DURABLE SUPERPAVE HOT-MIX ASPHALT MIXES IN KANSAS

by

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Abstract

A recent study at Kansas State University has shown that asphalt producers in Kansas are producing hot-mix asphalt (HMA) mixtures with lower asphalt contents than those in the job-mix formula. These drier mixtures are thought to be susceptible to moisture. This project evaluated the effect of asphalt content on rutting and moisture resistance of HMA. Two different mixtures and four varying asphalt contents, optimum and lower, were selected. Another large-size mixture with four varying asphalt contents was also studied. The Hamburg Wheel Tracking Device (HWTD) test (TEX-242-F) and the Kansas Standard Test-56 (KT-56), or modified Lottman test, were used to predict moisture damage and rutting potential of these mixes. All specimens tested were prepared with the Superpave gyratory compacter. Results of this study showed the drier mixtures performed better in rutting and were less susceptible to moisture. Asphalt content significantly affects the number of wheel passes in the HWTD test. The study also revealed a weak correlation between asphalt film thickness and performance test results. Thus, the effect of varying asphalt content is nonconclusive from a durability point of view. However, performance simulations using a theoretical model show that very dry mixes in asphalt pavements are likely to have shorter performance lives.

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Dedication

I dedicate this to my parents and teachers who gave financial and moral support.

Chapter 1 - Introduction

1.1 Background

The Kansas Department of Transportation (KDOT) is using Superpave hot-mix asphalt (HMA) mixtures, some of which may be susceptible to moisture damage. Moisture damage is currently evaluated by the Kansas standard test method KT-56. KT-56 closely follows the American Association of State Highway Transportation Officials AASHTO T 283 procedure adopted during Superpave research. KT-56 has minor modification in the conditioning procedure. This test is time consuming, takes four days to run. According to current KDOT specifications for Superpave mixes, it takes two failing tests to shut down the production. This potentially can result in eight days of Superpave mixture production that could be susceptible to stripping. It is to be noted that all of these mixtures satisfied the KT-56 criteria at design asphalt content. Another criticism of the current test procedure is the use of an anti-stripping agent to rectify the low tensile strength ratio (TSR), or to make the mixture meet minimum TSR requirements. Usually, instead of increasing the conditioned strength, the current test procedure lowers the tensile strength of the anti-stripping additive treated unconditioned specimen. This project investigated the moisture resistance of three different Superpave mixtures with reclaimed asphalt pavement (RAP) using the Hamburg Wheel Tracking Device (HWTD) test and KT-56 (resistance of compacted asphalt mixture to moisture-induced damage) tests.

1.2 Problem Statement

A recent study by Gedafa et al. (2010) showed that asphalt producers in Kansas are often producing mixtures with lower asphalt contents than those are in the job-mix formula. Figure 1-1 shows the binder content used in one of the projects constructed on US 77 in Cowley County. In almost all cases, the asphalt content is lower than the design asphalt content specified in the job-mix formula (JMF).

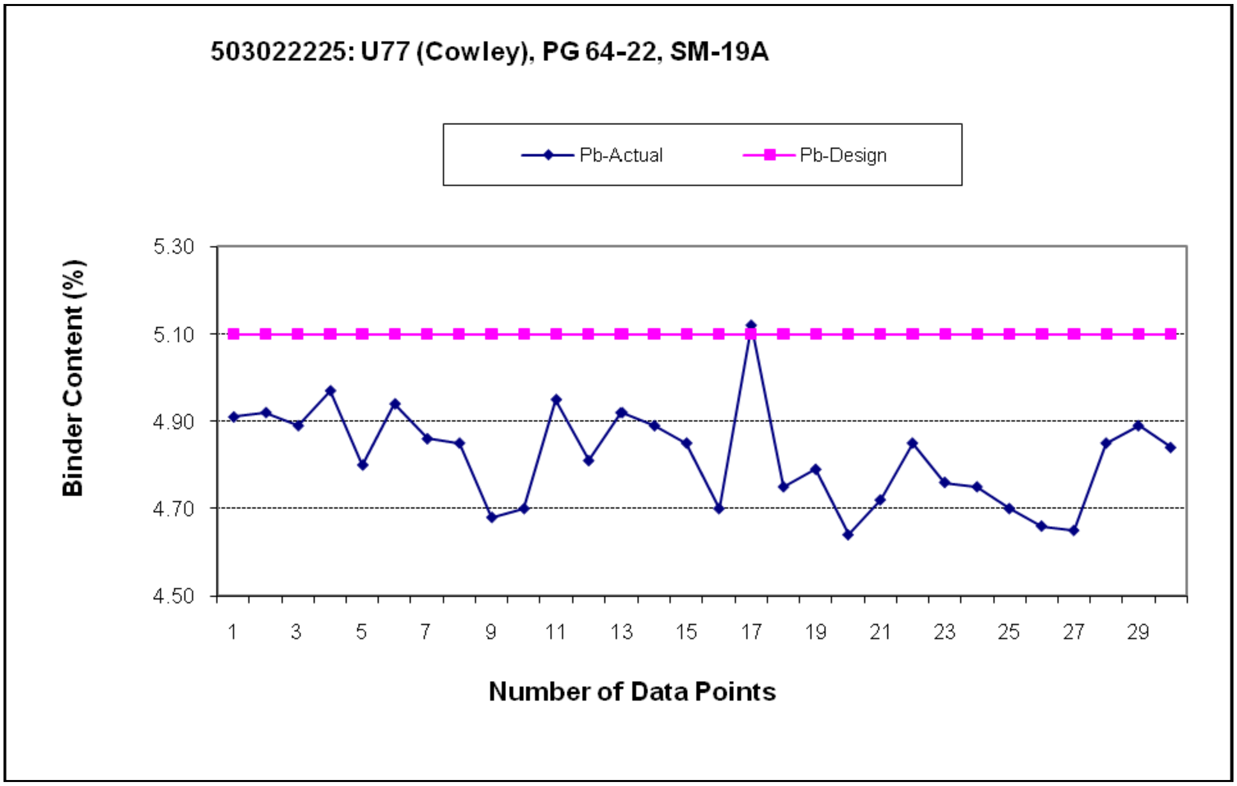


Figure 1-1. Typical comparison of design and actual AC when actual is lower and higher than design (Gedafa et al. 2010).

This “drier” mix has been found responsible for nonconforming moisture-susceptible mixes and sometimes, early cracking. In the recent past, KDOT has taken steps to incorporate more binder into asphalt mixtures that are being produced. These include designing mixtures at 3.5% air voids at N_{design}, lowering the design number of gyrations, etc. The contractors have introduced dust instead of extra binder to achieve lower air voids, yet some mixtures designed with lower N_{design} have also failed the Hamburg Wheel Tracking Device and KT-56 tests. Thus, nothing has seemed to resolve this issue of “dry” mixes. Therefore, it is important to study how these drier mixes will effect pavement performance.

1.3 Objective

There are two objectives of this study:

1. To investigate moisture resistance of Superpave HMA mixtures with varying asphalt content; and

2. To investigate effects of voids in mineral aggregate (VMA) and film thickness on the performance of the mixes, based on results obtained from the performance tests Hamburg Wheel-Tracking Device (HWTD) and Kansas Standard Test KT-56.

Three different Superpave mixture types were selected for this study. These mixtures had been used in past construction seasons.

1.4 Thesis Outline

This thesis consists of six chapters. Chapter 1 presents the introduction, problem, statement of research objective, and thesis outline. Chapter 2 provides a literature review of asphalt content, voids in mineral aggregate, film thickness relating to performance of HMA mixtures, and brief descriptions of HWTD and KT-56 performance test procedures and related research work. Chapter 3 describes materials used in the research and their properties. Test equipment and specimen preparation are also included. Chapter 4 presents results obtained from the HWTD and KT-56 tests. An analysis of the results is also included. Chapter 5 presents the effects of dry mixes on the asphalt pavement life. Chapter 6 summarizes the test results and presents conclusions from this project. Recommendations for future research are also included.

Chapter 2 - LITERATURE REVIEW

2.1 Superpave

About 94% of the paved roads in the United States are asphalt surfaced. The United States has nearly 4,000 asphalt plants producing 500 to 550 million tons of pavement material annually, worth more than $30 billion (National Asphalt Pavement Association-Asphalt Pavement Overview, 2011). Before the introduction of Superpave, the asphalt mixtures were designed using empirical laboratory design procedures, meaning that field experience was necessary to determine if the laboratory analysis correlated with pavement performance (Asphalt Institute, 1995). Superpave stands for superior performing Asphalt pavements. It is the final product of the $50-million Strategic Highway Research Program (SHRP), which represents an improved system for specifying asphalt binders and mineral aggregates, developing asphalt mixture design, and analyzing and establishing pavement performance predictions. The system includes 1) new binder specifications that use new binder physical properties tests like dynamic shear rheometer (DSR), rotational viscometer (RV), bending beam rheometer (BBR), direct tension tester (DTT), rolling thin film oven (RTFO), and pressure aging vessel (PAV); 2) series of aggregate tests and specifications, like coarse and fine aggregate angularity, flat and elongated particles (for coarse aggregate), and sand equivalent test (for fine aggregate); and 3) a hot-mix asphalt (HMA) design using Superpave gyratory compactor (SGC) (Asphalt Institute, 1995). However, the system has also some flaws; one of these is the fact that the design and analysis of asphalt mixture is purely volumetric and the performance of the mixture is evaluated through certain volumetric criteria established under limited conditions with no stability or rut test to verify designed mixes.

In this study, a literature review has been done on the effect of varying asphalt content on the field performance of the mix. Unfortunately, there is not much information published on this topic. However, the effects of voids in mineral aggregate and film thickness on the performance of hot-mix asphalt (HMA) have been reviewed instead.

2.2 Asphalt Content Requirements

In Superpave mix design, the amount of binder required varies depending upon aggregate gradation, angularity, absorption, and viscosity of the asphalt binder. Design asphalt content is established using the Superpave gyratory compactor (SGC) for each aggregate blend. Design asphalt content is selected such that it results in 4% air voids at N_{design}, and all other mixture properties (VMA and VFA at N_{design}, $\%G_{mm}$ at $N_{initial}$, $\%G_{mm}$ at N_{max}, and dust proportion) must meet the requirements at the design asphalt content or else the mixture will need to be redesigned. After designing the mixture, it should be evaluated for moisture susceptibility using the AASHTO T-283 test method. As a part of the quality control (QC)/quality assurance (QA) program, asphalt content is measured during production using the standard extraction method or a nuclear asphalt content gauge. The contractor is allowed a slight variation from the selected design asphalt content to account for inherent material and production variabilities. Currently, the Kansas Department of Transportation (KDOT) allows ±0.6% (single test value) or ±0.3% (4-point moving average value) variation from the design asphalt content mentioned in the job mix formula (Chen and Hossain, 1999).

2.3 Effects of Varying Asphalt Content

Asphalt content plays an important role in the performance of HMA mixtures. It affects mixture stiffness, strength, durability, fatigue life, raveling, rutting, and moisture damage. Insufficient binder in the HMA mix can lead to high permeability, high air voids, and thin asphalt coatings around the aggregates which will cause durability problems (Kandhal et al., 1998). On the other hand, excessive asphalt though durable and flexible, but may cause flushing and low mix stability. “An HMA pavement can ravel and/or crack if it is deficient in asphalt content by as little as ½ percent, whereas ½ percent excessive asphalt content can cause flushing and rutting” (Kandhal and Cross, 1993).

2.4 Durability of HMA Mixture

A mixture is said to be durable when it offers long-term resistance to weathering and aging, and provides good performance without abnormal raveling and cracking of the paved surface (Kumar and Goetz, 1977). Durability can have a significant impact on asphalt concrete mixture performance and significantly change other properties over time. The changes include 1) oxidation and volatilization of asphalt; and 2) disintegration, degradation, and freeze-thaw damage of aggregates. Durability can be controlled by high asphalt contents and proper air voids in the mix design process, which ensures that mixtures will be impervious to air and water. But higher asphalt contents in mixtures are associated with low stability. Therefore, a compromise must be reached between these two confounding properties.

2.5 Stability of HMA Mixture

Stability is defined as the resistance of a mix to permanent deformation under load and is often a concern at high temperatures and slow rates of loading. It largely depends upon the gradation of aggregates. The stability can be evaluated in the laboratory by performing repeated shear tests, frequency sweep shear tests and constant height creep shear tests (Sousa et al., 1991). Another approach to find stability is measuring rut depth using accelerated rut tests like the Hamburg Wheel Tracking Device, the Asphalt Pavement Analyzer, and French Rut Tester (Williams and Prowell, 1999).

2.6 Voids in Mineral Aggregate (VMA)

2.6.1 Definition

Voids in mineral aggregate (VMA) are the volume of intergranular void spaces between the aggregate particles of a compacted paving mixture. This void space includes air voids and effective asphalt content, which is the total asphalt content minus the quantity of asphalt lost to absorption in to the aggregate pores (The Asphalt Institute, 2007). It can be computed from the following equation:

$$VMA=100-\left(\frac{G_{mb}\times P_s}{G_{sb}}\right) \qquad \text{Equation 1}$$

where,

VMA = voids in mineral aggregates,

G_{mb} = bulk specific gravity of compacted mixture,

G_{sb} = bulk specific gravity of aggregate, and

P_s = percent of aggregate.

In Superpave mixture design, air voids, VMA, and VFA are important volumetric parameters in determining the performance of the mixture. Based on the percent air voids, the design binder content is selected. With an increase in binder content, the VMA decreases. HMA mixtures with binder content more than optimum binder content may have fewer air voids and may result in flushing, bleeding, and rutting of pavement. On the other hand, HMA mixtures with less binder content than the optimum binder content will have thinner asphalt film thickness and are susceptible to durability problems. Therefore optimum binder content is selected as corresponding to the minimum value of VMA requirements. The Superpave mix design adopted minimum VMA criteria to ensure the mixture will have adequate binder while at the same time will provide sufficient air voids so there will be no durability, rutting, or bleeding problems. Following are the current VMA requirements from the Superpave mix design.

Table 2-1. Current VMA requirements by Superpave mix design

Nominal Maximum Aggregate Size	Minimum VMA (%)
9.5 mm	15.0
12.5 mm	14.0
19 mm	13.0
25 mm	12.0
37.5 mm	11.0

The problem encountered by highway agencies in implementing the Superpave volumetric design is the difficulty in meeting the minimum voids in mineral aggregate (VMA). One of the contributors is the increased compaction effort by the Superpave gyratory compactor (SGC) (Kandhal et al., 1998). In Superpave mix design, selecting the aggregate gradation that will meet the minimum VMA requirement is the most difficult part of the design process (Anderson and Bahia, 1997). Some researchers have proposed that the specifications for minimum VMA requirements are too restrictive. It should be noted that not all mixes meeting minimum VMA will have acceptable performance. Also, in some cases, the mixes which do not meet minimum VMA, may have acceptable performance but are rejected (Hinrichsen and Heggen, 1996; Li et al., 2009).

2.6.2 Development of VMA

In late fifties, McLeod established criteria for a volumetric property called voids in mineral aggregate (VMA) to ensure that mixture gradation had sufficient air voids and design binder content. He graphically presented a number of charts demonstrating the relationship among density, bitumen content, and void properties of compacted HMA mixtures. The charts include aggregates whose dry bulk specific gravities are between 2.0 and 3.0 and bitumen specific gravity from 0.95 to 1.11, with minimum asphalt content of 4 percent by weight of aggregate and varying absorbed asphalt. McLeod stated minimum asphalt content should be 4.5 percent based on the dry bulk aggregate specific gravity of 2.65 and binder specific gravity of 1.01 (McLeod, 1956). In 1957, McLeod also suggested the volume of voids in a mineral aggregate should not be less than 15 percent, and volume of air voids should not be less than 3 percent or greater than 5 percent. This means there should be a minimum of 10 percent for binder content (4.5 % by weight). He also concluded that compacting a mixture with air voids in the range of 3-5% and minimum VMA of 15% is less restrictive when compared to a VFA of 65-78 % (McLeod, 1957). He graphically showed a VFA range of 65-80% was unachievable for the mixes containing asphalt contents greater than 10.5 percent by weight (20% by volume).

In 1959, McLeod also presented a relationship between the critical minimum VMA and the nominal maximum aggregate size for dense-graded mixtures (McLeod,1959). He stated that with further research experience and additional field data, VMA requirements are subject to change. In 1962 the Asphalt Institute modified Marshall Mix design guidelines by discontinuing VFA requirements and approving the minimum VMA requirements. In 1994, the Asphalt Institute restored the VFA requirements along with the already established minimum VMA requirements (Walter and Coree, 2000). Some older mix design methods such as Marshall, Hveem, etc. have kept the minimum VMA as a recommendation, but Superpave has made VMA as a requirement (Cross and Purcell, 2001).

Foster (1986) evaluated the effects of voids in mineral aggregate on pavement performance. He pointed out that no performance data has been provided in support of the suggested VMA criteria in McLeod’s 1956, 1957 and 1959 papers. Performance data of several projects were collected and compared. From the data, it is observed that 3-5% air voids and VFA of 68-77% will result in acceptable performance (Coree and Hislop , 1999).

Hislop and Coree (1999) also investigated the validity of minimum VMA requirements as a function of nominal maximum aggregate size required in the Superpave mix design. For that study, three different nominal maximum aggregate sizes (19 mm, 12.5 mm, and 9.5 mm) with fine, dense, and coarse gradations and combinations of natural and manufactured coarse and fine aggregates were selected as shown in Table 2-2.

Table 2-2 Experimental Matrix (Hislop and Coree, 1999)

		Nominal maximum aggregate size								
		9.5 mm			12.5 mm			19.0 mm		
CA	FA	Fine	Dense	Coarse	Fine	Dense	Coarse	Fine	Dense	Coarse
Crushed	Manufactured	x	x	x	x	x	x	x	x	x
Natural	Manufactured	x	x	x	x	x	x	x	x	x
50/50	50/50	x	x	x	x	x	x	x	x	x
Gravel	Natural	x	x	x	x	x	x	x	x	x

A total of 36 blends, each containing two specimens at different asphalt contents (4,5,6,7 and 8%) were fabricated using the Superpave gyratory compactor. Measured volumetric properties were compared with VMA and VFA requirements of Superpave mix design criteria. Then, the specimens were physically tested with the Nottingham asphalt tester (NAT), which is widely used in Europe for testing asphalt mixtures. NAT test results showed that out of 36 mixes, five were sound over a range of asphalt content used. The remaining 31 mixes became plastic. Results also indicate that although the minimum VMA requirements, based on nominal maximum aggregates size, seemed reasonable, they were too restrictive because only three mixes met the Superpave minimum VMA criteria (Walter and Coree, 2000).

Hinrichsen and Heggen (1996) calculated minimum VMA using an equation that considers gradation and volumetric properties. Assuming a minimum film thickness, they concluded that VMA criteria, solely on the basis of nominal maximum size of aggregates, is too restrictive and agencies may eliminate a significant percentage of aggregate gradations which otherwise would have acceptable performance (Hinrichsen and Heggen, 1996).

2.7 Asphalt Film Thickness

Asphalt film thickness is a computed parameter and cannot be measured physically. Since the concept of asphalt film thickness emerged in 1940s, different calculation schemes have been developed by researchers. These calculation algorithms are mostly based on the determination of surface area of aggregates. The surface area of aggregate depends on the gradation, because the surface area of fine aggregate per unit weight is more than that of coarse aggregates. Calculation of film thickness is a part of the Hveem mix design procedure. Hveem assumed the specific gravity of aggregates and also that all particles are spherical, so that all the aggregate particles are coated with uniform asphalt thickness (Radovskiy, 2003). In other words, asphalt film thickness is the effective asphalt content divided by the surface area of the aggregate. Average asphalt film thickness is calculated by multiplying surface area factors with percent passing various sieve sizes used (aggregate gradation). Surface area factors adopted by The Asphalt Institute are shown in Table 2-3.

Table 2-3 Surface area factors (Kandhal et al., 1998)

Sieve Size, mm	19	4.75	2.36	1.18	0.6	0.3	0.15	0.075
Surface Area Factor, m^2/kg	0.41	0.41	0.82	1.64	2.87	6.14	12.29	32.77

The equation to find asphalt film thickness where surface area of aggregate is used is given by

$$T_F = \frac{V_{be}}{SA \times M_s}(\rho_w) \qquad \text{Equation 2}$$

where,

T_F = Average Film Thickness (in microns),

V_{be} = Effective Volume of Asphalt Cement (liters),

SA = Specific Surface Area of the Aggregate (m^2/kg),

M_s = Mass of Aggregate (kg), and

ρ_w = Density of Water (gm/cm^3).

It is widely accepted that asphalt film thickness is related to the durability of mixes. Mixes with thick films are known to be durable, while mixtures with thin films are prone to cracking and raveling of pavement (Campen et al., 1959). Thin asphalt coatings along with high permeability and high air voids, will lead to excessive aging of asphalt and cause durability problems (Kandhal et al., 1998).

Minimum VMA requirements, as adopted by Superpave mix design, are sometimes difficult to meet. Some asphalt technologists recommended replacing minimum VMA with an average asphalt film thickness requirement which is a more direct method of ensuring durability (Li et al., 2009). However inaccuracies in film thickness computation are widely reported because approaches in calculating aggregate surface area by the researchers are different. To minimize these inaccuracies, historical data were analyzed and the best-fit criterion based on surface area was suggested by Hinrichsen and Heggen (1996).

