.00	54.0	60.0
#40	0.425	18.0	21.5
#200	0.075	6.0	3.8

Wilmar II
TH-15
Project 4303-22

Sieve Analysis			
Sieve Size		Gradation	
U.S.	mm	Mix Design	Post Ignition
1 1/2"	37.5	100.0	100.0
3/4"	19.0	100.0	100.0
5/8"	16.0	100.0	100.0
1/2"	12.5	100.0	91.9
3/8"	9.5	95.0	80.2
#4	4.75	75.0	63.4
#10	2.00	54.0	46.0
#40	0.425	23.0	17.0
#200	0.075	4.0	2.3

APPENDIX D

STATISTICS

Table D1 Air Void Statistics

Laboratory	*Cell Average, %*	*Cell Standard Deviation, %*	*Cell Deviation, %*	*Coefficient of Variation, %*	*Consistency Statistic, h*	*Consistency Statistic, k*
14 *.	7.48	0.376	0.220	5.03	0.759	1.444
45	7.23	0.493	-0.030	6.81	-0.102	1.889
58	6.67	0.445	-0.598	6.68	-2.059	1.707
66	7.27	0.216	0.004	2.97	0.013	0.8285
40	7.45	0.125	0.190	1.68	0.655	0.4807
47	7.18	0.133	-0.080	1.85	-0.274	0.5098
28	7.44	0.289	0.172	3.89	0.592	1.1083
81	7.85	0.489	0.587	6.23	2.021	1.8750
36	7.48	0.212	0.212	2.83	0.730	0.8124
29	7.18	0.098	-0.080	1.37	-0.274	0.3771
26	6.98	0.325	-0.280	4.65	-0.963	1.2467

* Note: Laboratory 14 was excluded from the overall statistic calculations.

Air Void Statistics, continued

Laboratory	*Cell Average, %*	*Cell Standard Deviation, %*	*Cell Deviation, %*	*Coefficient of Variation, %*	*Consistency Statistic, h*	*Consistency Statistic, k*
19	7.38	0.467	0.120	6.32	0.414	1.7893
73	7.82	0.105	0.552	1.34	1.901	0.4021
42	7.20	0.179	-0.063	2.48	-0.217	0.6861
93	7.12	0.160	-0.146	2.25	-0.504	0.6144
56	7.23	0.288	-0.030	3.97	-0.102	1.1027
33	7.22	0.267	-0.048	3.70	-0.165	1.0250
74	7.60	0.126	0.337	1.66	1.160	0.4851
61	7.05	0.281	-0.213	3.99	-0.733	1.0780
89	7.30	0.210	0.037	2.87	0.127	0.8045
44	7.45	0.339	0.187	4.55	0.644	1.3006
25	6.67	0.234	-0.596	3.51	-2.053	0.8967
87	6.68	0.183	-0.580	2.75	-1.996	0.7037

Air Void Statistics, continued

Laboratory	*Cell Average, %*	*Cell Standard Deviation, %*	*Cell Deviation, %*	*Coefficient of Variation, %*	*Consistency Statistic, h*	*Consistency Statistic, k*
16	7.23	0.137	-0.030	1.89	-0.102	0.5240
79	7.26	0.114	-0.008	1.58	-0.028	0.4391
51	7.12	0.223	-0.146	3.13	-0.504	0.8547
38	7.44	0.264	0.177	3.55	0.609	1.0133
3	7.77	0.121	0.504	1.56	1.734	0.4645
99	7.28	0.214	0.020	2.93	0.070	0.8196
85	7.20	0.155	-0.063	2.15	-0.217	0.5942
4	7.02	0.147	-0.246	2.10	-0.848	0.5645
32	7.40	0.253	0.137	3.42	0.472	0.9703

Average of the Cell Averages, %	7.263
Standard Deviation of the Cell Averages, %	0.290
Repeatability of Standard Deviation, %	0.261
Repeatability Limit	0.730
Reproducibility of Standard Deviation, %	0.376
Reproducability Limit	1.051
F-Value	3.722
Overall Coefficient of Variation, %	4.0

Table D2 Percent Saturation Statistics

Laboratory	Cell Average, %	*Cell Standard Deviation, %*	*Cell Deviation, %*	*Coefficient of Variation, %*	*Consistency Statistic, h*	*Consistency Statistic, k*
14 *	64.10	2.621	-2.532	4.09	-0.460	1.027
45	65.83	5.090	-0.798	7.73	-0.145	1.994
58	68.33	2.479	1.702	3.63	0.309	0.971
66	67.3	2.89	0.70	4.29	0.127	1.1308
40	65.7	1.54	-0.90	2.34	-0.164	0.6018
47	66.1	3.28	-0.56	4.97	-0.103	1.2851
28	69.8	2.91	3.20	4.17	0.582	1.1395
81	65.6	2.59	-1.01	3.95	-0.184	1.0157
36	79.2	0.50	12.60	0.63	2.289	0.1956
29	70.4	1.76	3.80	2.50	0.691	0.6900
26	59.2	5.22	-7.43	8.81	-1.350	2.0433

* Note: Laboratory 14 was excluded from the overall statistic calculations.