Campen et al. (1959) presented the relationship between voids, surface area, film thickness, and stability for dense-graded mixtures. They found that asphalt required to produce minimum aggregate voids does increase with surface area but at a much slower rate than that guided by a relationship of direct proportionality. They also concluded that film thickness decreases with an increase in surface area. From the data analysis and experience, they suggested a film thickness of 6-8 microns for the most desirable HMA mixes (Campen et al., 1959).

Kandhal and Chakraborthy (1996) investigated the effects of asphalt film thickness on short and long-term aging of asphalt paving mixtures. They quantified the relationship between film thickness and aged properties such as tensile strength and resilient modulus of the asphalt paving mixtures. An optimum film thickness of 9-10 microns was found for mixtures compacted at 8% air voids (Kandhal and Chakraborty, 1996). Kandhal et al. (1998) concluded that current VMA requirements adopted by Superpave are inadequate for ensuring durability and recommended an average film thickness of 8 microns (Kandhal et al., 1998).

Xinjun et al. (2009) investigated the relationship between in-place asphalt film thickness and performance of HMA mixtures. They analyzed the flat surface assumption and pointed out that it has significant effect on smaller aggregate sizes (<0.3 mm in diameter or No. 50 sieve). They computed aggregate surface area based on two different methods, surface area method and index method, which also quantifies the effect of aggregate shape. The results indicated that the shape of aggregate and fine aggregate particles significantly affect the calculation. The study concluded that in-place asphalt film thicknesses significantly affect the rutting performance of the HMA mixtures (Li et al., 2009).

In another study, Hmoud (2011) evaluated VMA and film thickness requirements in HMA mixtures. He calculated film thickness using surface area factors developed by Hveem. At optimum asphalt content, average film thickness of surface course mixtures is slightly more than 9 microns, and for base course mixtures it is 9.65 microns. Based on the literature review and results obtained in the study, Hmoud recommended an average film thickness of 8 microns for durability of HMA mixtures (Hmoud, 2011).

2.8 Moisture Sensitivity

The presence of water in HMA pavement is one of the factors that affects its durability. Moisture-induced damage may be associated with two mechanisms, adhesive failure and cohesive failure. In adhesive failure, the water strips the binder from the aggregate surface. In cohesive failure, the presence of water in contact with binder reduces the cohesion within the binder and decreases the stiffness of mixture (Hick et al., 2003).

2.8.1 Stripping

"Stripping is defined as the weakening or eventual loss of the adhesive bond usually in presence of moisture between the aggregate surface and the asphalt cement in HMA mixture" (Kandhal, 1994). Some of the ways in which moisture enters into HMA pavements are inadequate subsurface/surface drainage, run off through road surface, and seepage from ditches and surrounding areas. Strength of HMA depends on the binder (cohesion) and aggregate (interlocking and internal friction). Some of the mechanisms that contribute to stripping include the following:

- Detachment— asphalt cement is separated from the surface area of aggregate by action of a thin film of water, with asphalt being intact.
- Displacement— Removal of asphalt film from the surface area of aggregate with a break in asphalt film.
- Spontaneous emulsification— inverted emulsions of water droplets in asphalt cement. Presence of clays and amines aggravates the emulsification.
- Pore pressure— stresses induced due to presence of water in the pore structure; during traffic loading and freeze-thaw cycle, would cause the pavement to strip.

- Hydraulic scour— similar to pore pressure, occurs in a saturated pavement under vehicle tires, resulting in compressive stress.

Generally, stripping starts at the bottom and progresses upward. It is very difficult to identify stripping because the surface of the pavement exhibits failures like rutting, shoving, corrugations, raveling, and cracking. The only way determine if the pavement is subjected to distress (stripping), is to take cores and observed visually.

Stripping in HMA pavement can be controlled by adding anti-stripping additives to the HMA mixture. The additives may be liquid (mixed with asphalt cement prior to mixing the asphalt cement with the aggregates) or solid (mixed with the aggregate prior to mixing with asphalt cement with aggregate). One of the most commonly used and effective antistripping additives is lime, hydrated lime, or quick lime. Antistripping additives reduce surface tension of the aggregate and asphalt (Brown et al., 2009).

2.9 Test Methods to Predict Moisture Sensitivity of HMA Mixtures

The first moisture-damage test of compacted specimen, immersion-compression, dates back to the 1950s established under ASTM standards**.** Since then many attempts on developing various test methods that can predict moisture sensitivity have been done. Lottman test protocol, which uses vacuum saturation followed by freezing and hot water conditioning, became widely accepted (Solaimanian et al., 2003). Later AASHTO slightly modified this test and named it AASHTO T283. Following adoption of AASHTO T283 by Superpave, many state agencies started using the AASHTO T283 procedure for evaluating moisture sensitivity of mixes. Current Kansas Standard Test KT-56 used in the study is similar to T283 except for some minor modifications in the conditioning process. Moisture-sensitivity tests can be classified into two classes: tests on loose mixtures and tests on compacted mixtures. These test types have been listed in Tables 2-4 and 2-5 respectively.

Table 2-4. Moisture-sensitivity tests on loose samples (Solaimanian et al., 2003)

Test	ASTM	AASHTO	Other
Methylene blue			Technical Bulletin 145, International Slurry Seal Association
Film stripping			(California Test 302)
Static immersion	D1664*	T182	
Dynamic immersion			
Chemical immersion			Standard Method TMH1 (Road Research Laboratory 1986, England)
Surface reaction			Ford et al. (1974)
Quick bottle			Virginia Highway and Transportation Research Council (Maupin 1980)
Boiling	D3625		Tex 530-C Kennedy et al. 1984
Rolling bottle			Isacsson and Jorgensen, Sweden, 1987
Net adsorption			SHRP A-341 (Curtis et al. 1993)
Surface energy			Thelen 1958, HRB Bulletin 192 Cheng et al., AAPT 2002
Pneumatic pull-off			Youtcheff and Aurilio (1997)

* No longer available as ASTM standard.

Table 2-5. Moisture-sensitivity tests on compacted specimens (Solaimanian et al., 2003)

Test	ASTM	AASHTO	Other
Moisture vapor susceptibility			California Test 307 Developed in late 1940s
Immersion–compression	D1075	T165)	ASTM STP 252 (Goode 1959
Marshal immersion			Stuart 1986
Freeze–thaw pedestal test			Kennedy et al. 1982
Original Lottman indirect tension			NCHRP Report 246 (Lottman 1982);Transportation Research Record 515(1974)
Modified Lottman indirect		T 283	NCHRP Report 274 (Tunnicliff and Root1984), Tex 531-C
Tunnicliff–Root	D 4867		NCHRP Report 274 (Tunnicliff and Root1984)
ECS with resilient modulus			SHRP-A-403 (Al-Swailmi and Terrel1994)
Hamburg wheel tracking			1993 Tex-242-F
Asphalt pavement analyzer			
ECS/SPT			NCHRP 9-34 2002-03
Multiple freeze-thaw			

2.9.1 Hamburg Wheel Tracking Device Test

The Hamburg Wheel Tracking Device (HWTD) measures the combined effects of rutting and moisture damage. The HWTD is gaining popularity for its fast and reliable performance of testing various HMA mixes (Yildirim, et al., 2007; Lu & Harvey, 2006).The HWTD was developed in the 1970s by Esso A.G of Helmut-Wind, Inc. in Hamburg, Germany (Romero and Stuart, 1998). This device was introduced in the U.S.A following a European asphalt study tour of a group of pavement engineers and asphalt technologists in 1990 (AASHTO, 1991).

The HWTD test indicates susceptibility to premature failing of HMA mixtures due to weak aggregate structure, inadequate binder stiffness, moisture damage, and inadequate adhesion between aggregate and binder. It is observed that HWTD results are influenced by aggregate quality, binder stiffness, duration of short-term aging, binder source, anti-stripping treatments, and compaction temperature (Aschenbrener, 1995; Aschenbrener, 1994; Aschenbrener & Far, 1994).

This device was built based on a similar British device, which uses rubber tires instead of steel wheels. The device is operated by moving a pair of reciprocating steel wheels across the surface of HMA specimens (cylindrical or slab/cubicle) submerged in hot water, generally held at $50^0C^.$ The device is capable of testing a pair of specimens simultaneously. The specimens are compacted to 7±1 percent air voids. The steel wheels have a diameter of 203 mm (8 inches) and width of 47 mm (1.85 inches) and are capable of making 53±2 passes per minute. Each steel wheel weighs 158 lbs. Typical length of the slabs are 320 mm (12.6 inches) long by 260 mm (10.2 inches) wide, thickness varies from 40 mm (1.6 inches) to 80 mm (3.2 inches) and dimensions of the cylindrical specimens are 150 mm (6 inches) in diameter and 62 mm (2.5 Inches) in height as shown in Figure 2-1. Linear variable differential transformers (LVDTs) measure rut depth or deformation at 11 different points along the length of each specimen. The LVDTs measure rut depth at an accuracy of 0.01 mm. The device automatically ends the test when the preset numbers of wheel passes are reached or a rut depth of 20 mm (0.8 inch), whichever comes earlier. Duration of the test (considering 20,000 passes) is approximately 7 hours including the initial wait time of 30 minutes. However, in some tests the samples fail early and test times are shorter.

Figure 2-1. (Clockwise) Final test setup of Hamburg Wheel Tracking Device, closeup of samples under the wheel load, samples ready for testing in HWTD, and failed samples having high rut depth.

HWTD test outputs include post compaction consolidation, creep slope, stripping slope, and stripping inflection point as illustrated in Figure 2-2. These parameters are obtained by plotting a curve between rut depth and number of cycles. Post compaction consolidation is the deformation (mm) at 1,000 wheel passes. It is assumed the wheel densifies the mixture within the first 1,000 passes and is named post –compaction consolidation. Creep slope is the inverse of the rate of deformation in the linear region of plot between the post compaction and stripping inflection point (if stripping occurs). Creep slope relates to rutting primarily due to plastic flow. It is the number of wheel passes required to create 1 mm of rut depth. Stripping inflection point and stripping slope are related to moisture resistance of HMA. Stripping inflection point is the number of wheel passes at the intersection of creep slope and stripping slope. Stripping slope is the inverse rate of deformation after the stripping inflection point. It relates to rutting primarily due to moisture damage. It is the number of wheel passes required to create 1 mm of rut depth after stripping inflection point (Yildirim et al., 2007).

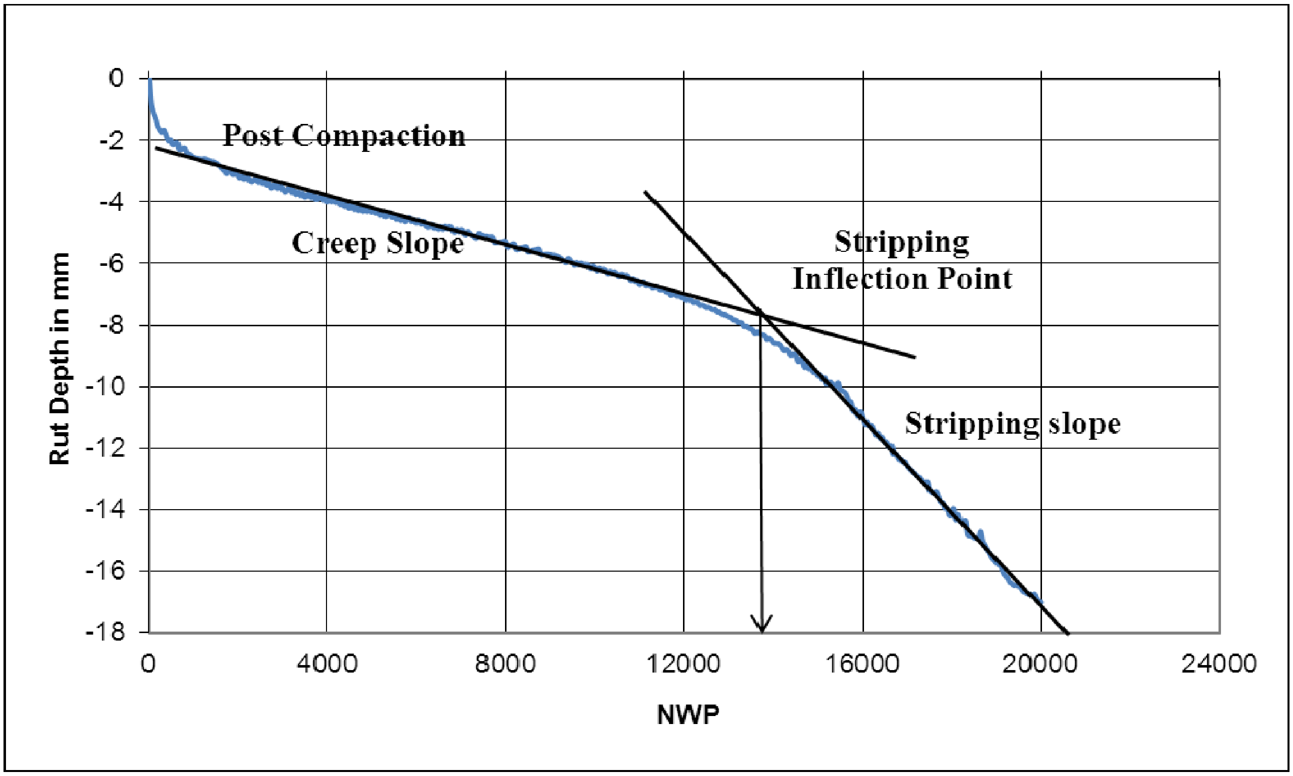


Figure 2-2. Typical Hamburg Wheel Tracking Test results

2.9.1.1 Past Research and Experience

Since the HWTD was introduced in United States, various entities have utilized it for evaluating moisture susceptibility of HMA mixtures. However, the test procedure and specification may vary slightly from one agency to another depending upon the mixture type. For example, Hamburg, Germany, specifies allowable rut depth of less than 4 mm at 20,000 passes. Colorado Department of Transportation (CDOT) uses the test temperature according to the site and specifies a rut depth of less than 10 mm after 20,000 passes (Izzo and Tahmoressi, 1999). The Texas Department of Transportation (TxDOT) follows the TEX-242-F procedure. Requirements for TEX-242-F are listed in Table 2-6.

Table 2-6. Hamburg Wheel Tracking Device test criteria (Zhou et al., 2005)

High-temperature Binder Grade	Number of Wheel Passes	Maximum Rut Depth in mm
PG 64-22	10,000	12.5 mm(0.5 in)
PG 70-22	15,000	12.5 mm(0.5 in)
PG 76-22	20,000	12.5 mm(0.5 in)

Aschenbrener (1995) evaluated factors that influence results from HWTD. He conducted tests on 20 different mixtures whose stripping performance was known and then compared them with test results obtained. Results indicated an excellent correlation existed between results from HWTD and pavements of known field performance. The study concluded that HWTD results are sensitive to quality of aggregate, asphalt cement stiffness, length of short-term aging, refining process, liquid and hydrated lime, and compaction temperature.

Izzo and Tahmoressi (1999) evaluated the HWTD and its capability in assessing moisture susceptibility of HMA in Texas. Six different mixtures were prepared with and without antistripping additives and tested at 40^0c and 50^0c. Mixtures were modified with hydrated lime and liquid antistripping additives. Asphalt binder used for all mixtures was the same (AC-20). For mixtures tested at 40^0c, test results indicated that use of antistripping additives improved the performance of the mixture i.e., mixtures with hydrated lime performed better followed by mixtures modified with liquid antistripping additive, and worst performance was observed for mixtures without any additives. For mixtures tested at 50^0c, results were inconsistent (Izzo and Tahmoressi, 1999).

In another study, Gogula et al. (2003) showed the effect of performance-grade binder and air voids on HWTD results. PG 52-28, PG 64-22, PG 58-28, and PG 70-28 were studied and PG 70-28 performed better. It also indicated mixtures with lower air voids (7%) performed better when compared to mixtures compacted to 2 percent higher air voids (9%).

2.9.2 Kansas Test Method KT-56(Resistance of Compacted Asphalt Mixture to Moisture-induced Damage) or Modified Lottman Test

The KT-56 method is used to evaluate Superpave HMA mixtures susceptible to moisture or stripping. It is commonly known as Modified Lottman Test (Hossain et al., 2011). This test compares the average indirect tensile strength of unconditioned specimens to that of conditioned specimens.

Figure 2-3. (clockwise) Sample loaded in tensile strength machine, closeup of sample in load frame, sample after broken in tensile strength machine.

A total of six specimens are fabricated using the Superpave gyratory compactor. Air voids of these specimens should be 7±0.5 percent. The specimens should be 6 inches (150mm) in diameter and 3.75±0.2 inches (95±5mm) in height. The specimens are divided into two subsets so that the average air voids of both are approximately equal. One subset is kept at room temperature without any conditioning until testing for indirect tensile strength and the other subset is subjected to conditioning. The conditioning process includes a freeze-thaw cycle. Each specimen of this subset is first kept in a vacuum container and using a vacuum pump, has a partial vacuum of 25 to 66 cm (10 to 26 inches) of Hg applied for a short time to bring the specimen saturated to 70 to 80% of air voids. After the specimens are saturated, they are subjected to freezing at a temperature of 0 ± 5^0F ($-18^0\pm3^0$C) for a minimum of 16 hours, followed by a thawing cycle where the specimens are kept at 140 ± 2^0F (60 ± 1^0C) in a water bath for 24±1 hours. The final step in the conditioning process is to keep the specimens in a water bath maintained at a 77 ± 1^0F (25 ± 0.5^0C). Then all specimens are tested for indirect tensile strength at 77 ± 1^0F (25 ± 0.5^0C) at a loading rate of 2 inches per minute (51mm per minute), and the corresponding peak loads and displacements are recorded. The ratio of tensile strength of conditioned subset to the unconditioned subset is calculated. This is called tensile strength ratio and should be a minimum of 0.8 (or 80%) as adopted by the Superpave mix design and KDOT. Tensile strength is given by the following equation:

$$S_t = \frac{2000 \times P}{\pi \times t \times D} \qquad \text{Equation 2.2}$$

where S_t = tensile strength, Psi (kPa),

P = maximum load, lbf (N),

t = specimen thickness in (mm), and

D = specimen diameter in (mm).

The tensile strength ratio (TSR) in percent is calculated as follows

$$TSR = \frac{T_1}{T_2} \times 100 \qquad \text{Equation 2.3}$$

where TSR = tensile strength ratio,

T_1 = average tensile strength of unconditioned subset, and

T_2 = average strength of conditioned subset.

Chapter 3 - Methodology

3.1 Aggregates and Binder Sources

Three different mixes were evaluated in this study. SR-19A, SR-12.5A, and SR-9.5A were collected from different projects in Kansas, Phillips County, Republic County, and Kingman-Sumner County, respectively. The binder used was PG 58-28 for all mix types. Liquid antistripping additive (Arr Maz HP^{+}) was also used in all mixtures. The following table provides the sources of aggregates and binder for each project.

Table 3-1. Source of aggregates and binder

Project Number	Mix Type	Aggregate Source	Binder (PG 58-28) Source
36-74 KA-1734-01	SR-19A	Phillips County	Valero, Sherin
079 KA 1380-01	SR-12.5A	Republic County	Flint Hills, Omaha
42-106 KA 1461-01	SR-9.5A	Kingman-Sumner County	Murphy Oil (NE)

3.2 Aggregate Tests

Aggregates, major components of the HMA mixture, constitute 92 to 96% of the mixture by weight or 80 to 85 percent of the mixture by volume. The following aggregate tests were performed on all aggregates brought to the laboratory:

- gradation analysis (KT-2);
- material finer than US No. 200 sieve by wash-sieve analysis (KT-3);
- specific gravity of fine aggregates (Pycnometer method) for aggregates passing through a No. 4 sieve (ASTM C128); and
- specific gravity of coarse aggregate (KT-6) for aggregates retained on a No. 4 sieve.

3.3 Gradations and Blending

Aggregate gradation or the distribution of aggregate particle size is the most important characteristic to be considered in the asphalt mix design process. One of the important volumetric properties, voids in the mineral aggregate (VMA), is calculated based on the aggregate gradation. Some of the properties influenced by aggregate gradation are stability, durability, stiffness, workability, skid resistance, permeability, and resistance to moisture damage (Roberts et al., 1996). Superpave gradation requirements include use of the Federal Highway Administration (FHWA) 0.45- power chart, which is based on the Fuller gradation formula. Performance of the gradation is evaluated based on the maximum density line of the 0.45 power chart. The maximum density line is obtained by drawing a straight line from origin to the maximum aggregate size. Figures 3-1, 3-2, 3-3 and Tables 3-2, 3-3, 3-4, 3-5, 3-6, and 3-7 show the combined aggregate gradations of all mixtures.

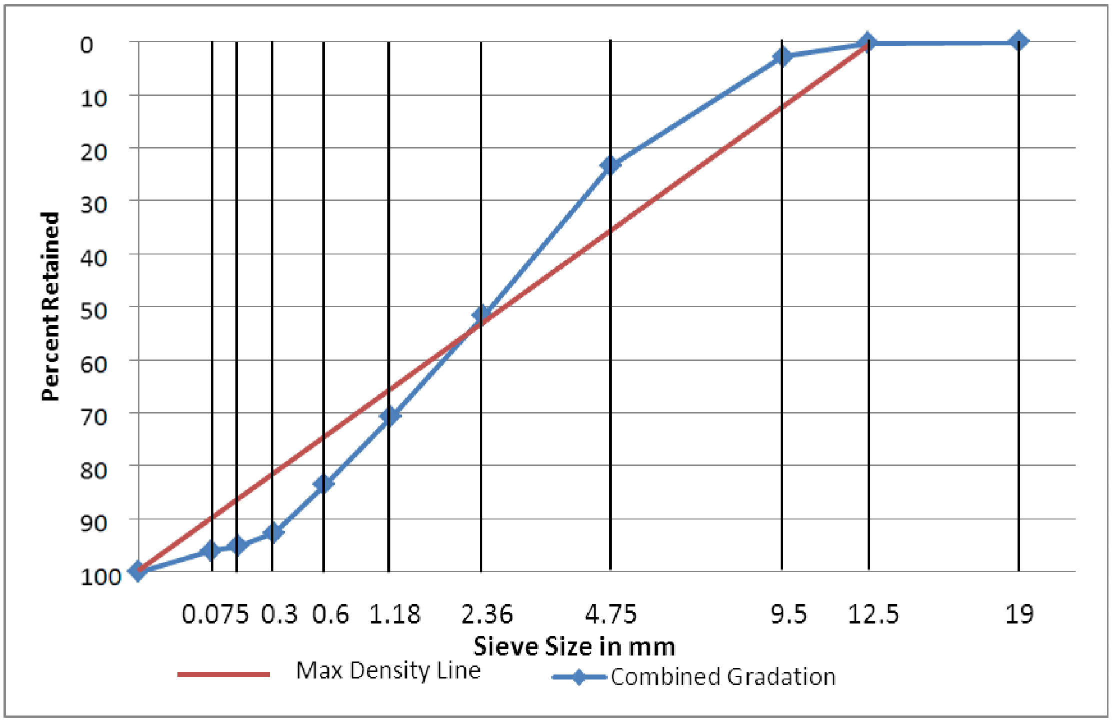


Figure 3-1. 0.45-power chart of SR 9.5A Superpave mixture.

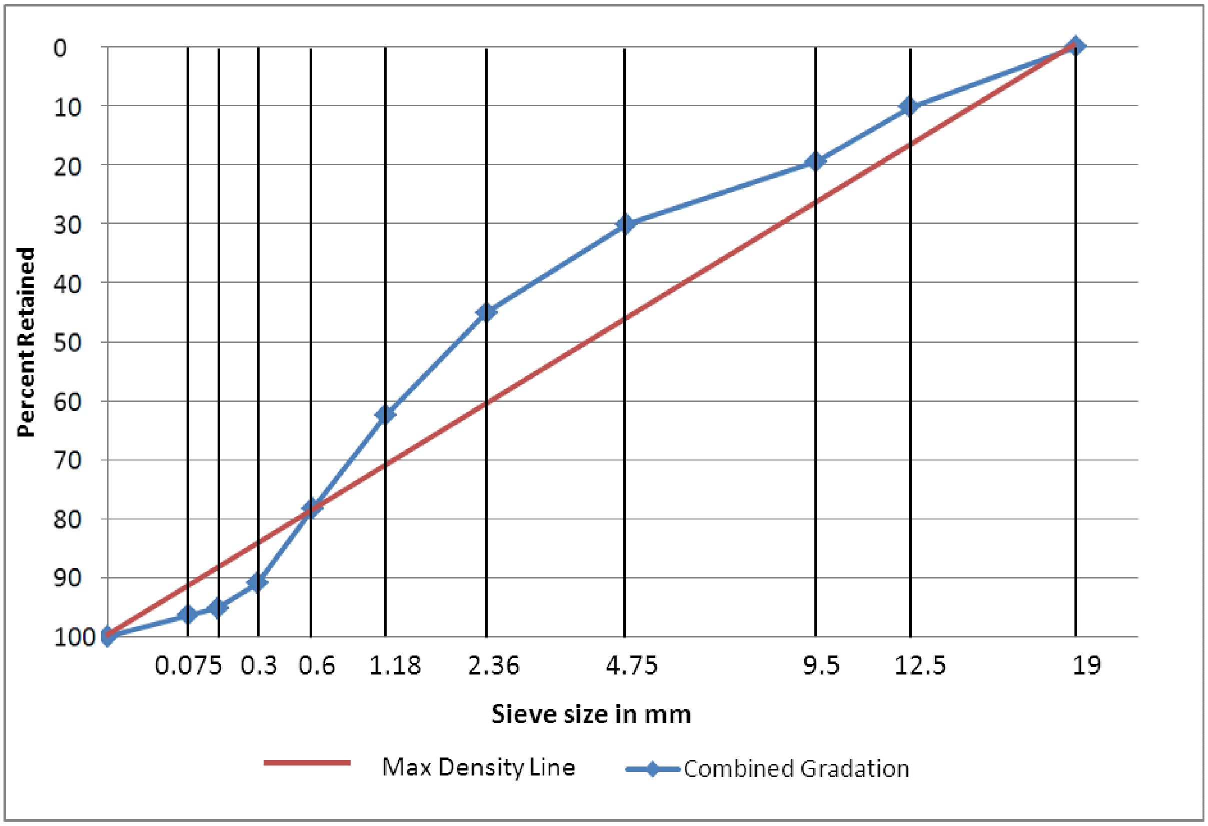


Figure 3-2. 0.45-power chart of SR 12.5A Superpave mixture.

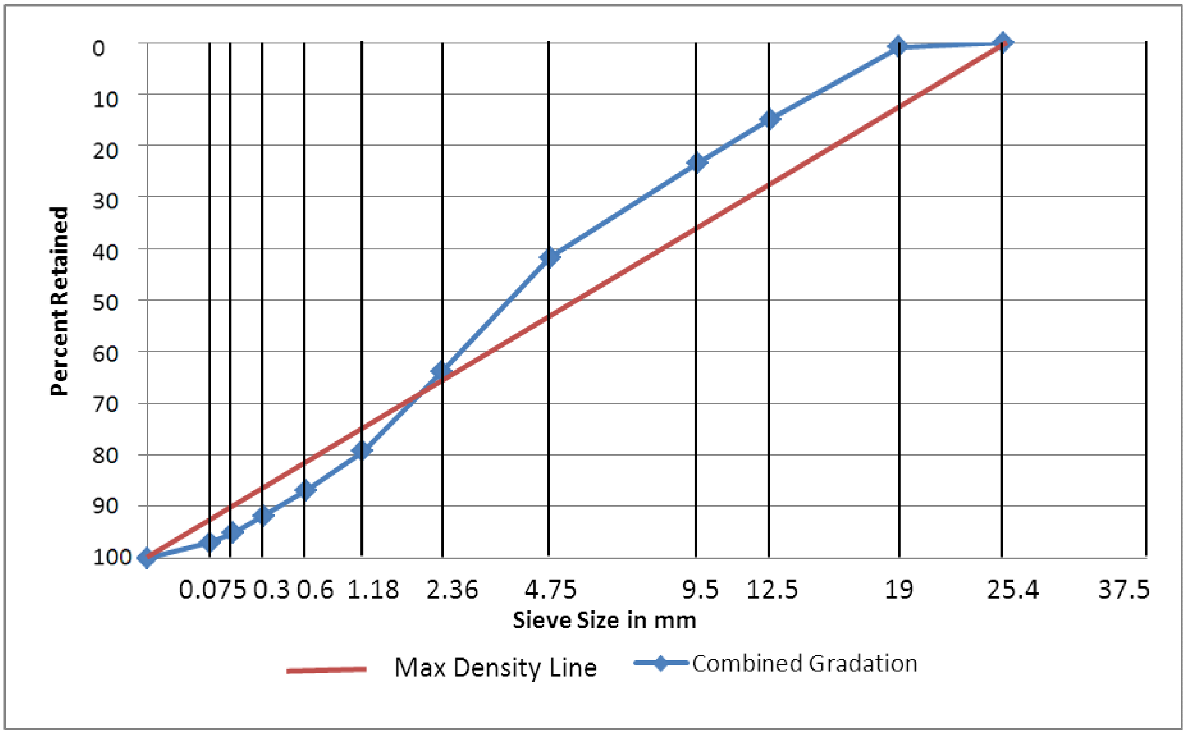


Figure 3-3. 0.45-power chart of SR 19A Superpave mixture.

Table 3-2. Blending of aggregates for SR 9.5A mixture

Sieve Size	Percent of Total Mix					Final Blend used	Superpave Specification
	17	12	11	35	25		
	CS-1	CS-2	CS-2A	SSG-1	RAP		
1/2 in.	0.0	0.0	0.0	0.0	0.3	0	0
3/8 in.	0.9	0.0	0.0	0.4	1.6	3	0-10
No. 4	12.0	1.2	0.2	4.1	6.1	23	10 Min
No. 8	16.6	3.8	6.6	13.6	11.2	52	33-53
No. 16	16.7	6.0	9.6	22.5	16.0	71	
No. 30	16.7	7.4	10.2	28.7	20.6	84	
No. 50	16.7	8.5	10.4	33.5	23.6	93	
No. 100	16.7	9.1	10.5	34.6	24.4	95	
No. 200	16.7	9.4	10.5	34.7	24.8	96.1	90-98

Table 3-3. Blending of aggregates for SR 12.5A mixture

Sieve Size	Percent of Total Mix					Final Blend used	Superpave Specification
	15	5	20	35	25		
	CS-1	CS-2	SSG-1	CG-5	RAP		
1/2 in.	6.6	0.0	0.1	0.0	3.5	10	0-10
3/8 in.	12.8	0.1	0.6	0.0	5.7	19	10 min
No. 4	14.8	1.6	2.0	2.0	9.8	30	
No. 8	14.8	2.6	4.6	7.9	15.1	45	42-61
No. 16	14.8	3.2	8.6	17.0	18.9	63	
No. 30	14.8	3.6	13.2	24.8	22.0	78	
No. 50	14.8	3.8	17.9	30.3	24.2	91	
No. 100	14.8	4.0	19.4	32.3	24.8	95	
No. 200	14.8	4.1	19.5	33.0	25.0	96.4	90-98

Table 3-4. Blending of aggregates for SR 19A mixture

Sieve Size	Percent of Total Mix					Final Blend used	Superpave Specification
	18	18	12	27	25		
	CG-1	CG-2	CG-2A	SSG-1	RAP		
1 in.	0.1	0.0	0.0	0.0	0.8	1	0-10
1/2 in.	10.5	0.0	0.0	1.4	3.0	15	10 min
3/8 in.	15.4	0.0	0.0	3.1	4.8	23	
No. 4	17.8	3.2	1.4	8.0	11.3	42	
No. 8	17.9	7.2	4.8	16.0	18.0	64	51-65
No. 16	17.9	9.9	7.1	22.5	21.9	79	
No. 30	17.9	11.7	8.6	24.9	23.8	87	
No. 50	17.9	13.3	9.8	26.2	24.6	92	
No. 100	17.9	14.8	11.0	26.7	24.9	95	
No. 200	17.9	15.9	11.5	26.7	25.0	97	92-98

Table 3-5. Aggregate gradation of SR 9.5A mixture

Sieve Size	Aggregate designation				
	CS-1	CS-2	CS-2A	SSG-1	RAP
1 in.					0
3/4 in.	0				0
1/2 in.	0			0	1
3/8 in.	5	0	0	1	6
No. 4	70	10	2	12	24
No. 8	97	31	60	39	45
No. 16	98	50	87	64	64
No. 30	98	62	93	82	82
No. 50	98	71	94	96	94
No. 100	98	75	95	99	98
No. 200	98.5	78.3	95.5	99.0	99.0

Table. 3-6 Aggregate gradation of SR 12.5A mixture

Sieve Size	Aggregate designation				
	CS-1	CS-2	SSG-1	CG-5	RAP
1 in.	0	0	0	0	0
3/4 in.	0	0	0	0	0
1/2 in.	44	0	1	0	14
3/8 in.	86	3	3	0	23
No. 4	99	32	10	6	39
No. 8	99	52	23	23	60
No. 16	99	64	43	49	76
No. 30	99	71	66	71	88
No. 50	99	76	90	86	97
No. 100	99	80	97	92	99
No. 200	98.8	81.8	97.5	94.3	99.68

Table 3-7. Aggregate gradation of SR 19A mixture

Sieve Size	Aggregate designation				
	CG-1	CG-2	CG-2A	SSG-1	RAP
1.5 in.	0	0	0	0	0
1 in.	0	0	0	0	0
3/4 in.	1	0	0	0	3
1/2 in.	58	0	0	5	12
3/8 in.	86	0	0	12	19
No. 4	99	18	11	29	45
No. 8	99	40	40	59	72
No. 16	99	55	59	83	88
No. 30	99	65	71	92	95
No. 50	99	74	82	97	98
No. 100	99	82	91	99	99
No. 200	99.5	88.4	95.5	99.0	99.7

3.4 Specific Gravity

3.4.1Bulk Specific Gravity of Aggregates

The bulk specific gravity of each aggregate of each mix type was determined in the laboratory following Kansas Standard Test KT-6 method. Results are summarized in Table 3-8.

Table 3-8. Summary of Bulk Specific Gravity of Aggregates

SR-9.5A		**SR-12.5A**		**SR-19A**	
Aggregate	**Specific Gravity**	**Aggregate**	**Specific Gravity**	**Aggregate**	**Specific Gravity**
CS-1	2.527	**CS-1**	2.447	**CG-1**	2.576
CS-2	2.511	**CS-2**	2.463	**CG-2**	2.454
CS-2A	2.508	**CG-5**	2.589	**CG-2A**	2.512
SSG-1	2.590	**SSG-1**	2.586	**SSG-1**	2.394
RAP	2.688	**RAP**	2.643	**RAP**	2.625
Bulk Specific Gravity	2.582	**Bulk Specific Gravity**	2.572	**Bulk Specific Gravity**	2.505

3.4.1.1 Bulk Specific Gravity of Fine Aggregate

The bulk specific gravity of fine aggregate was calculated by the equation $G_{sb}= \frac{A}{(B+S-C)}$,

where A= mass of oven dry sample, g;

B= mass of pycnometer filled with water to the calibration mark, g;

S= mass of saturated surface dry sample in, g; and

C= mass of pycnometer, specimen and water to the calibration mark, g.

Note: The cone test was used for determining the saturated-surface dry condition (AASHTO T 84) instead of using the drying pan with rusted bottom as mentioned in the KT-6 procedure.

Figure 3-4. Making of saturated-surface dry condition for fine aggregate using the cone test.

Figure 3-5. Rotation of flask in inclined position to expel all the air bubbles.

3.4.1.2 Bulk Specific Gravity of Coarse Aggregate

The bulk specific gravity of coarse aggregate was calculated by the equation $G_{sb}= \frac{A}{(B-C)}$,

where A= mass of oven-dry sample in air, g;

B= mass of saturated surface-dry sample in air, g; and

C= mass of saturated sample in water, g.

Figure 3-6. Making of saturated-surface dry condition using a dampened, absorbent towel for the coarse aggregate sample.

3.4.1.3 Bulk Specific Gravity of Reclaimed Asphalt Pavement (RAP) Aggregate

Bulk specific gravity of RAP aggregate is an important property and used for calculating voids in mineral aggregate (VMA). If the source of the RAP and original construction data are available, then the bulk specific gravity of RAP aggregate is taken as the bulk specific gravity of virgin aggregate. If the data is not available, then the bulk specific gravity is calculated in three simple steps described below.

1. The theoretical maximum specific gravity of RAP mixture G_{mm} is found following the AASHTO T 209 procedure.
2. Then, the effective specific gravity of the RAP is calculated.

$$G_{se}=\frac{100-P_b}{\left(\frac{100}{G_{mm}}-\frac{P_b}{G_b}\right)}$$

where P_b= percent binder in the RAP mixture;

G_{mm}=theoretical maximum specific gravity of RAP mixture; and

G_b=assumed specific gravity of binder (McDaniel and Anderson assumed a value of 1.020).

3. The bulk specific gravity of RAP aggregate is then estimated using the assumed value of asphalt absorption P_{ba}, and G_{se}.

$$G_{sb}=\frac{G_{se}}{\frac{(P_{ba}-G_{se})}{100G_b}+1}$$

P_{ba} is obtained from historical data. If the historical data are not available, typical water-absorption value of aggregate (60-65%) may be taken as P_{ba}.

3.4.2 Specific Gravity of Binder

The specific gravity of the binder was obtained from the KDOT mix design information**.**

Table 3-9. Specific gravity of binders

Binder	**Specific Gravity**		
	SR-9.5A	**SR-12.5A**	**SR-19A**
PG 58-28	1.0270	1.0430	1.0224

3.5 Preparation of Samples for Hamburg Wheel Tracking Device Test and KT-56 Test

The samples were prepared following the Kansas Test Method KT-58 Procedure: Method for preparing and determining the density of hot-mix asphalt (HMA) specimens by means of the Superpave gyratory compactor. The main steps involved in preparing HWTD specimens include drying aggregates to constant weight, batching of aggregates, heating of aggregates and binder to mixing temperature, mixing of binder and aggregates, and conditioning (short-term aging) and compacting the specimen to appropriate percent air voids using the Superpave gyratory compactor. The detailed steps involved in the preparation of specimens are described below and shown in Figure 3-7.

1. All required aggregates are weighed in steel pans separately and are combined to form a desired batch weight. Typically a batch weight of 13,800 to 14,000 grams of aggregate

produce five HWTD specimens (150±2mm in diameter and 62±2 mm in height), 1,500 grams of G_{mm} sample, and 5% wastage, considering the combined aggregate bulk-specific gravity between 2.55-2.70.

2. The batched aggregates and binder are heated in the oven to an appropriate mixing temperature. Since the study contained mixtures with RAP material, the RAP material is heated separately (about 140ºF) i.e., much lower than the mixing temperature to prevent additional hardening of the RAP asphalt cement. The virgin aggregates are heated above the mixing temperature to compensate lower mixing temperature of RAP, so that the temperature of the total mix is within the actual range of the mixing temperature.
3. After the aggregates and binder reach the mixing temperature, the heated aggregates are introduced to a mechanical mixer and a crater is formed. The required amount of binder and additive is added and mixing is continued until every particle is uniformly coated with binder. Since the mixture contains RAP material, the amount of binder to be added is adjusted because the RAP material also contains some binder. The weight of new binder to be added is calculated as follows:

$$\frac{\text{Percent binder(total)} \times \text{Total weight}}{100} - (\text{weight of binder in RAP})$$

where, weight of binder in RAP= (percent binder in RAP) × (weight of RAP)

4. After mixing, the mixture is placed in a pan, spread evenly, and transferred to an oven at compaction temperature for about 2 hours ± 5 minutes for short-term aging. The mixture is stirred after 60±5 minutes to maintain uniform aging.
5. The mixture is now ready to be compacted using the Superpave gyratory compactor (SGC).

(a) Heating aggregate to mixing temperature in oven

(b) Adding asphalt to the aggregate in the mixer

(c) Mixing of asphalt and aggregate in the mixer

(d) Mixture kept at compaction temperature for 2 hrs (short term aging)

Figure 3-7. HMA mixing process.

3.5.1 Compaction Using Superpave Gyratory Compactor (SGC) *(Kansas Test Method KT-58)*

The molds, plates of SGC, and pouring pan are preheated to compaction temperature for about 45-60 minutes before the start of compaction. The SGC is switched on and all required settings such as height of specimen, number of gyrations, angle of gyration, pressure, etc. are configured.

The compaction parameters for all mixture types (SR-9.5A, SR-12.5A and SR-19A) are listed in Table 3-10.

Table 3-10. Compacting parameters for Superpave gyratory compactor

Parameter	HWTD	KT-56
Specimen height	62	95
Pressure	600±18 kPa	600±18 kPa
Angle of gyration	1.16° ± 0.02°	1.16° ± 0.02°
Number of gyrations	$N_{initial}$=7,N_{design}=75,N_{max}=115	$N_{initial}$=7,N_{design}=75,N_{max}=115
Speed of rotation	30±0.5 gyrations per minute	30±0.5 gyrations per minute

The mold and base plate are removed from the oven and the mold is charged with the required amount of mixture using a pouring pan. The mixture is leveled with a spatula and the top plate is placed in the mold. To avoid the mixture sticking to the plates, paper disks are placed in between the plates and mixture. The mold is now transferred into the SGC. The mixtures are compacted with applicable parameters listed in Table 3-10. The SGC will stop automatically when it reaches the specified number of gyrations. The mold is then removed from the SGC and the sample is extruded from the mold and cooled for 5 minutes in front of a fan.

Table 3-11. Superpave gyratory compaction effort (Kansas Test Method KT-58)

Design ESALs (Millions)	Travelway		
	$N_{initial}$	N_{design}	N_{max}
<0.3	6	50	75
0.3 to < 3	7	75	115
3 to <30	8	100	160
≥30	9	125	205
	Shoulder		
A*	6	50	75
B*	**	**	**

* At the contractor's option, A or B may be used.