Percent Saturation Statistics, continued

Laboratory	**Cell Average, %**	***Cell Standard Deviation, %***	***Cell Deviation, %***	***Coefficient of Variation, %***	***Consistency Statistic, h***	***Consistency Statistic, k***
19	67.2	2.02	0.57	3.01	0.103	0.7922
73	69.8	0.89	3.17	1.27	0.576	0.3482
42	63.6	2.80	-3.03	4.40	-0.551	1.0968
93	70.6	2.25	3.97	3.19	0.721	0.8829
56	66.4	3.71	-0.26	5.59	-0.048	1.4521
33	74.7	0.75	8.04	1.01	1.460	0.2940
74	63.2	2.76	-3.46	4.37	-0.629	1.0815
61	68.5	3.59	1.90	5.24	0.346	1.4065
89	70.0	2.20	3.34	3.14	0.606	0.8603
44	59.3	1.11	-7.33	1.88	-1.332	0.4362
25	68.0	1.00	1.37	1.47	0.249	0.3917
87	65.5	1.80	-1.16	2.75	-0.212	0.7054

Percent Saturation Statistics, continued

Laboratory	Cell Average, %	*Cell Standard Deviation, %*	*Cell Deviation, %*	*Coefficient of Variation, %*	*Consistency Statistic, h*	*Consistency Statistic, k*
16	64.5	1.14	-2.13	1.76	-0.387	0.4449
79	69.5	2.02	2.89	2.91	0.524	0.7914
51	70.3	3.79	3.70	5.38	0.673	1.4830
38	69.6	2.81	2.94	4.04	0.533	1.1012
3	70.9	2.46	4.30	3.47	0.782	0.9629
99	62.4	1.81	-4.20	2.91	-0.763	0.7109
85	65.6	1.50	-1.06	2.28	-0.193	0.5867
4	61.3	0.61	-5.33	0.99	-0.969	0.2383
32	68.75	4.91	1.42	7.14	0.341	1.8195

Average of the Cell Averages, %	67.3
Standard Deviation of the Cell Averages, %	4.2
Repeatability of Standard Deviation, %	2.7
Repeatability Limit	7.6
Reproducibility of Standard Deviation, %	4.7
Reproducibility Limit	13.2
F-Value	7.1
Overall Coefficient of Variation, %	6.2

Table D3 Dry Tensile Strength Statistics

Laboratory	*Cell Average, kPa*	*Cell Standard Deviation, kPa*	*Cell Deviation, kPa*	*Coefficient of Variation, %*	*Consistency Statistic, h*	*Consistency Statistic, k*
14 *	3774.88	149.177	2490.374	3.95	8.157	2.842
45	831.28	77.860	-453.228	9.37	-1.485	1.483
58	1273.23	89.550	-11.274	7.03	-0.037	1.706
66	1063.6	44.00	-220.87	4.14	-0.723	0.8381
40	1168.7	17.52	-115.82	1.50	-0.379	0.3337
47	1507.7	3.75	223.19	0.25	0.731	0.0715
28	2289.7	94.26	1005.24	4.12	3.293	1.7955
81	1210.7	8.80	-73.79	0.73	-0.242	0.1677
36	562.9	3.24	-721.62	0.57	-2.364	0.0616
29	1423.8	70.46	139.26	4.95	0.456	1.3422
26	1403.8	72.57	119.27	5.17	0.391	1.3824
19	1166.1	46.18	-118.37	3.96	-0.388	0.8797

* Note: Laboratory 14 was excluded from the overall statistic calculations.