** Use travelway design traffic properties for B.

(a) Pine Superpave Gyratory Compactor

(b) Pouring HMA into SGC mold

(c) Extruding the specimen from the mold

Figure 3-8. Compacting specimen using Superpave gyratory compactor

3.5.2 Determining the Weight of Mixture Required to Produce a Specimen with Desired Percent Air Voids

The weight of mixture needed to produce a specimen with specified air voids (7±1 % air voids for HWTD and 7±0.5 % air voids for KT-56) is determined theoretically by the following equation:

$$\text{Weight of specimen 'W'} = \%G_{mm} @N_f \times G_{mm} \times \text{volume of sample}$$

where, $\%G_{mm}$ @ N_f = 0.93 (for HWTD and KT-56 test specimens);

G_{mm}= theoretical maximum specific gravity of loose mixture; and

Volume=$\frac{\Pi d^2 h}{4}$; d=150 mm, h=62 mm for HWTD specimen and 95 mm for KT-56 specimen, respectively.

After obtaining the theoretical weight of the specimen, three trial specimens are prepared with the theoretical weight of specimen W, W+10 grams, and W-10 grams, to find out the exact weight of mixture needed to produce a compacted specimen with air voids in the desired range.

3.5.3 Determining the Bulk-Specific Gravity of Compacted Specimen (G_{mb}) and Uncompacted HMA Mixture (G_{mm}) *(Kansas Test Method KT-15)& (Kansas Test Method KT-39)*

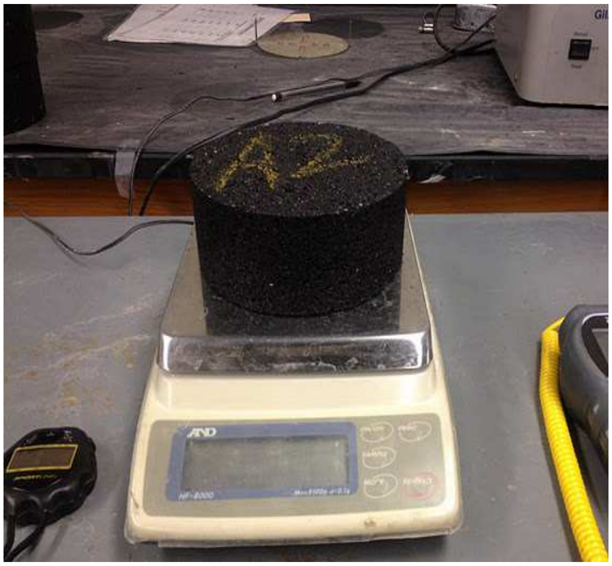
The bulk-specific gravity (G_{mb}) of compacted specimens is determined following Kansas Test Method KT-15 as shown in Figure 3-9: bulk-specific gravity and unit weight of compacted asphalt mixtures. (Procedure III). The steps are as follows:

1 The specimen is dried to a constant mass. The specimen is weighed at room temperature (77^o ± 2F or 25 ± 1^O C) to the nearest 0.1 grams and recorded as A.

2 The specimen is immersed in the water bath at 77^o ± 2F or 25 ± 1^O C and saturated for 4±1 minutes. The submerged mass is recorded as C.

3 The submerged specimen is brought to saturated-surface dry (SSD) condition using terry cloth. The SSD specimen is weighed and recorded as B.

$$\text{Bulk specific gravity, } G_{mb} = \frac{A}{(B-C)}$$



Dry mass in air

Mass in water

Making of SSD

SSD mass in air

Figure 3-9. Process of determining the bulk-specific gravity of the compacted specimen.

The theoretical maximum specific gravity of the asphalt paving mixture (G_{mm}) is determined using Kansas Test Method KT-39 as shown in Figure 3-10. The steps are:

1. The laboratory-mixed sample is taken from the oven after short-term aging and cooled to room temperature. During this cooling process, the particles are separated so that no particle is larger than 6.3 mm (1/4 inch).
2. A sample of known mass is loaded into a calibrated conical flask and the mass of the flask with the sample is recorded as B.

3 The flask is filled with water till the sample is fully submerged.

4 Using a vacuum pump, a partial pressure of 27±3 mm of Hg is applied for 15 minutes to remove the air entrapped in the sample.

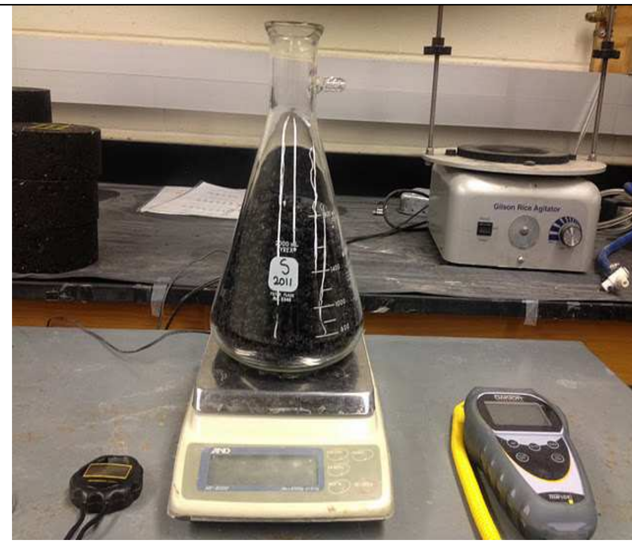
5 The conical flask is submerged in the water for 10±1 minutes and the weight is recorded as C. The temperature of water should be $77^{o} \pm 2$F or 25 ± 1^{O} C.

6 The mass of conical flask in air is recorded as A and the mass of conical flask in water after 10 minutes immersion is recorded as D.

7 Theoretical maximum specific gravity of the uncompacted HMA mixture is given by

$$G_{mm} = \frac{(B-A)}{(B+D)-(A+C)}$$

Making of loose sample



Mass of sample + flask in air

Expelling air using vacuum apparatus

Mass of sample + flask in air

Figure 3-10. Determining the theoretical maximum specific gravity (G_{mm}) of loose HMA mixture.

3.6 Performance Testing Procedures

3.6.1 Hamburg Wheel Tracking Device Procedure *(TEX-242-F, June 2009)*

The HWTD used in this study was manufactured by Precision Machine & Welding Company, Salina, Kansas. The TEX-242-F procedure was followed for the HWTD test. The laboratory-molded specimens were placed in a cutting template under the masonry saw to cut across the specimen, as shown in Figure 3-11, to fit into polyethylene molds. The specimens were then placed into the polyethylene mold and mounted into the tray. If there was a gap in between the specimens, it was necessary to fill the gap with Plaster of Paris and allow it to set for one hour before starting the procedure. The mounting trays were placed in an empty water bath. The software was started and required information entered. Test specifications were as follows:

a) Testing temperature: 122±1.8^{o}F (50±1^{o}C).
b) Load: 158 lb. ± 5 lb. (705±22 N).
c) Number of passes per minute: 50±2.
d) Maximum number of passes setting: nonrestrictive for SR-9.5A, SR-12.5A and 20,000 for SR-19A.
e) Maximum speed of wheel: 1.1 ft./sec (approximately)
f) Maximum rut depth: 20 mm
g) Rut-depth measurements: every 100 passes.

Once water reached the designated temperature, the specimen was saturated for an additional 30 minutes. After the saturation, the arms were lowered so that they would rest on the specimen and the test was begun. The testing device automatically stopped when it reached either operator-specified maximum rut depth or the number of wheel passes, whichever came first. The linear variable differential transducers (LVDTs) connected to the machine on either sides measured the vertical deformation (rut depth) at 11 different points along the wheel path of the specimen. The rut depth was recorded to the file using a computer-based automated data acquisition system connected to the HWTD device. Post compaction, creep slope, stripping inflection point, and stripping slope were obtained from the plot of the number of wheel passes versus rut depth.

(a) Sample being cut along edge of the mold using masonry saw

(b) Vertical-cut samples (approx. 5/8 inch)

(c) Samples placed in molds and mounted in tray, ready for testing

(d) Failed sample(rut depth>20mm)

Figure 3-11. Process of testing samples in Hamburg Wheel Tracking Device.

Experimental Design Matrix (HWTD)

The HWTD test was conducted on all three mixtures. The only variable that changed was the mixture asphalt content. The test was conducted on specimens prepared at four different asphalt contents, starting from the design asphalt content and decrement of two tenths (0.2%) of a percent each time. For statistical analysis purposes, three sets of HWTD specimens were prepared for each asphalt content. The design matrix is presented in Table 3-12.

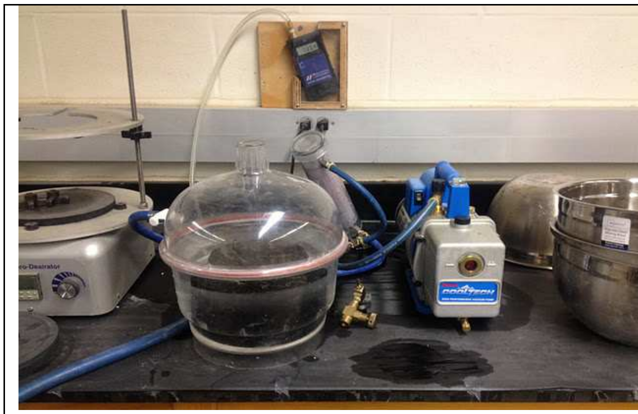
Table 3-12. Experimental design matrix for Hamburg Wheel Tracking test

SR-9.5A				SR-12.5A				SR-19A*			
Design asphalt content=5.54% Additive: Arr Maz HP^+ (0.6%)				design asphalt content=5.3% Additive: Arr $MazHP^+$ (0.45%)				design asphalt content=4.4% Additive: Arr Maz HP^+ (0.6%)			
5.54%	5.34%	5.14%	4.94%	5.3%	5.1%	4.9%	4.7%	5.75%	5.55%	5.35%	5.15%
3 sets for each asphalt content (3×4=12 plugs)				3 sets for each asphalt content (3×4=12 plugs)				3 sets for each asphalt content (3×4=12 plugs)			
Air voids=7±1%				Air voids=7±1%				Air voids=7±1%			

* The asphalt content was more than the designed asphalt content; this is because the asphalt in the RAP material was not considered.

3.6.2 Resistance of Compacted Asphalt Mixture to Moisture-Induced Damage (Kansas Test Method KT-56)

The procedure of KT-56 has been discussed in Chapter 2 and is shown in Figure 3-12. Using SGC, a minimum of six compacted specimens were produced at approximately 7±0.5 percent air voids for each of the cells in the design matrix shown in Table 3-13. Some of the compacted specimens were found to be out of the prescribed air-void range and were discarded.

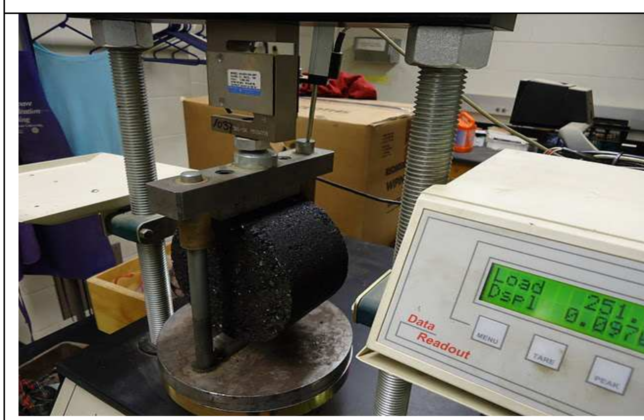


(a) Saturating the specimen using vacuum apparatus

(b) Specimen wrapped with plastic film enclosed in heavy-duty, leak-proof bag with 10 ml of water

(c) Specimen freezing @ -18^{o}C for at least 16 hours

(d) Specimen in water bath at 60o C for 24 hours



(e) Indirect tensile strength determination

(f) Inspection of stripping on interior surface

Figure 3-12. Steps involved in determination of tensile strength of conditioned samples (KT-6).

Experimental Design Matrix

The design matrix shown in Table 3-13 is similar to the HWTD design matrix. The only parameter changed was asphalt content.

Table 3-13. Experimental design matrix for Kansas Standard Test KT-56

SR-9.5A				SR-12.5A				SR-19A*			
Design asphalt content=5.54% Additive: Arr Maz HP+ (0.6%)				Design asphalt content=5.3% Additive: Arr MazHP+ (0.45%)				Design asphalt content=4.4% Additive: Arr Maz HP+ (0.6%)			
5.54%	5.34%	5.14%	4.94%	5.3%	5.1%	4.9%	4.7%	5.75%	5.55%	5.35%	5.15%
A total of six plugs (3 conditioned, 3 unconditioned) for each asphalt content				A total of six plugs (3 conditioned, 3 unconditioned) for each asphalt content				A total of six plugs (3 conditioned, 3 unconditioned) for each asphalt content			
Air voids=7±0.5%				Air voids=7±0.5%				Air voids=7±0.5%			

* The asphalt content was more than the designed asphalt content; the asphalt in the RAP material was not considered.

3.7 Calculation of VMA and Film Thickness (Brown et al., 2009)

Voids in the mineral aggregate (VMA) are given by the following equation:

$$\text{VMA}=100\left(\frac{G_{mb}(1-P_b}{G_{sb}}\right)$$

where

G_{mb} = bulk-specific gravity of compacted mixture;

G_{sb} = bulk-specific gravity of aggregate; and

P_b = percent of asphalt.

Asphalt film thickness is calculated based on the surface area factors mentioned in Chapter 2. The following formula gives the asphalt film thickness:

$$T_F=1000\left(\frac{V_{asp}}{SA\times W}\right)$$

where

T_F= asphalt film thickness, microns;

V_{asp}= effective volume of asphalt cement, liters;

SA= surface area of the aggregate, m^2/kg; and

W= weight of aggregate, kg.

Calculated surface areas for SR-9.5A, SR-12.5A, and SR-19A are 3.937 m^2/kg, 4.686 m^2/kg and 3.717 m^2/kg, respectively.

Chapter 4 - Results and Discussion

4.1 Hamburg Wheel Tracking Device Test Results

4.1.1 Number of Wheel Passes for SR-9.5A and SR-12.5A Mixtures

All specimens were compacted to 7±1 air voids and tested in wet condition. In general, the HWTD specimens were subjected to 20,000 wheel passes or rut depth of 12.5 mm (TxDOT), 20 mm (CDOT), whichever came first. In this study, for the SR-19A mixture, the specimens were subjected to 20,000 wheel passes or 20-mm maximum rut depth. For the SR-12.5A, and SR-9.5A mixtures, specimens were subjected to unlimited wheel passes or 20-mm rut depth, whichever came first.

The only variable in the study was asphalt content (%). For each asphalt content, three replicates were fabricated and tested. Average number of wheel passes and corresponding rut depth are tabulated in Table 4-1.

Table 4-1 shows the average number of wheel passes is lower for the design asphalt content when compared to the number of wheel passes for drier mixes. The highest average number of wheel passes recorded was 11,861 when 4.94% of asphalt (0.6 % below design asphalt content) was used. The lowest number of wheel passes recorded was 5,087 when 5.54 % of asphalt used (design asphalt content). For specimens SB-2,3 and SB-4,5, the average number of wheel passes was too high when compared to the other specimens in the same subset (samples with 5.34 % AC), thus they were discarded and not taken into consideration while calculating the average number of wheel passes.

Table 4-1. Summary of Hamburg Wheel Tracking test results for SR-9.5A mixture

Sample ID	Asphalt content (%)	Virgin asphalt added (%)	No of passes	Rut depth in mm
SA-1,SA-4	5.54 (Design Asphalt Content)	4.11	5,759	20
SA-2,SA-3			4,789	20
SA-6,SA-7			2,250	20
SA-9,SA-10			3,700	20
SA-11,SA-12			7,433	20
SA-13,SA-15			6,589	20
Average	5.54	4.11	5,087	20
SB-2,SB-3	5.34	3.91	20,855*	20
SB-4,SB-5			24,187*	20
SB-6,SB-7			8,367	20
SB-8,SB-11			9,091	20
SB-12,SB-13			9,450	20
SB-14,SB-15			11,621	20
Average	5.34	3.91	9,632	20
SC-3,SC-4	5.14	3.71	8,867	20
SC-5,SC-6			11,689	20
SC-7,SC-8			12,679	20
SC-9,SC-10			13,033	20
SC-11,SC-14			9,217	20
SC-13,SC-15			9,649	20
Average	5.14	3.71	10,856	20
SD-2,SD-4	4.94	3.51	11,547	20
SD-3,SD-5			11,049	20
SD-6,SD-14			9,550	20
SD-7,SD-12			10,903	20
SD-8,SD-15			12,091	20
SD-9,SD-10			16,023	20
Average	4.94	3.51	11,861	20

Table 4-2. Summary of Hamburg Wheel Tracking test for SR-12.5A mixture

Sample ID	Asphalt content (%)	% Virgin Asphalt added	No of passes	Rut depth in mm
RA-2,RA-3	**5.3** (Design asphalt content)	4.14	15,723*	20
RA-4,RA-5			26,211*	20
RA-7,RA-6			4,113	20
RA-9,RA-10			4,583	20
RA-12,RA-13			5,291	20
RA-14,RA-15			7,533	20
Average	5.3	4.14	5,380	20
RB-1,RB-3	5.1 .	3.94	7,127	20
RB-4,RB-5			11,347	20
RB-6,RB-9			14,653	20
RB-8,RB-10			12,621	20
RB-11,RB-13			11,967	20
RB-14,RB-15			25,563*	20
Average	5.1	3.94	11,543	20
RC-1,RC-4	4.9	3.74	8,373*	20
RC-2,RC-3			15,401	20
RC-6,RC-10			29,541*	20
RC-8,RC-9			26,893*	20
RC-11,RC-13			16,637	20
RC-12,RC-14			18,519	20
Average	4.9	3.74	16,852	20
RD-1,RD-4	4.7	3.54	19,125*	20
RD-2,RD-3			18,355*	20
RD-6,RD-9			42,335	20
RD-7,RD-8			38,153	20
RD-11,RD-13			25,650*	13.3*
RD-14,RD-15			25,650*	6.1*
Average	4.7	3.54	40,244	20

From Table 4-2, we can see that performance was better at lower asphalt content i.e., below the design asphalt content. As the asphalt content decreased, the average number of wheel passes increased. The average number of wheel passes increased from 5,380 to 40,244 when the asphalt content was decreased from 5.3% to 4.7%.

When testing specimens RD-11, 13 and RD-14 and 15, the test stopped due to technical error in the HWTD machine. Thus the final number of wheel passes could not be determined but the test still yielded valuable information in the form of creep slope, stripping slope, and stripping inflection points, if any.

Table 4-3. Summary of Hamburg Wheel Tracking test for SR-19A mixture

Sample ID	Asphalt content (%)	% Virgin asphalt added	No of passes	Rut depth in mm
3C-3,3C-4	5.75	4.4	15,683	20
3C-2,3C-10	5.75	4.4	11,447	20
3C-6,3C-9	5.75	4.4	16,250	20
3C-7,3C-8	5.75	4.4	21,975	20
3C-11,3C-12	5.75	4.4	21,050	20
3C-13,3C-14	5.75	4.4	35,241	20
Average	5.75	4.4	20274	20.0
3D-1,3D-3	5.55	4.2	20,000	12.8
3D-2,3D-4	5.55	4.2	20,000	13.1
3D-5,3D-7	5.55	4.2	13,565	20
3D-6,3D-8	5.55	4.2	11,959	20
3D-9,3D-10	5.55	4.2	4,950	20
3D-11,3D-12	5.55	4.2	8,200	20
Average	5.55	4.2	13112	17.7
3E-1,3E-4	5.35	4	20,000	14.9
3E-2,3E-3	5.35	4	20,000	11.1
3E-5,3E-8	5.35	4	8,938	20
3E-6,3E-7	5.35	4	8,038	20
3E-10,3E-14	5.35	4	20,000	14.4
3E-11,3E-13	5.35	4	20,000	17
Average	5.35	4	16163	16.2
3F-1,3F-4	5.15	3.8	20,000	11.6
3F-2,3F-3	5.15	3.8	20,000	17
3F-5,3F-7	5.15	3.8	20,000	16.5
3F-8,3F-9	5.15	3.8	20,000	14
3F-10,3F-13	5.15	3.8	20,000	14.8
3F-12,3F-14	5.15	3.8	20,000	11.7
Average	5.15	3.8	20000	14.3

The samples were prepared at a much higher asphalt content than the design asphalt content. The design asphalt content was 4.4%, but the specimens were prepared with asphalt content starting at 5.75 % and up to 5.15 % with a 0.2% decrement. Initially, the quantity of binder available in the RAP aggregates was not considered. Thus the mixes had more asphalt content than the prescribed content. However, we can make comparisions based on the results obtained.

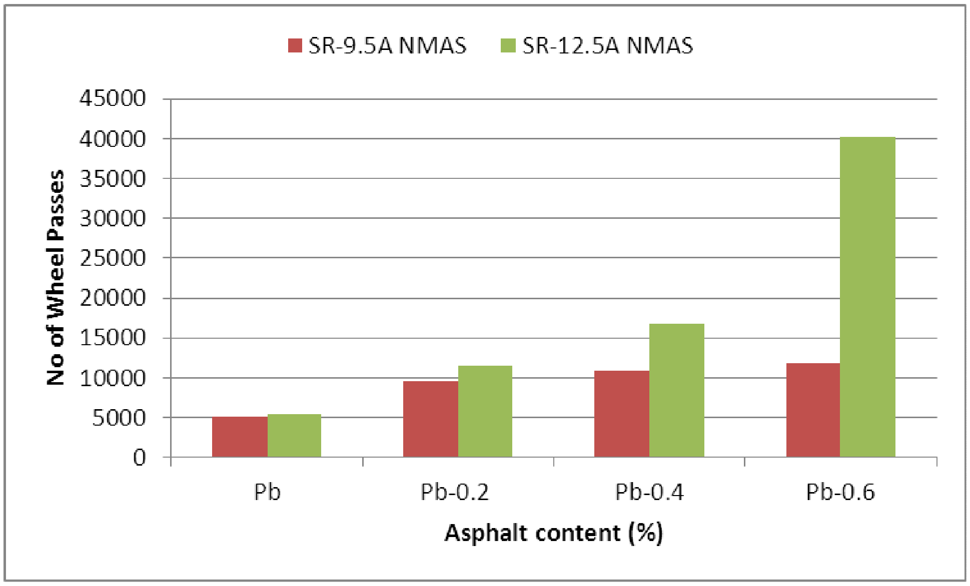


Figure 4-1. Effect on average number of wheel passes due to asphalt content variation.

From the Figure 4-1, we can clearly see that for both mixtures the lowest number of wheel passes was recorded when design asphalt content (Pb) was used, and the highest number of wheel passes was recorded when the lowest binder content was used in the mixture (Pb-0.6%), where Pb is the design asphalt content. There was a large variation in the average number of wheel passes for the SR-12.5A mixture when compared to the SR-9.5A mixture.

Scatter plots

From the Table 4-1, we can see the number of wheel passes recorded for samples prepared with 5.34% asphalt content varies from 24,187 to 8,367. Although all these samples were prepared following the same procedure and under the same conditions, this large variation was observed. The passes for replicates SB-2, 3, 4, 5 are much higher than the other samples SB-6, 7,8,11,12,13,14 and 15. From Figure 4-2 it is obvious that replicates 1 and 2 with asphalt content of 5.34% (SB-2, 3, 4, 5) stand apart from other samples in that subset. If we consider these passes, there will be a considerable effect on the average number of wheel passes. Therefore, it is important to study these values by performing influence diagnostic tests available in the area of statistics.

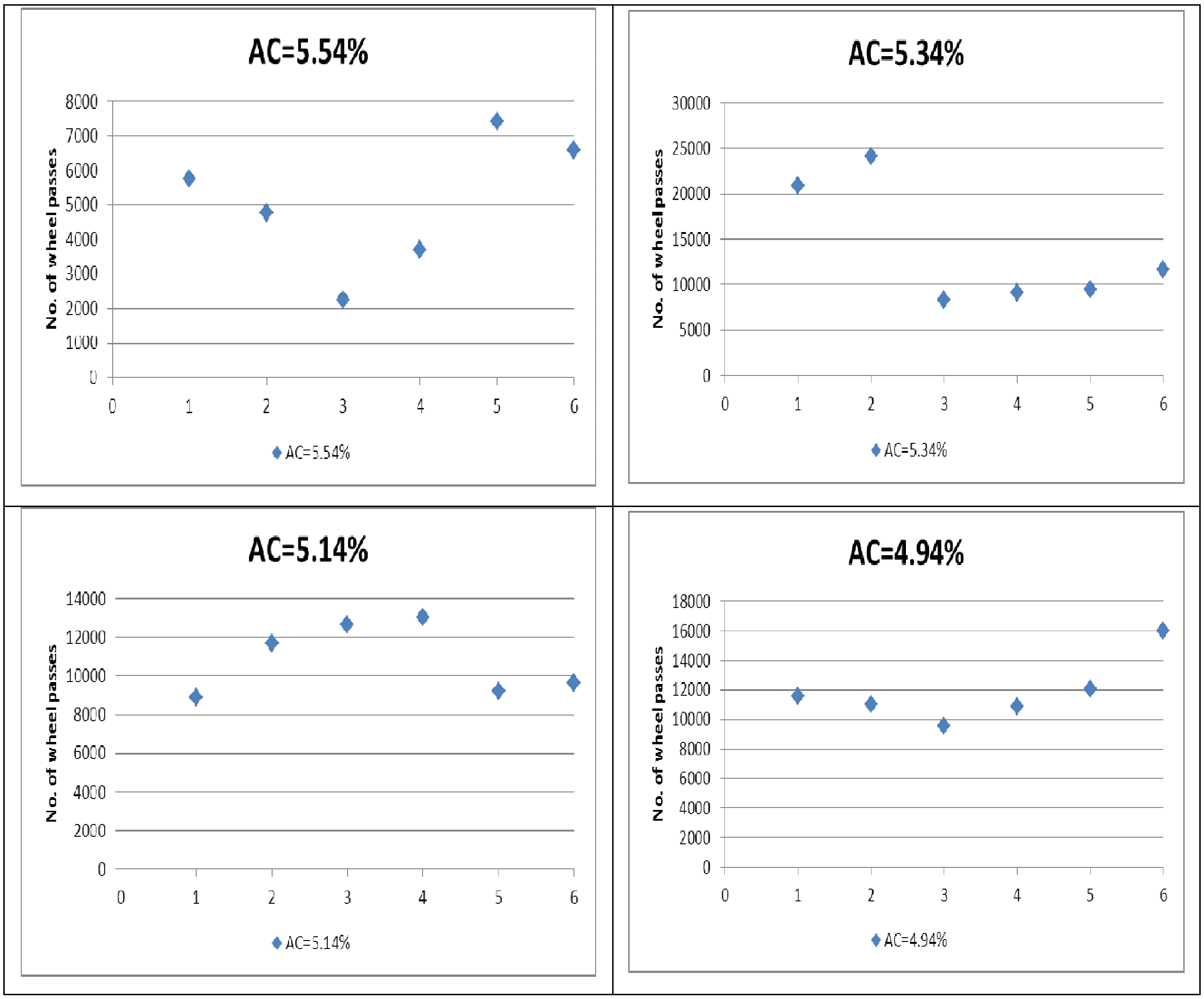


Figure 4-2 Scatter plot of the No. of wheel passes for different replicates of SR-9.5A mixture

Similar variations were also observed for the SR-12.5A mixture as can be seen in the scatter plots illustrated in Figure 4.3.

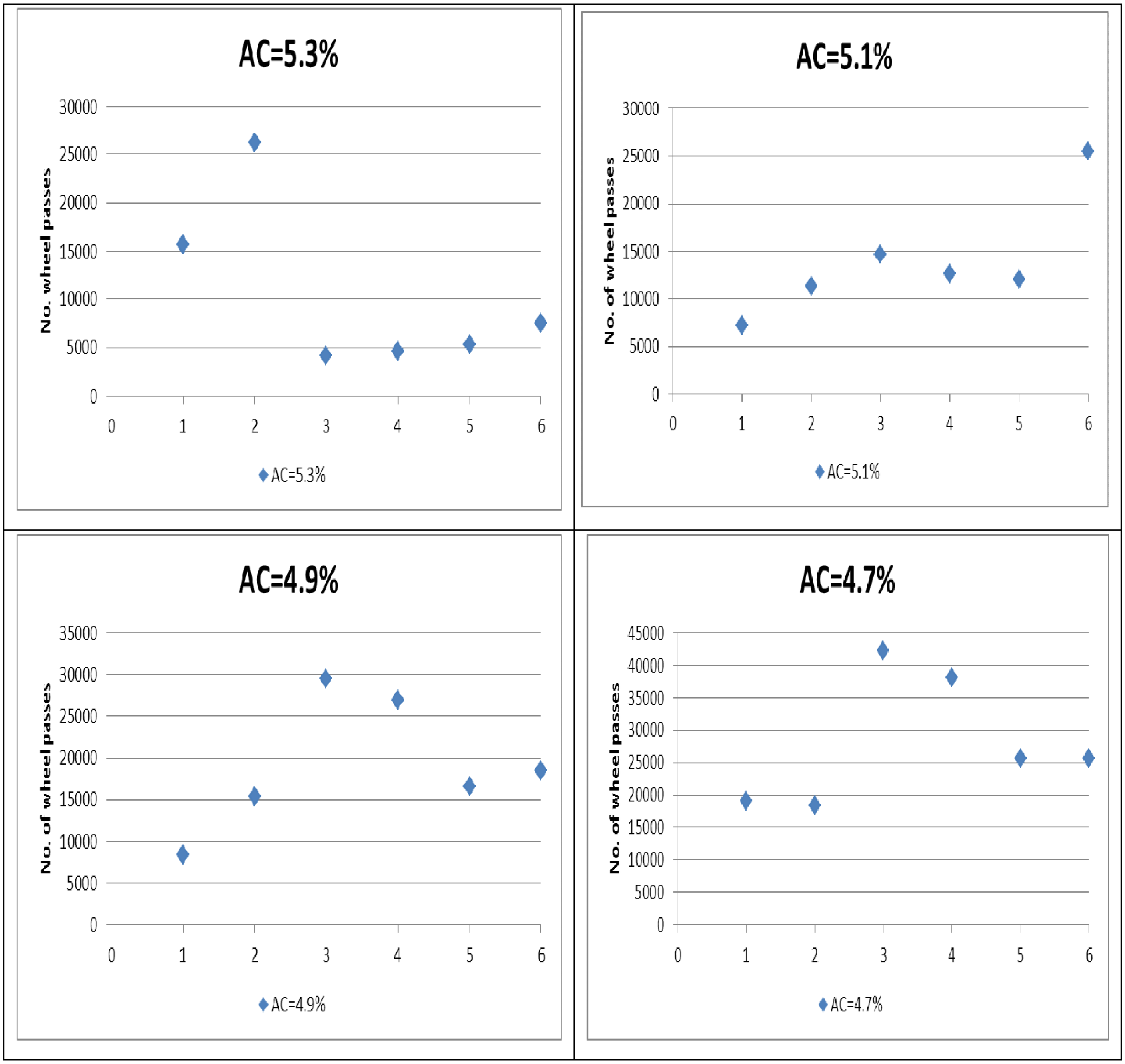


Figure 4-3 Scatter plot of the No. of wheel passes for different replicates of SR-12.5A mixture

4.1.2 Identification of outliers/influence observations

According to Hawkin (1980) *"an outlier is an observation that deviates so much from other observations as to arouse suspicion that it was generated by a different mechanism."* In this study, influence statistics such as Cook's Distance, Dffits and Rstudent were performed (SAS 9.3 User Guide, 2011).

Cook's Distance or cook's d measures the change in the parameter estimates caused by deleting each observation. A general cut-off value is 1 and a size adjusted cut-off value is 4/n, where n is the number of observations. In this study, the cook's d criteria is 0.166 (n=24) for the SR-9.5A mixture and 0.181 (n=22) for the SR-12.5A mixture.

Dffits measures the change in the predicted value for the i^{th} observation and is calculated by deleting the i^{th} observation. A general cut-off value of 2 and a size-adjusted cut-off value of 2* $(P/N)^{0.5}$, where n is number of observations and p is number of parameters, are used. In this study the dffits criteria is 0.408 (n=24) for the SR-9.5A mixture and 0.426 (n=22) for the SR-12.5A mixture.

Rstudent is the raw residual value divided by the standard error. The error variance is calculated by not considering the deleted i^{th} observation. Observations with rstudent values greater than 2 need some attention.

Tables 4-4 and 4-5 list the outputs of the influence diagnostic tests for SR-9.5A and SR-12.5A, respectively. In SR-9.5A mixture, according to cook's d criteria (>0.166), observations 7 and 8 are the outliers; according to dffits criteria (0.408) observations 7, 8, 9, 10, 11 and 24 are the outliers; and according to rstudent criteria (>2), observations 7 and 8 need some attention. As a result, observations 7 and 8 were discarded, since these were identified as outliers by all three influence diagnostic tests used in statistics.

Table 4-4 Outputs of influence diagnostic tests for SR-9.5A mixture

Observations	Asphalt content	No. of wheel passes	Cook's d	Dffits	Rstudent	Leverage H
1	5.54	5759	0.002	0.084	0.188	0.167
2	5.54	4789	0.000	-0.037	-0.083	0.167
3	5.54	2250	0.033	-0.360	-0.805	0.167
4	5.54	3700	0.008	-0.174	-0.389	0.167
5	5.54	7433	0.023	0.296	0.663	0.167
6	5.54	6589	0.009	0.188	0.421	0.167
7	5.34	20855	0.197	0.965	2.158	0.167
8	5.34	24187	0.432	1.699	3.800	0.167
9	5.34	8367	0.127	-0.743	-1.662	0.167
10	5.34	9091	0.096	-0.635	-1.421	0.167
11	5.34	9450	0.082	-0.584	-1.305	0.167
12	5.34	11621	0.022	-0.291	-0.651	0.167
13	5.14	8867	0.016	-0.250	-0.560	0.167
14	5.14	11689	0.003	0.104	0.233	0.167
15	5.14	12679	0.014	0.229	0.513	0.167
16	5.14	13033	0.019	0.275	0.614	0.167
17	5.14	9217	0.011	-0.206	-0.460	0.167
18	5.14	9649	0.006	-0.151	-0.338	0.167
19	4.94	11547	0.000	-0.039	-0.088	0.167
20	4.94	11049	0.003	-0.101	-0.227	0.167
21	4.94	9550	0.022	-0.292	-0.652	0.167
22	4.94	10903	0.004	-0.120	-0.268	0.167
23	4.94	12091	0.000	0.029	0.064	0.167
24	4.94	16023	0.071	0.539	1.206	0.167

Table 4-5 Outputs of influence diagnostic tests for SR-12.5A mixture

Observations	Asphalt content	No. of wheel passes	Cook's d	Dffits	Rstudent	Leverage H
1	1	15723	0.021	0.286	0.639	0.167
2	1	26211	0.195	0.969	2.166	0.167
3	1	4113	0.033	-0.361	-0.808	0.167
4	1	4583	0.029	-0.334	-0.747	0.167
5	1	5291	0.022	-0.293	-0.656	0.167
6	1	7533	0.007	-0.168	-0.375	0.167
7	2	7127	0.036	-0.378	-0.845	0.167
8	2	11347	0.005	-0.139	-0.311	0.167
9	2	14653	0.000	0.042	0.095	0.167
10	2	12621	0.001	-0.069	-0.154	0.167
11	2	11967	0.003	-0.105	-0.235	0.167
12	2	25563	0.109	0.683	1.528	0.167
13	3	8373	0.094	-0.629	-1.406	0.167
14	3	15401	0.012	-0.211	-0.472	0.167
15	3	29541	0.085	0.594	1.329	0.167
16	3	26893	0.047	0.432	0.966	0.167
17	3	16637	0.005	-0.142	-0.319	0.167
18	3	18519	0.000	-0.039	-0.087	0.167
19	4	19125	0.158	-0.818	-1.417	0.250
20	4	18355	0.183	-0.887	-1.536	0.250
21	4	42335	0.243	1.047	1.814	0.250
22	4	38153	0.111	0.672	1.163	0.250

For SR-12.5A mixture, according to cook's d criteria (>0.181), observations 2, 20 and 21 are the outliers; according to dffits criteria (0.426) observations 2, 12, 13, 15, 16, 19, 20, 21 and 22 are the outliers; and according to rstudent criteria (>2), observation 2 needs some attention. Ultimately observation 2 was discarded since it was identified by all three tests as an outlier.

4.1.3 Test of Significance

The next step after identifying the outliers was fitting a regression line between the asphalt content and the number of wheel passes, so that the trend of the wheel passes with varying asphalt content could be found. In this study, the number of wheel passes was taken as the response variable, Y, and asphalt content was taken as the predictor variable, X. Tables 4-6 and 4-7 show the results of the Analysis of Variance (ANOVA) and simple linear regression for SR-9.5A and SR-12.5, respectively.

Table 4-6 Regression results for SR-9.5A mixture

Analysis of Variance					
Source	**Degrees of Freedom**	**Sum of Squares**	**Mean Squares**	**F Value**	**Pr > F**
Model	1	143146201	143146201	34.48	<.0001
Error	20	83042684			
Corrected Total	21	226188886			

Root MSE	2037.6	**R-Square**	0.63
Dependent Mean	93333.9	**Adjusted R-Square**	0.61
Coefficient of Variance	21.8		

Parameter Estimates					
Variable	**Degrees of Freedom**	**Parameter Estimates**	**Standard Error**	**t Value**	**Pr > |t|**
Intercept	1	66992	9829.49	6.82	<.0001
Content	1	-11023	1877.28	-5.87	<.0001

Table 4-7 shows the results of the regression for SR-12.5A mixture

Analysis of Variance					
Source	**Degrees of Freedom**	**Sum of Squares**	**Mean Squares**	**F Value**	**Pr > F**
Model	1	1139687113	1139687113	19.84	0.0003
Error	19	1091458564	57445188		
Corrected Total	20	2231145677			

Root MSE	7579.26	**R-Square**	0.51
Dependent Mean	16850	**Adjusted R-Square**	0.48
Coefficient of Variance	44.98		

Parameter Estimates					
Variable	**Degrees of Freedom**	**Parameter Estimates**	**Standard Error**	**t Value**	**Pr > |t|**
Intercept	1	192790	39535	4.88	0.0001
Content	1	-35088	7877.49	-4.45	0.0003

For SR-9.5A, the equation resulting from simple linear regression can be written as:

$Y_i = 66992 - 11023 * X_i + e_i$

where, Y_i= number of wheel passes,

X_i = asphalt content (%), and

e_i = random error.

From the value of the coefficient of determination ($R^2 = 0.63$), the number of wheel passes to be predicted by this model will be reasonable. Also, since the p value is less than 0.05 (95% confidence interval), we conclude that the quantity of asphalt in mixture has a significant effect on the number of wheel passes.

For SR-12.5A mixture, the equation for simple linear regression can be written as:

$$Y_i = 192790 - 35088 * X_i + e_i$$

where, Y_i= number of wheel passes,

X_i= asphalt content (%), and

e_i = random error.

From the value of the coefficient of determination ($R^2 = 0.51$) we can conclude that the number of wheel passes to be predicted by this model may not be very accurate. However, the p value is less than 0.05 (95% confidence interval). Thus we can conclude that the quantity of asphalt in mixture has a significant effect on the number of wheel passes.

4.1.4 Hamburg Wheel Tracking Device Test Output Parameters (Creep slope, Stripping Slope and Stripping Inflection Point)

The performance of the mixtures can be better studied with the HWTD output parameters. Figures 4-4, 4-5, and 4-6 show creep slopes, stripping slopes, and stripping inflection points for SR-9.5A, SR-12.5A, and SR-19A mixtures, respectively.

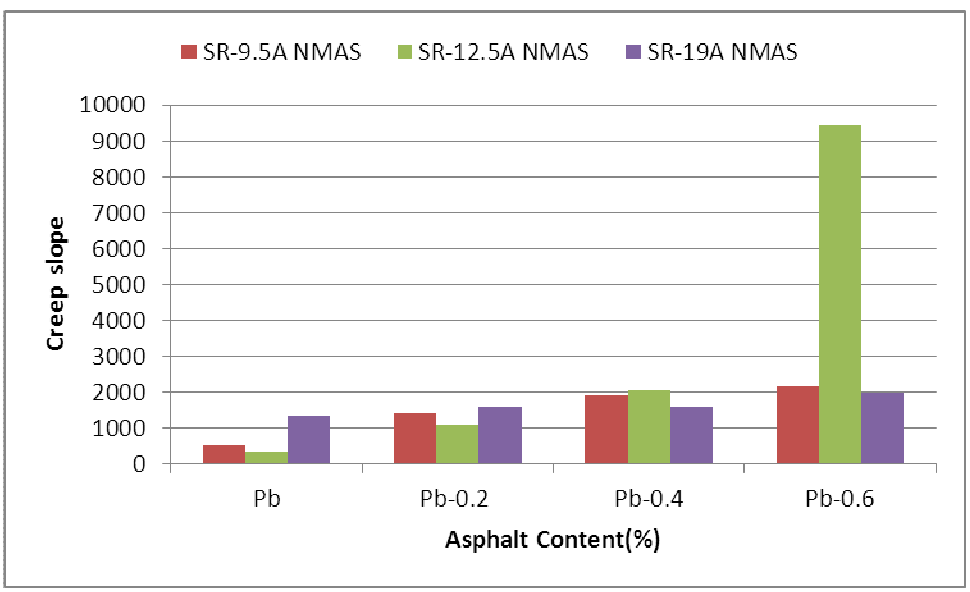


Figure 4-4. Effect of varying asphalt content on creep slope (Passes/mm).