Dry Tensile Strength Statistics, continued

Laboratory	*Cell Average, kPa*	*Cell Standard Deviation, kPa*	*Cell Deviation, kPa*	*Coefficient of Variation, %*	*Consistency Statistic, h*	*Consistency Statistic, k*
73	1191.0	18.14	-93.55	1.52	-0.306	0.3456
42	1288.6	17.48	4.12	1.36	0.014	0.3330
93	1349.5	8.17	65.03	0.61	0.213	0.1556
56	1307.5	135.04	22.97	10.33	0.075	2.5723
33	1208.9	66.44	-75.63	5.50	-0.248	1.2655
74	880.0	21.10	-404.51	2.40	-1.325	0.4019
61	1077.9	18.00	-206.63	1.67	-0.677	0.3428
89	1061.8	73.40	-222.71	6.91	-0.730	1.3981
44	1610.6	51.37	326.11	3.19	1.068	0.9785
25	1640.3	80.65	355.76	4.92	1.165	1.5361
87	1598.7	21.38	314.16	1.34	1.029	0.4072
16	1115.3	24.96	-169.16	2.24	-0.554	0.4754
79	1131.5	21.80	-153.03	1.93	-0.501	0.4153

Dry Tensile Strength Statistics, continued

Laboratory	***Cell Average, kPa***	***Cell Standard Deviation, kPa***	***Cell Deviation, kPa***	***Coefficient of Variation, %***	***Consistency Statistic, h***	***Consistency Statistic, k***
51	1521.7	17.40	237.17	1.14	0.777	0.3315
38	1575.5	7.17	290.95	0.45	0.953	0.1365
3	1092.4	90.00	-190.47	8.24	-0.622	1.6672
99	1343.6	12.72	59.05	0.95	0.193	0.2423
85	1507.9	16.72	223.38	1.11	0.732	0.3185
4	1169.4	14.33	-115.16	1.23	-0.377	0.2730
32	1194.4	57.98	-90.10	4.85	-0.295	1.1045

Average of the Cell Averages, kPa	1282.8
Standard Deviation of the Cell Averages, kPa	306.2
Repeatability of Standard Deviation, kPa	54.0
Repeatability Limit	151.2
Reproducibility of Standard Deviation, kPa	309.4
Reproducibility Limit	866.3
F-Value	96.5
Overall Coefficient of Variation, %	23.9

Table D4 Wet Tensile Strength Statistics

Laboratory	*Cell Average, kPa*	*Cell Standard Deviation, kPa*	*Cell Deviation, kPa*	*Coefficient of Variation, %*	*Consistency Statistic, h*	*Consistency Statistic, k*
14 *	2635.64	575.420	1527.154	21.83	6.521	10.949
45	728.32	34.476	-380.166	4.73	-1.623	0.656
58	1568.79	50.696	460.305	3.23	1.966	0.965
66	966.9	20.88	-141.61	2.16	-0.605	0.3974
40	993.2	52.58	-115.27	5.29	-0.492	1.0006
47	1252.7	34.46	144.23	2.75	0.616	0.6558
28	1666.5	83.31	557.98	5.00	2.383	1.5853
81	1045.0	24.90	-63.47	2.38	-0.271	0.4738
36	532.1	3.46	-576.34	0.65	-2.461	0.0659
29	1236.0	16.85	127.52	1.36	0.545	0.3206
26	1080.4	24.13	-28.07	2.23	-0.120	0.4592
19	1034.0	67.27	-74.50	6.51	-0.318	1.2801

* Note: Laboratory 14 was excluded from the overall statistic calculations.

Wet Tensile Strength Statistics, continued

Laboratory	***Cell Average, kPa***	***Cell Standard Deviation, kPa***	***Cell Deviation, kPa***	***Coefficient of Variation, %***	***Consistency Statistic, h***	***Consistency Statistic, k***
73	992.2	17.93	-116.33	1.81	-0.497	0.3411
42	1088.0	54.97	-20.49	5.05	-0.087	1.0460
93	1143.4	83.52	34.90	7.30	0.149	1.5893
56	1111.2	125.50	2.72	11.29	0.012	2.3880
33	879.5	31.45	-228.94	3.58	-0.978	0.5984
74	785.8	44.40	-322.71	5.65	-1.378	0.8448
61	930.6	34.98	-177.92	3.76	-0.760	0.6657
89	916.8	14.62	-191.71	1.59	-0.819	0.2781
44	1410.7	59.81	302.19	4.24	1.290	1.1381
25	1359.6	54.42	251.16	4.00	1.073	1.0356
87	1322.6	30.79	214.16	2.33	0.915	0.5859
16	985.7	19.99	-122.76	2.03	-0.524	0.3803
79	1065.9	22.37	-42.55	2.10	-0.182	0.4257