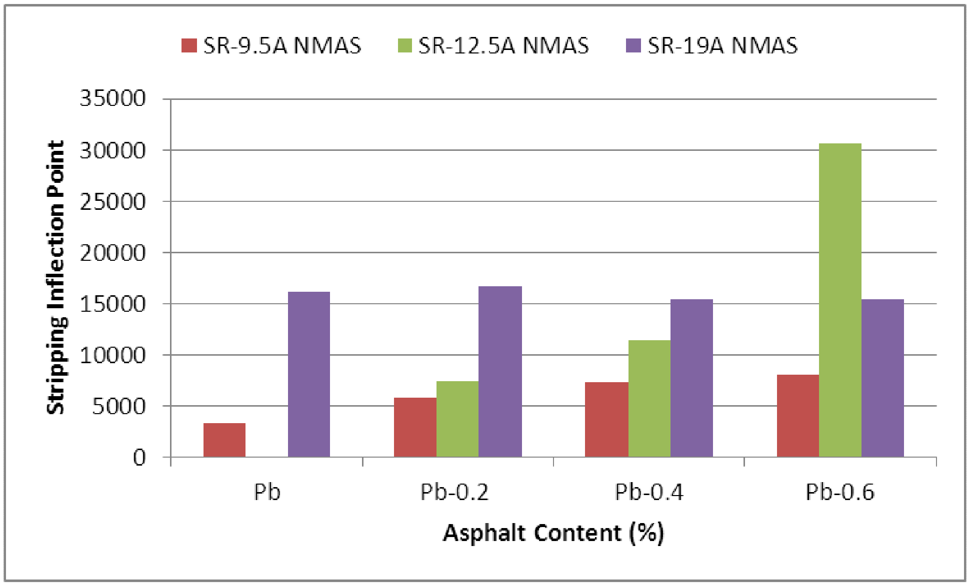


Figure 4-5. Effect of varying asphalt content on stripping inflection points

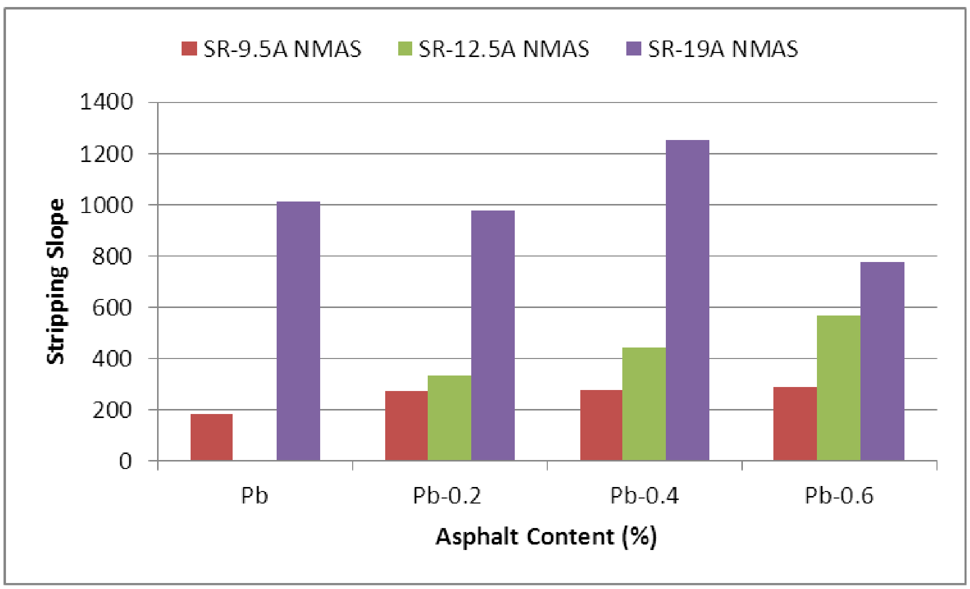


Figure 4-6. Effect of varying asphalt content on stripping slope (passes/mm)

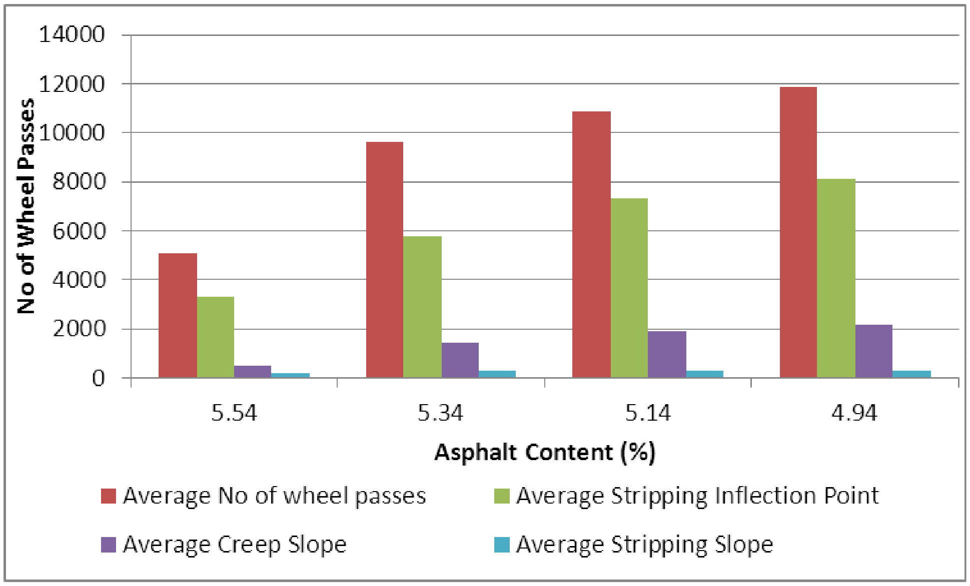


Figure 4-7. Effect of varying asphalt content on HWTD parameters for SR-9.5A mixture

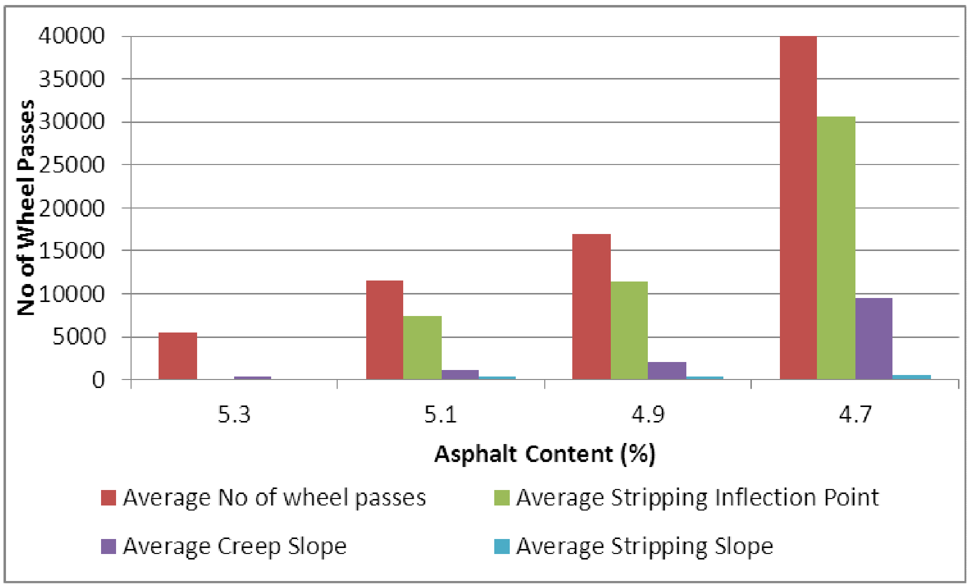


Figure 4-8. Effect of varying asphalt content on HWTD parameters for SR-12.5A mixture.

From the number of wheel passes data, we concluded that mixtures performed better in HWTD tests at asphalt contents lower than the design asphalt contents. This also can be affirmed from the HWTD parameters for Figures 4-7 and 4-8. The creep slope and stripping inflection points also increased with a decrease in asphalt content indicating the performance of the mixture was better for drier mixes. Figure 4-9 shows the trends in results are similar for the SR-19A mixture.

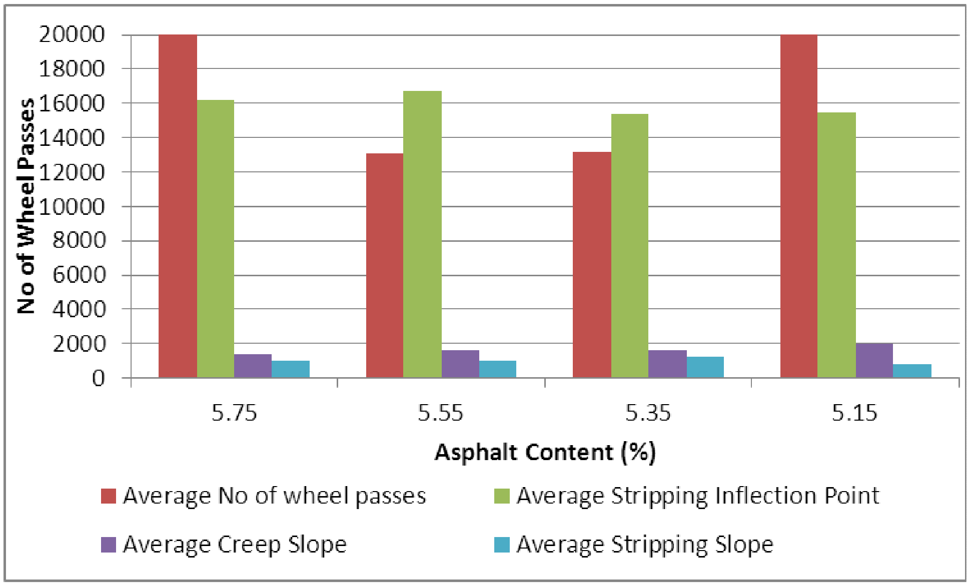


Figure 4-9. Effect of varying asphalt content on HWTD parameters for SR-19A mixture.

4.2 Kansas Standard Test KT-56

All KT-56 specimens were compacted to 7±0.5% air voids. For each asphalt content, at least six specimens were fabricated. Based on the air voids, the specimens were divided into two sets. One set (three specimens) was conditioned (freeze/thaw) and other set (three specimens) was unconditioned. The tensile strength of all specimens was determined in the indirect tension mode. The summary of the tensile strength and tensile strength ratios for the SR-9.5A mixture is presented in Table 4-8.

Table 4-8. Summary of tensile strength ratios for SR-9.5A mixture type

Asphlat content (%)	% Virgin Asphalt Added	Sample ID	Conditioned / Unconditioned	Air Voids (%) @Ndes	Tensile Strength (KPa)	Tensile Strength Ratio (%)	Avg Tensile Strength Ratio (%)
5.54	4.11	I1	Conditioned	8.0	690	86.0	96.5
		I2	Unconditioned	8.0	802		
		I3	Conditioned	8.1	751	105.5	
		I4	Unconditioned	8.0	712		
		I5	Conditioned	7.9	678	99.5	
		I6	Unconditioned	7.9	682		
5.34	3.91	J3	Conditioned	8.3	695	98.4	98.2
		J1	Unconditioned	8.5	707		
		J4	Conditioned	8.5	682	103.2	
		J2	Unconditioned	8.5	661		
		J5	Conditioned	8.4	660	93.3	
		J6	Unconditioned	8.4	707		
5.14	3.71	K2	Conditioned	8.2	620	80.4	80.2
		K1	Unconditioned	8.2	771		
		K3	Conditioned	8.3	577	80.0	
		K5	Unconditioned	8.1	721		
		K6	Conditioned	7.9	576	80.2	
		K4	Unconditioned	8.0	718		
4.94	3.51	L2	Conditioned	8.2	671	80.6	85.1
		L1	Unconditioned	8.2	833		
		L3	Conditioned	8.2	726	94.0	
		L5	Unconditioned	8.3	772		
		L6	Conditioned	8.5	603	80.8	
		L4	Unconditioned	8.4	746		

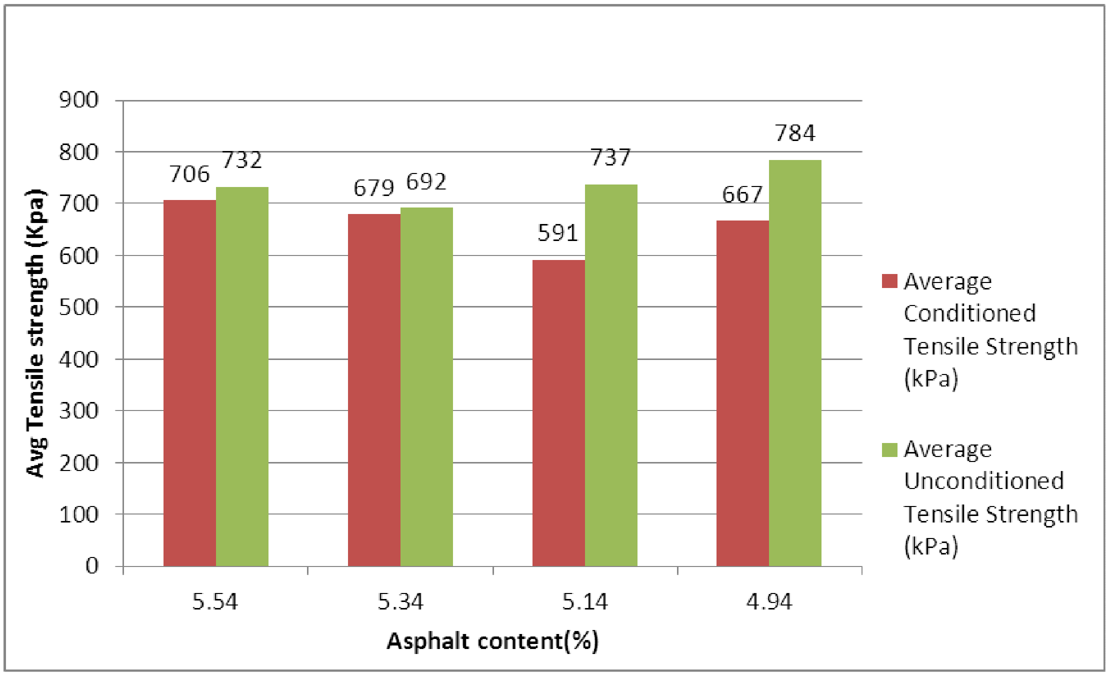


Figure 4-10. Tensile strength results for SR-9.5A mixture type.

The above plot shows the average tensile strengths of conditioned and unconditioned specimens prepared with varying asphalt content starting from the design asphalt content and continuing on to the drier side. The results indicate that as the asphalt content decreases, the tensile strength of unconditioned specimen increases, while the tensile strength of conditioned specimen decreases. For unconditioned specimens, the highest average tensile strength of 784 kPa was observed at an asphalt content of 4.94% and for conditioned, the highest average tensile strength of 706 kPa was observed at 5.54% (design asphalt content). At design asphalt content, the mix performed better in stripping.

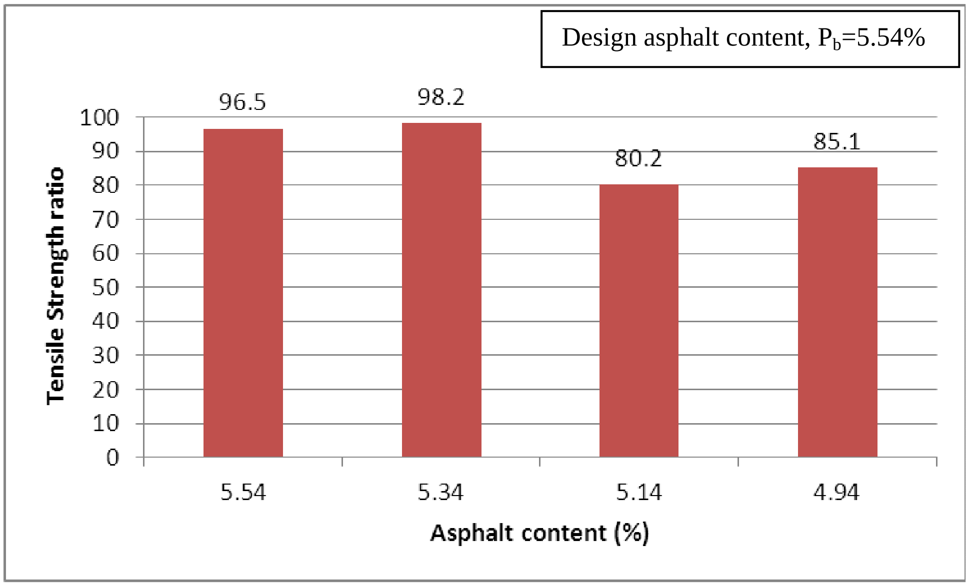


Figure 4-11. Tensile strength ratios for SR-9.5A mixture type.

For the SR-9.5A mixture, the highest tensile strength ratio (96.5 %) was observed at design asphalt content (5.54%) and the lowest tensile strength ratio (80.2 %) was observed at Pb-0.4% asphalt content (5.14%). Most state agencies require the tensile strength ratio to be greater than 80 and some agencies require greater than 70. The Kansas Department of Transportation (KDOT) criteria for TSR is ≥ 80. Thus tensile strength ratios for SR-9.5A ranged from a maximum of 96.5% to a minimum of 80.2%. TSR values were within KDOT specifications. From Figures 4-10 and 4-11, we concluded there was a significant decrease in the TSR, tensile strength of conditioned specimens and a significant increase in the tensile strength of unconditioned specimens, when the asphalt content in mix design changed from design asphalt content to Pb-0.6%.

Table 4-9. Tensile strength ratios for SR-12.5A mixture

Asphalt content (%)	% Virgin Asphalt added	Sample ID	Conditioned / Unconditioned	Air Voids (%)	Tensile Strength (KPa)	Tensile Strength Ratio	Avg Tensile Strength Ratio
5.3	4.14	E1	Conditioned	7.3	538.7	69.5	75.8
		E4	Unconditioned	7.2	775.4		
		E2	Conditioned	6.9	543.9	73.2	
		E3	Unconditioned	7.0	743.3		
		E6	Conditioned	7.1	675.9*	84.3	
		E5	Unconditioned	7.1	802.1		
5.1	3.94	F1	Conditioned	7.2	525.3	70.6	80.2
		F5	Unconditioned	7.0	744.4		
		F2	Conditioned	7.3	577.7	77.1	
		F3	Unconditioned	7.3	749.3		
		F4	Conditioned	6.9	663.1	93.6	
		F6	Unconditioned	7.0	708.6		
4.9	3.74	G1	Conditioned	7.0	684.8	75.5	87.7
		G2	Unconditioned	6.9	907.2		
		G3	Conditioned	6.6	803.8	92.0	
		G5	Unconditioned	6.8	873.5		
		G6	Conditioned	7.0	876.1	95.8	
		G4	Unconditioned	7.0	914.4		
4.7	3.54	H2	Conditioned	6.6	779.8	94.3	94.7
		H3	Unconditioned	6.7	827.2		
		H5	Conditioned	7.1	809.4	107.1	
		H1	Unconditioned	6.8	755.7		
		H6	Conditioned	7.2	682.1	83.6	
		H4	Unconditioned	7.3	815.7		

In Table 4-9, there is an outlier in the tensile strength for specimen E6 . Thus the value was omitted in calculations. The outlier was establised by conducting t-tests. The process is described here for the conditioned specimens subset at design asphalt content.

d_1 = lowest strength of specimen in subset = 538.7 kPa;

d_b = average strength of the subset = 586.2 kPa;

d_n = highest strength of specimen in subset = 675.9 kPa;

s = sample standard deviation (n-1) of the subset= 77.77, here n= no of specimens in subset= 3;

$t_{0.95}$ = “t” statistic value = 1.15, when n=3;

t_1= lower “t” value = $(d_b-d_1)/s$ = 0.61;

t_n = upper “t” value = $(d_n-d_b)/s$ = 1.15;

If the value of $t_{0.95}$ is greater than both the values of t_1 and t_n, then there are no outliers. In this case, the value of $t_{0.95}$ (1.15) was greater than t_1 (0.61) but equal to t_n (1.15). Therefore, the specimen with the highest strength was classified as an outlier and was not considered in calculating the average strength.

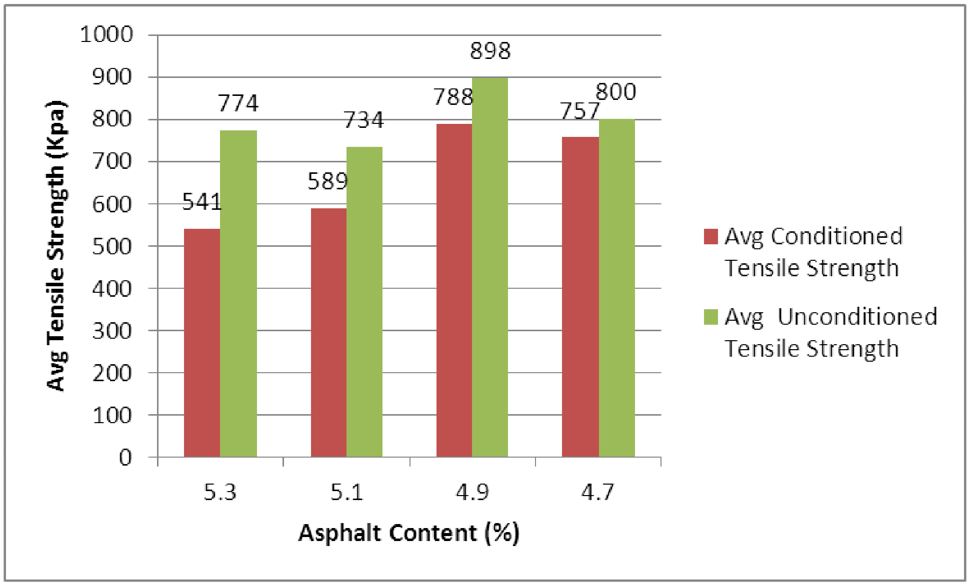


Figure 4-12. Tensile strength results for SR-12.5A mixture type.

From Figure 4-12, we can see the tensile strength of conditioned specimens increased from 541 kPa to 788 kPa when the asphalt content was decreased from 5.3 to 5.1%. The tensile strength decreased from 898 kPa to 800 kPa when the asphalt content was decreased from 4.9 to 4.7% . Overall, the decrease in asphalt content from the design asphalt content has increased the tensile strength rapidly by 31% and then decreased slightly. However, there was no definite trend observed in the tensile strength of the unconditioned specimens. The highest average tensile strength of conditioned specimens was observed at Pb-0.3% (4.9%). The range in the strengths of unconditioned specimens (164 kPa) was less when compared to the range of the conditioned specimens (247 kPa). This indicates the conditined specimen was more sensitive than the unconditioned specimen when the quantity of asphalt content varied. Thus, we concluded that providing lower asphalt content than that mentioned in the job mix formula would cause a slight increase in the tensile strength.

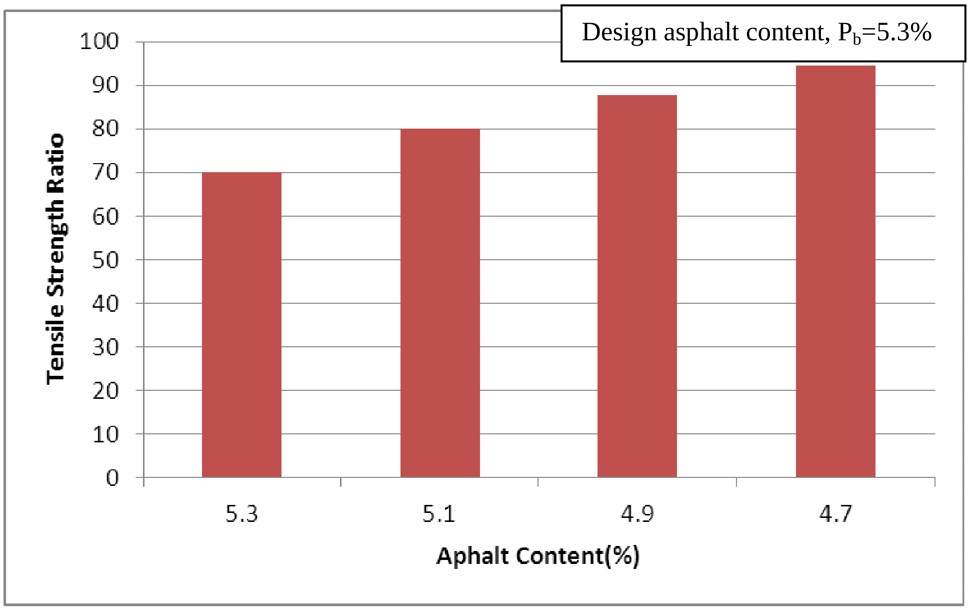


Figure 4-13. Tensile strength ratios for SR-12.5A mixture type.

Figure 4-13 shows the tensile strength ratio increased linearly as the asphalt content decreased. It was interesting to note the tensile strength ratio (TSR) was lowest (70 %) at the design asphalt content (5.3) and was maximum (94.7 %) at an asphalt content (P_b-0.6 %) much lower than the design asphalt content. The specimens prepared with the design asphalt content failed KDOT TSR criteria of ≥ 80, but the specimens with asphalt content P_b-0.2%, P_b-0.45, and P_b-0.6% passed. From Figures 4-12 and 4-13, we concluded that an increase in tensile strength and TSR occurred when the asphalt content in the mix design was below the design asphalt content.

Table 4-10. Summary of tensile strength ratios for SR-19A mixture type

Asphalt content (%)	Virgin asphalt added (%)	Sample ID	Conditioned / Unconditioned	Air voids (%)	Tensile strength (KPa)	Tensile strength ratio	Avg tensile strength ratio
5.75	4.4	A1	Conditioned	6.9	430.8	94.2	97.2
		A4	Unconditioned	6.5	457.0		
		A2	Conditioned	7.1	469.4	101.2	
		A5	Unconditioned	6.9	463.9		
		A6	Conditioned	7.0	450.1	96.2	
		A3	Unconditioned	7.2	467.9		
5.55	4.2	B1	Conditioned	6.5	455.7	86.3	94.1
		B4	Unconditioned	6.5	528.0		
		B2	Conditioned	6.9	465.4	99.5	
		B3/T2	Unconditioned	7.0	467.7		
		B5	Conditioned	6.5	522.7	97.0	
		B6	Unconditioned	6.5	538.7		
5.35	4	C2/T1	Conditioned	7.4	756.1	111.5	75.5
		C1	Unconditioned	7.4	678.3		
		C4	Conditioned	7.0	474.8	95.6	
		C5	Unconditioned	6.6	496.5		
		C6	Conditioned	7.1	484.6	66.1	
		C3	Unconditioned	7.4	732.5		
5.15	3.8	D3	Conditioned	6.4	493.5	100.0	94.3
		D2	Unconditioned	6.8	493.7		
		D5	Conditioned	7.4	426.9	97.9	
		D4	Unconditioned	7.3	436.3		
		D6	Conditioned	7.5	407.8	85.2	
		D1	Unconditioned	7.0	478.8		

The conditioned specimen C2 is an outlier in Table 4-10**.** Tensile strengths of the condition/unconditioned specimens of this mixture were much lower than the tensile strengths of the condition/unconditioned specimens of SR-9.5A and SR-12.5A mixtures. This may be due to the presence of high asphalt content in the plug.

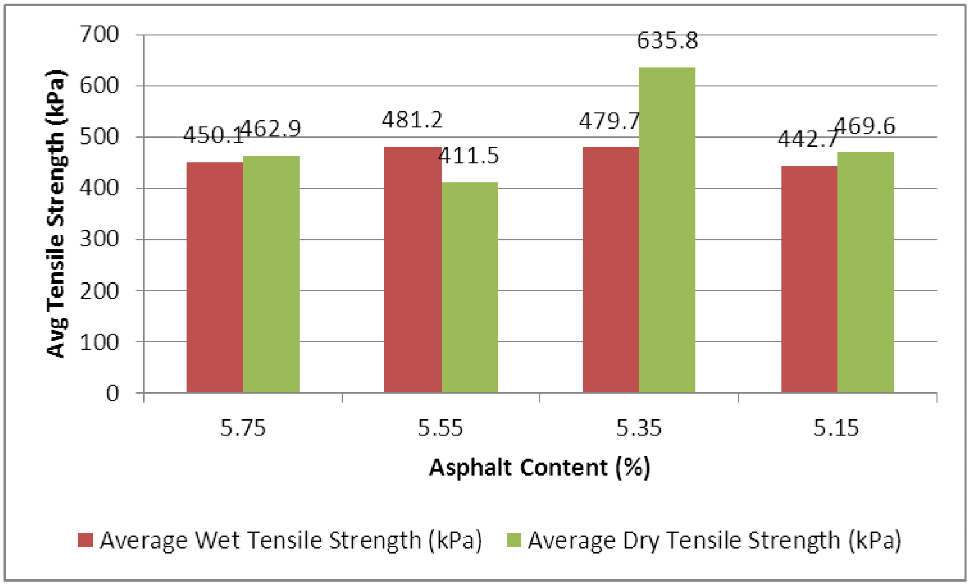


Figure 4-14. Tensile strength results for SR-19A mixture type.

Figure 4-14 shows there was not much variation in the tensile strength of the conditioned specimen. With the decrease in asphalt content, the strength increased and then decreased gradually. The trend was the same in the case with conditioned specimens, but there was an abrupt increase and decrease in tensile strengths when asphalt content changed from 5.55 to 5.15%. The average highest tensile strength (635.7 kPa) was observed at an asphalt content of 5.35% for the unconditioned sample and the lowest average tensile strength (442.7 kPa) was observed at an asphalt content of 5.15%.

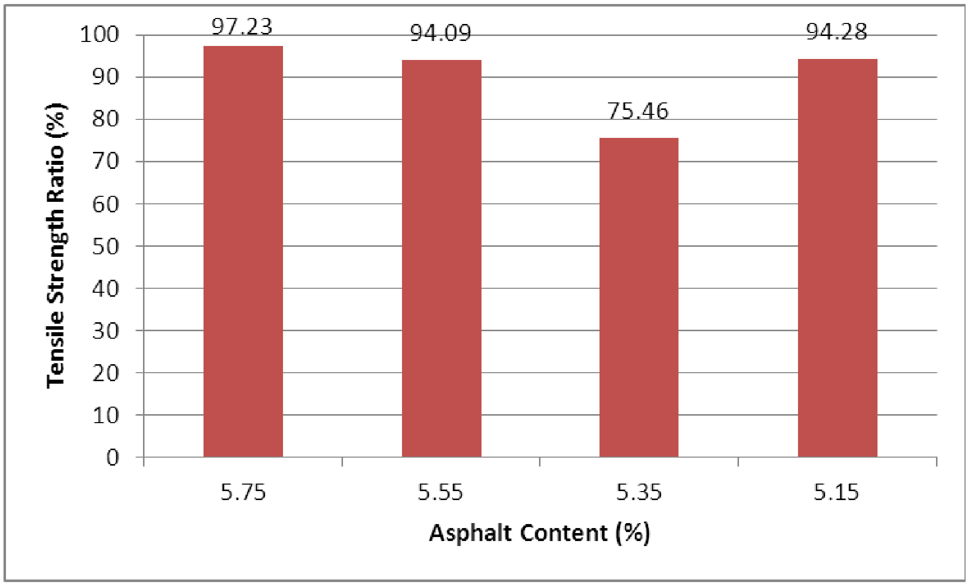


Figure 4-15. Tensile strength ratios for SR-19A mixture type.

The tensile strength ratios were higher except for a mixture designed with 5.35% asphalt content. Except for asphalt content of 5.35%, the TSR for other mixtures met KDOT criteria. But it should be noted that although TSR values were greater than 94%, it doesn't mean tensile strengths of the specimens were great. From Figure 4-14, we can see tensile strengths of the specimens were very low, ranging from as low as 442.7 kPa and to as high as 511.4 kPa (excluding tensile strengths at 5.35 % asphalt content). With the decrease in asphalt content, the trend of tensile strength ratio decresed and then suddenly increased after 5.35% asphalt content.

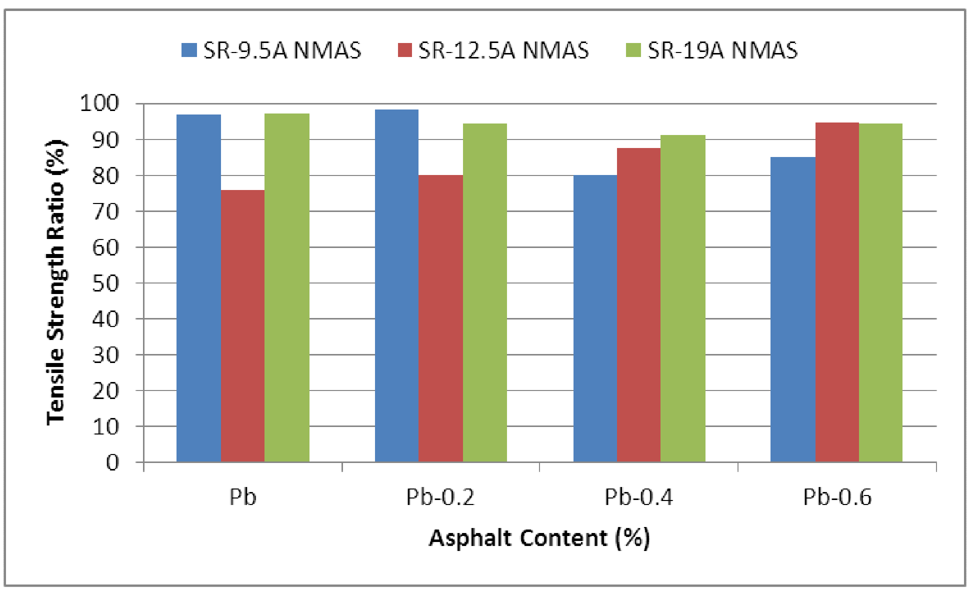


Figure 4-16. Tensile strength ratios for SR-9.5A, SR-12.5A, and SR-19A mixture types.

Figure 4-16 shows tensile strength ratios of all mixtures at each asphalt content, starting from the design asphalt content P_b and then in 0.2% decrements up to P_b-0.6%. Except for the 12.5A mixture at design asphalt content, all other mixtures at each asphalt content had TSR values greater than 80%, which meets current KDOT criteria for TSR.

4.3 Comparison of HWTD and KT-56 Results

Table 4-11. Summary of HWTD results and KT-56 results for SR-9.5A mixture

Asphalt Content (%)	Virgin Asphalt Added (%)	Tensile Strength Ratio (%)	Average No. of Wheel Passes To Reach 20 mm of Rut Depth	Average Stripping Inflection Point	Average Creep Slope (Passes/mm)	Average Stripping Slope (Passes/mm)
5.54(Pb)	4.11	96.5	5,087	3,295	523	181
5.34	3.91	98.2	13,929	5,763	1,413	275
5.14	3.71	80.2	10,856	7,327	1,928	276
4.94	3.51	85.1	11,861	8,128	2,153	289

A comparison was made between the HWTD results and KT-56 results. It is interesting to note that mixtures in HWTD performed worst at design asphalt content, while in TSR, performed better. At design asphalt content (5.54%), the TSR was 97 and the number of wheel passes from the HWTD test was 5,087, which were lower when compared to wheel passes obtained for drier mixtures. This can also be seen from the creep slope. At design asphalt content the TSR was 97% and creep slope 523, which is much lower when compared to the highest creep slope of 2,153 obtained at Pb-0.6% asphalt content. The mix with Pb-0.2% (5.34%) performed better in the KT-56 test and the mix with Pb-0.6% (4.94%) performed better in the HWTD test.

Table 4-12. Summary of HWTD results and KT-56 results for SR-12.5A mixture

Asphalt Content (%)	Virgin Asphalt Added (%)	Tensile Strength Ratio (%)	Average No of Wheel Passes To Reach 20 mm of Rut Depth	Average Stripping Inflection Point	Average Creep Slope (Passes/mm)	Average Stripping Slope (Passes/mm)
5.3(Pb)	4.14	70.0	5,380	0	348	0
5.1	3.94	80.2	11,543	7,383	1,107	331
4.9	3.74	87.7	16,852	11,450	2,043	444
4.7	3.54	94.7	40,244	30,650	9,438	567

In the SR-12.5A mixture, results from KT-56 (TSR) and HWTD (no. of passes/creep slope) indicated the mixture with design asphalt content performed worst. At the design asphalt content, the average number of wheel passes was 5,380, creep slope of 348, and TSR of 70%. The mixture with the lowest asphalt content (Pb-0.6%) performed better in both KT-56 and HWTD tests. At Pb-0.6% (4.7%) asphalt content, the number of wheel passes was 30,650, creep slope of 9,438, and TSR of 94.7%.

Table 4-13. Summary of HWTD results and KT-56 results for SR-19A mixture

Asphalt Content (%)	Virgin Asphalt Added (%)	Tensile Strength Ratio (%)	Average Stripping Inflection Point	Average Creep Slope (Passes/mm)	Average Stripping Slope (Passes/mm)
5.75	4.4	97.2	16,200	1,356	1,011
5.55	4.2	94.3	16,700	1,607	977
5.35	4	91.1	15,375	1,595	1,250
5.15	3.8	94.3	15,450	1,987	778

In Table 4-13, the number of wheel passes in the HWTD test was not included in the summary. This was because some of the HWDT tests samples had not failed till 20,000 passes or 20 mm rut depth, whichever came first. However, we can compare the stripping slope with the TSR values. The highest average TSR of 97.22% was observed for the mixture with 5.75% asphalt content, and least average TSR of 91.09% was observed for the mixture with 5.35% asphalt content; the highest average stripping slope of 16,700 was observed for the mixture with 5.55% asphalt content and the least average creep slope of 15,375 was observed for the mixture with 5.35% asphalt content.

4.4 Paired t- test

To compare two different asphalt contents, paired t-tests were also performed. The following tables show the results of the paired t-tests between different asphalt contents.

Table 4-14 Shows output of paired test for SR-9.5A mixture

Contrast	DF	Contrast SS	Mean Square	F Value	Pr > F
A_B	1	234534050.1	234534050.1	16.04	0.0007
A_C	1	99844083.0	99844083.0	6.83	0.0166
A_D	1	137654454.1	137654454.1	9.41	0.0061
B_C	1	28326914.1	28326914.1	1.94	0.1793
B_D	1	12829872.0	12829872.0	0.88	0.3601
C_D	1	3029070.1	3029070.1	0.21	0.6539

Parameter	Estimate	Standard Error	t Value	Pr > |t|
A_B	-8841.83	2207.73	-4.00	0.0007
A_C	-5769.00	2207.73	-2.61	0.0166
A_D	-6773.83	2207.73	-3.07	0.0061
B_C	3072.83	2207.73	1.39	0.1793
B_D	2068.00	2207.73	0.94	0.3601
C_D	-1004.83	2207.73	-0.46	0.6539

Note: A, B, C and D are asphalt contents, where A =5.54 %, B =5.34%, C =5.14%, and D =4.94%

From Table 4-14, at a 95% confidence interval ($p< 0.05$), there are significant differences in the number of wheel passes between A (5.54%) and B (5.34%); A (5.54%) and C (5.14%); and A (5.54%) and D (4.94%). There are no significant differences between B (5.34%) and C (5.14%); B (5.34%) and D (4.94%); and C (5.14%) and D (4.94%).

Table 4-15 Shows output of paired test for SR-12.5A mixture

Contrast	DF	Contrast SS	Mean Square	F Value	Pr > F
RA_RB	1	124073283	124073283	2.33	0.1422
RA_RC	1	416210965	416210965	7.83	0.0111
RA_RD	1	1457725633	1457725633	27.42	<.0001
RB_RC	1	85792616	85792616	1.61	0.2185
RB_RD	1	731234856	731234856	13.76	0.0014
RC_RD	1	316090145	316090145	5.95	0.0242

Parameter	Estimate	Standard Error	t Value	Pr > |t|
RA_RB	-6431.00	4209.28	-1.53	0.1422
RA_RC	-11778.66	4209.28	-2.80	0.0111
RA_RD	-22043.33	4209.28	-5.24	<.0001
RB_RC	-5347.66	4209.28	-1.27	0.2185
RB_RD	-15612.33	4209.28	-3.71	0.0014
RC_RD	-10264.66	4209.28	-2.44	0.0242

Note- RA, RB, RC and RD are asphalt contents where RA =5.3 %, RB =5.1%, RC =4.9%, RD =4.7%

As listed in Table 4-15, at a 95% confidence interval ($p< 0.05$), there are significant differences in the number of wheel passes between RA (5.3%) and RC (4.9%); RA (5.3%) and RD (4.7%); RB (5.1%) and RD (4.7%); RC (4.9%) and RD (4.7%). There are no significant differences between RA (5.3%) and RB (5.1%); RB (5.1%) and RC (4.9%).

4.5 Correlation of Asphalt Film Thickness and Voids in Mineral Aggregate (VMA) with HWDT and KT-56 Results

The study tried to establish a correlation between film thickness or VMA and the results obtained from the KT-56 and HWTD tests. The VMA and film thickness of each plug tested in HWDT and KT-56 tests were calculated. Tables 4-16, 4-17, 4-18, 4-19, and 4-20 tabulate the summaries of VMAs, film thickness, TSR values, and number of wheel passes for different mixtures.

4.5.1 KT-56

Table 4-16. Summary of VMAs, asphalt film thickness and tensile strength for SR-9.5A mixture

Asphalt Content (%)	Conditioned				Unconditioned			
	Air Voids (%) @Ndes	VMA (%) @Ndes	Film Thickness in Microns	Tensile Strength (KPa)	Air Voids (%) @Ndes	VMA (%) @Ndes	Film Thickness in Microns	Tensile Strength (KPa)
5.54	8.0	18.4	11.3	690	8.0	18.4	11.3	802
5.54	8.1	18.5	11.3	751	8.0	18.6	11.5	712
5.54	7.9	18.5	11.5	678	7.9	18.5	11.5	682
5.34	8.3	18.4	10.9	695	8.5	18.5	10.9	707
5.34	8.5	18.3	10.7	682	8.5	18.5	10.9	661
5.34	8.4	18.2	10.7	660	8.4	18.3	10.7	707
5.14	8.2	17.8	10.4	620	8.2	17.8	10.4	771
5.14	8.3	17.9	10.4	577	8.1	17.8	10.4	721
5.14	7.9	17.6	10.4	576	8.0	17.8	10.4	718
4.94	8.2	17.6	10.0	671	8.2	17.6	10.0	833
4.94	8.2	17.6	10.0	726	8.3	17.5	9.8	772
4.94	8.5	17.6	10.0	603	8.4	17.5	9.8	746

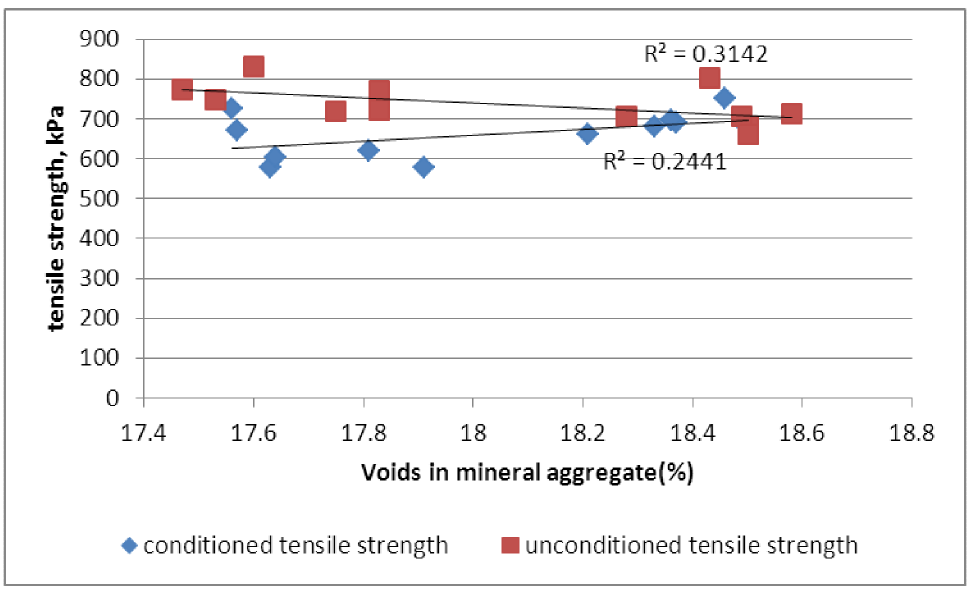


Figure 4-17. Relationship between VMA and tensile strength for SR-9.5A mixture.