Wet Tensile Strength Statistics, continued

Laboratory	*Cell Average, kPa*	*Cell Standard Deviation, kPa*	*Cell Deviation, kPa*	*Coefficient of Variation, %*	*Consistency Statistic, h*	*Consistency Statistic, k*
51	1234.9	27.29	126.37	2.21	0.540	0.5193
38	1300.6	107.05	192.10	8.23	0.820	2.0370
3	984.4	54.14	-122.76	5.50	-0.523	1.0321
99	1185.4	40.32	76.96	3.40	0.329	0.7673
85	1296.9	46.57	188.42	3.59	0.805	0.8862
4	1075.4	76.86	-33.13	7.15	-0.141	1.4624
32	1160.6	22.47	52.14	1.94	0.223	0.4276

Average of the Cell Averages, kPa	1107.6
Standard Deviation of the Cell Averages, kPa	234.6
Repeatability of Standard Deviation, kPa	52.5
Repeatability Limit	146.9
Reproducibility of Standard Deviation, kPa	238.5
Reproducibility Limit	667.8
F-Value	60.0
Overall Coefficient of Variation, %	21.2

Table D5 Tensile Strength Ratio Statistics

Laboratory	Cell Average, %	Cell Standard Deviation, %	Cell Deviation, %	Coefficient of Variation, %	Consistency Statistic, h	Consistency Statistic, k
14 *	69.80	15.229	-15.845	21.82	-2.921	3.773
45	87.57	4.150	1.921	4.74	0.354	1.028
58	81.17	5.680	-4.479	7.00	-0.826	1.407
66	90.9	1.95	5.22	2.15	0.963	0.4831
40	85.0	4.50	-0.66	5.29	-0.122	1.1144
47	83.1	2.27	-2.51	2.73	-0.463	0.5618
28	72.8	3.65	-12.85	5.01	-2.368	0.9041
81	86.3	2.08	0.65	2.41	0.121	0.5155
36	94.5	0.61	8.89	0.65	1.639	0.1516
29	86.8	1.22	1.19	1.41	0.219	0.3027
26	77.2	1.80	-8.45	2.34	-1.557	0.4466
19	88.7	5.77	3.02	6.51	0.557	1.4302
73	83.0	2.14	-2.68	2.58	-0.494	0.5298

* Note: Laboratory 14 was excluded from the overall statistic calculations.

Tensile Strength Ratio Statistics, continued

Laboratory	Cell Average, %	Cell Standard Deviation, %	Cell Deviation, %	Coefficient of Variation, %	Consistency Statistic, h	Consistency Statistic, k
42	84.4	4.25	-1.25	5.03	-0.230	1.0519
93	84.8	6.21	-0.88	7.32	-0.162	1.5375
56	85.0	9.56	-0.65	11.25	-0.119	2.3693
33	72.8	2.60	-12.85	3.57	-2.368	0.6436
74	89.3	5.02	3.69	5.62	0.680	1.2436
61	86.4	3.27	0.72	3.78	0.133	0.8089
89	86.4	1.37	0.72	1.58	0.133	0.3381
44	87.6	3.72	1.92	4.25	0.354	0.9217
25	82.9	3.30	-2.75	3.98	-0.506	0.8167
87	82.7	1.94	-2.91	2.35	-0.537	0.4812
16	88.4	1.82	2.72	2.06	0.502	0.4502
79	94.2	2.01	8.52	2.14	1.571	0.4981
51	81.2	1.75	-4.46	2.15	-0.823	0.4326

Tensile Strength Ratio Statistics, continued

Laboratory	Cell Average, %	Cell Standard Deviation, %	Cell Deviation, %	Coefficient of Variation, %	Consistency Statistic, h	Consistency Statistic, k
38	82.6	6.76	-3.08	8.19	-0.568	1.6758
3	90.2	4.97	4.39	5.51	0.801	1.2289
99	88.2	3.00	2.59	3.40	0.477	0.7433
85	86.0	3.12	0.35	3.63	0.065	0.7723
4	92.0	6.55	6.32	7.13	1.166	1.6237
32	97.2	1.86	11.52	1.91	2.124	0.4603

Average of the Cell Averages, %	85.8
Standard Deviation of the Cell Averages, %	5.5
Repeatability of Standard Deviation, %	4.0
Repeatability Limit	11.3
Reproducibility of Standard Deviation, %	6.4
Reproducibility Limit	17.9
F-Value	5.5
Overall Coefficient of Variation, %	6.4