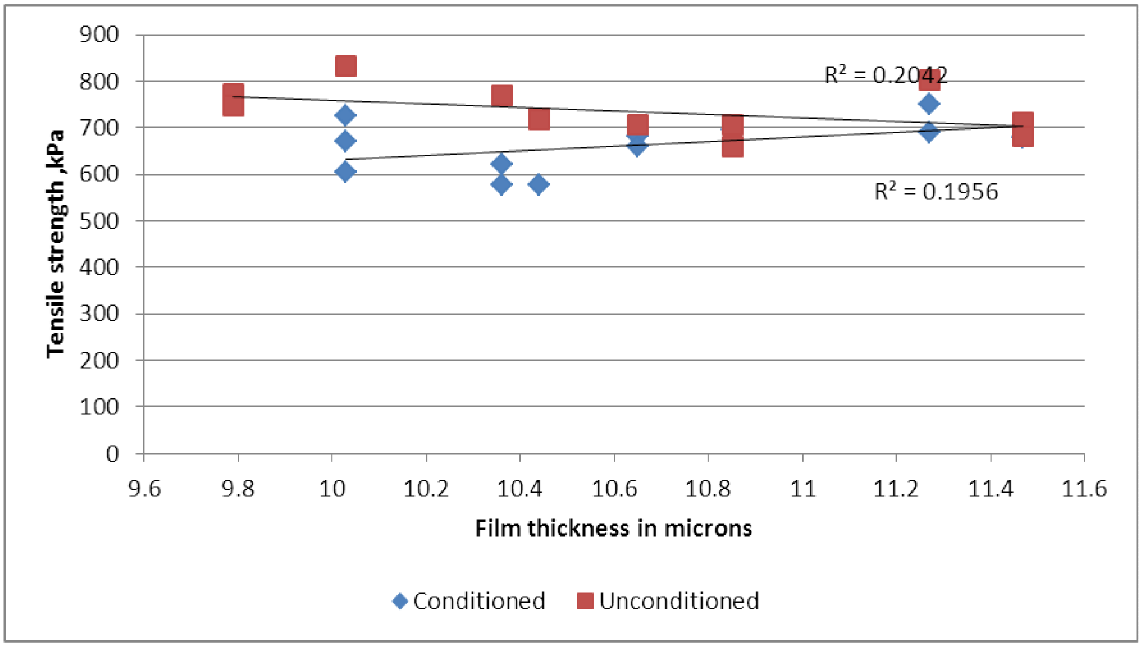


Figure 4-18. Relationship between asphalt film thickness and tensile strength for SR-9.5A mixture.

Figure 4-17 shows that as VMA increased,there was a decrease in unconditioned tensile strength and an increase in conditioned strength. Similarly from Figure 4-18, it appears that as film thickness increased, there was a decrease in unconditioned tensile strength and increase in conditioned strength. However, the coefficients of determination, R^2 values for the trendlines were not high. It appears that VMA and film thickness did not contribute much to the actual tensile strength of both conditioned and unconditioned specimens for the SR-9.5A mixture.

Table 4-17. Summary of VMAs, asphalt film thickness, and tensile strength for SR-12.5A mixture

Asphlat content (%)	Conditioned				Unconditioned			
	Air Voids (%) @Ndes	VMA (%) @Ndes	Film thickness in Microns	Tensile Strength (KPa)	Air Voids (%) @Ndes	VMA (%) @Ndes	Film thickness in Microns	Tensile Strength (KPa)
5.3	8.1	18.5	10.6	538.7	7.9	18.4	10.7	775.4
5.3	7.6	18.1	10.6	543.9	7.7	18.2	10.6	743.3
5.3	7.9	18.4	10.7	675.9	7.9	18.4	10.7	802.1
5.1	8.0	18.0	10.1	525.3	7.7	17.9	10.3	744.4
5.1	8.0	18.0	10.1	577.7	8.0	18.0	10.1	749.3
5.1	7.7	17.5	10.3	663.1	7.7	17.9	10.3	708.6
4.9	7.7	17.6	10.0	684.8	7.6	17.5	10.0	907.2
4.9	7.2	17.2	10.0	803.8	7.5	17.1	9.6	873.5
4.9	7.7	17.2	9.6	876.1	7.7	17.2	9.6	914.4
4.7	7.4	17.0	9.6	779.8	7.4	17.0	9.6	827.2
4.7	7.8	16.9	9.0	809.4	7.5	17.1	9.6	755.7
4.7	7.9	17.0	9.0	682.1	8.0	17.1	9.0	815.7

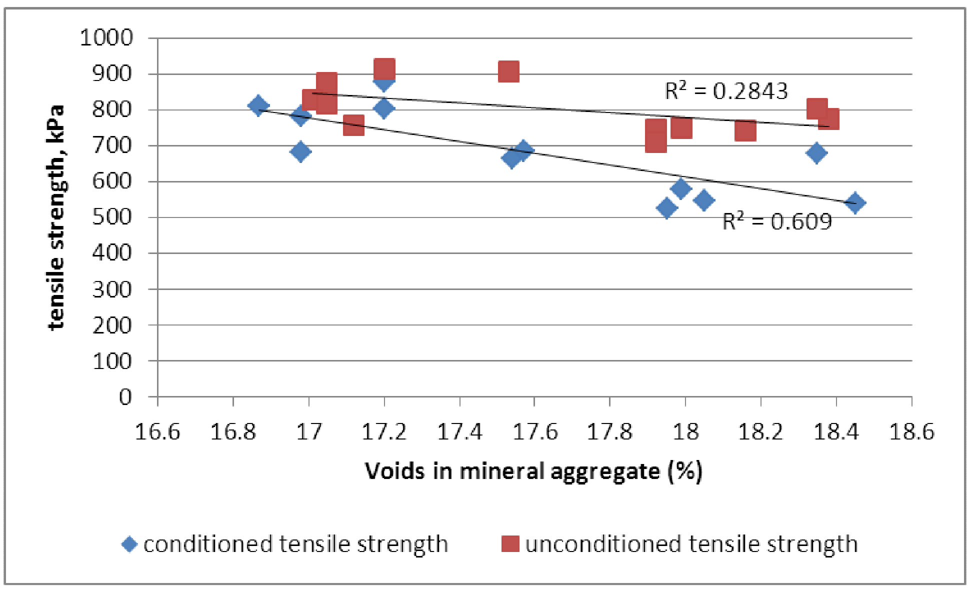


Figure 4-19. Relationship between VMA and tensile strength for SR-12.5A mixture.

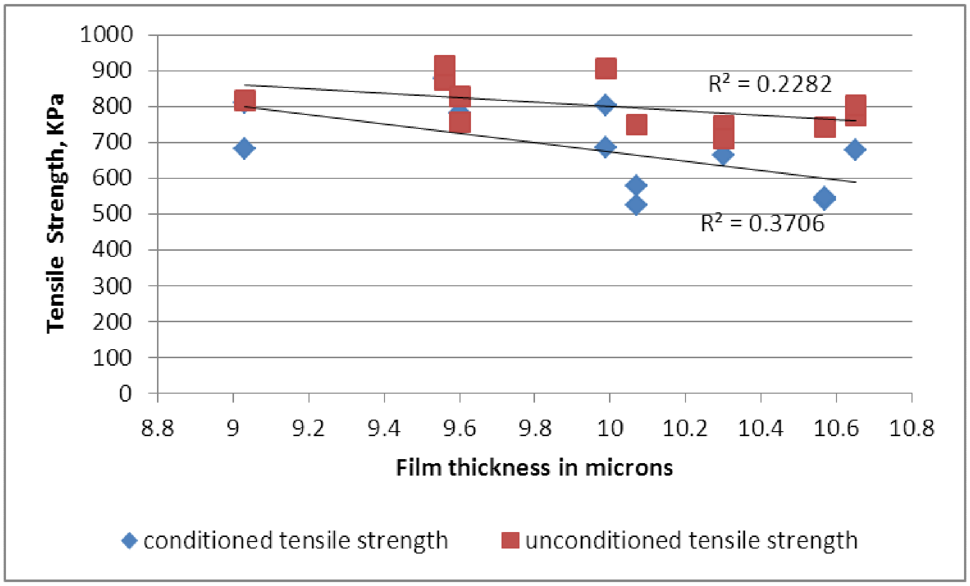


Figure 4-20. Relationship between asphalt film thickness and tensile strength for SR-12.5A mixture.

Figure 4-19 shows that as VMA increases,there is a decrease in tensile strengths (unconditioned and conditioned). Similarly Figure 4-20 shows that as film thickness increases, there is a decrease in tensile strength (unconditioned and conditioned). Again R^2 values for the trendlines obtained were not high. VMA and film thickness did not appear to contribute much to conditioned and unconditioned strengths for the SR-12.5A mixtures.

Table 4-18. Summary of VMAs, asphalt film thickness, and tensile strength for SR-19A mixture

Asphlat Content (%)	Conditioned				Unconditioned			
	Air Voids (%) @Ndes	VMA (%) @Ndes	Film Thickness in Microns	Tensile Strength (KPa)	Air Voids (%) @Ndes	VMA (%) @Ndes	Film Thickness in Microns	Tensile Strength (KPa)
5.75	7.2	16.3	11.7	430.8	6.8	16.6	12.6	457.0
5.75	7.4	16.5	11.7	469.4	7.2	17.0	12.6	463.9
5.75	7.3	17.0	11.7	450.1	7.5	16.6	12.6	467.9
5.55	6.8	15.4	10.9	455.7	6.9	15.5	10.9	528.0
5.55	7.2	15.7	10.9	465.4	7.3	15.9	10.9	467.7
5.55	6.8	15.4	10.9	522.7	6.8	15.5	10.9	538.7
5.35	7.6	15.2	9.6	756.1	7.6	15.6	10.1	678.3
5.35	7.3	15.5	10.5	474.8	6.9	15.2	10.5	496.5
5.35	7.3	15.6	10.5	484.6	7.6	15.5	10.1	732.5
5.15	7.0	15.2	10.3	493.5	7.4	15.5	10.3	493.7
5.15	8.0	15.9	10.1	426.9	7.9	15.8	10.1	436.3
5.15	8.1	16.0	10.1	407.8	7.6	15.7	10.3	478.8

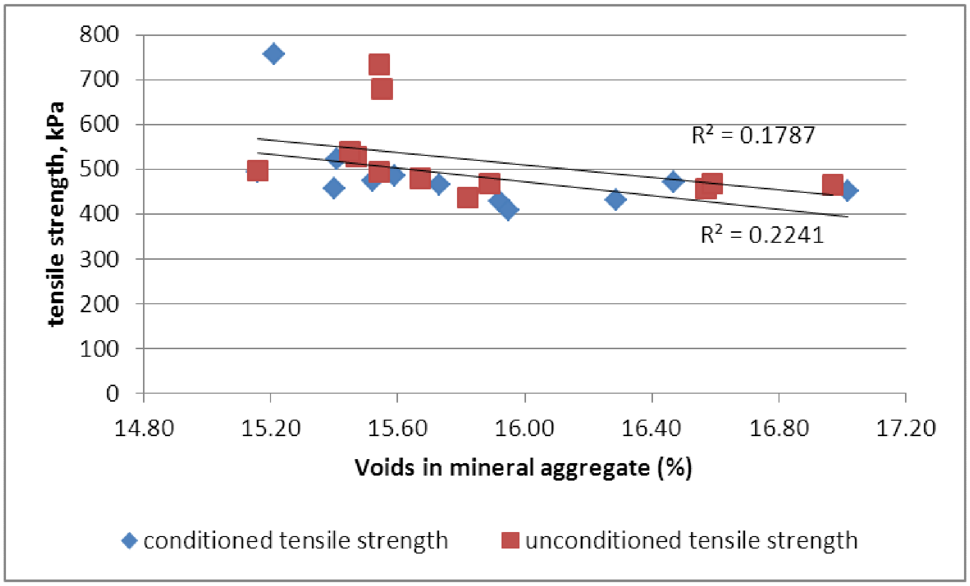


Figure 4-21. Relationship between VMA and tensile strength for SR-19A mixture.

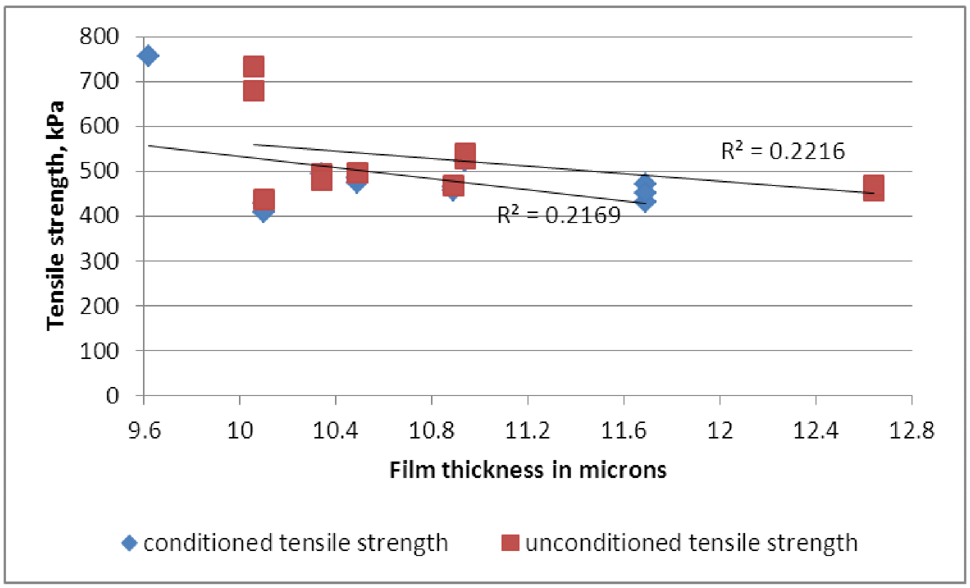


Figure 4-22. Relationship between asphalt film thickness and tensile strength for SR-19A mixture.

From Figure 4-21, it appears that as VMA increases,there is a decrease in tensile strengths (unconditioned and conditioned). Similarly from Figure 4-22, it appears that as film thickness increases, there is a decrease in tensile strength (unconditioned and conditioned). Again, R^2 values for the trendlines obtained were not high. Thus VMA and film thickness do not influence conditioned and unconditioned strengths much.

4.5.2 HWTD

Table 4-19. Summary of VMAs, asphalt film thickness, and number of wheel passes for SR-9.5A mixture

Asphalt Content (%)	Voids in Mineral Aggregate @ N_f (VMA)	Film Thickness in Microns	No. of Wheel Passes to Reach 20 mm of Rut Depth
5.54	17.5	11.4	5,759
	17.6	11.4	4,789
	17.4	11.4	2,250
	17.5	11.4	3,700
	17.5	11.4	7,433
	17.5	11.4	6,589
5.34	16.8	10.4	20,855*
	16.9	10.4	24,187*
	17.0	10.8	8,367
	17.0	10.7	9,091
	16.9	10.5	9,450
	16.9	10.5	11,621
5.14	16.8	10.2	8,867
	16.7	10.2	11,689
	16.6	10.1	12,679
	16.5	10.1	13,033
	16.7	10.2	9,217
	16.6	10.2	9,649
4.94	16.3	9.8	11,547
	16.4	9.8	11,049
	16.3	9.9	9,550
	16.4	9.9	10,903
	16.4	9.9	12,091
	16.3	9.8	16,023

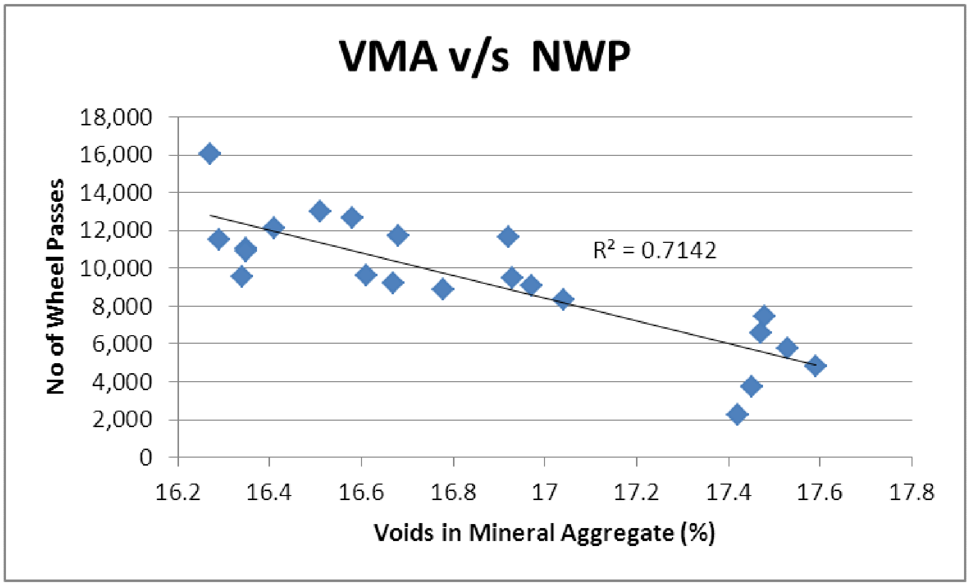


Figure 4-23. Relationship between VMA and number of wheel passes for SR-9.5A mixture.

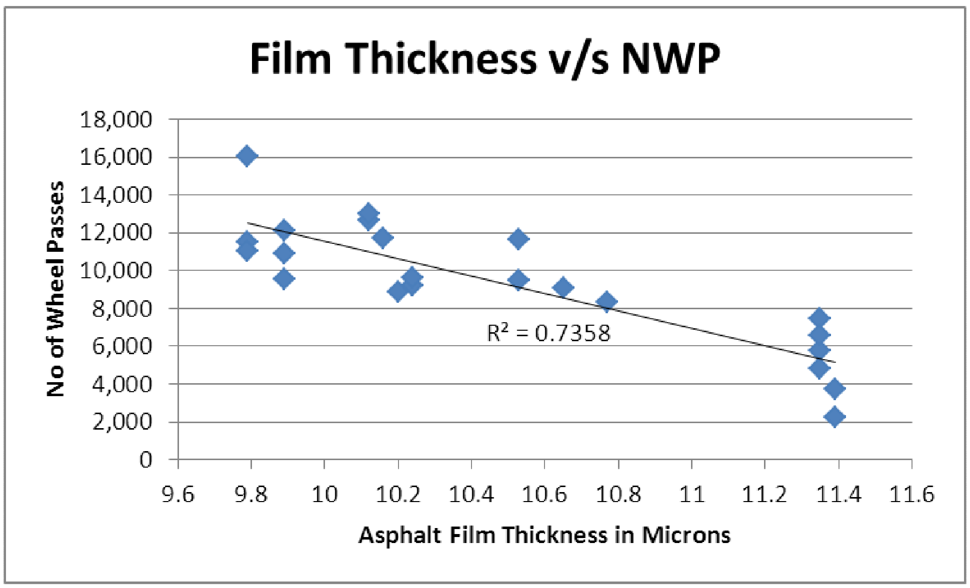


Figure 4-24. Relationship between asphalt film thickness and number of wheel passes for SR-9.5A mixture.

Figure 4-23 shows that as VMA increases, the number of wheel passes decreases. The trendline has an R^2 value of 0.71, meaning there is a good correaltion between the two. From Figure 4-24, it is evident that the number of wheel passes decreases as asphalt film thickness increases. The trendline has an R^2 value of 0.74 , which is a fairly high value.

Table 4-20. Summary of VMAs, asphalt film thickness, and number of wheel passes for SR-12.5A mixture

Asphalt Content (%)	Voids in Mineral Aggregate @ N_f (VMA)	Film Thickness in Microns	No. of Wheel Passes To Reach 20 mm of Rut Depth
5.3	17.5	10.8	15,723*
	17.6	10.8	26,211*
	17.5	10.5	4,113
	17.6	10.5	4,583
	17.6	10.7	5,291
	17.5	10.7	7,533
5.1	17.2	10.0	7,127
	17.2	10.0	11,347
	17.0	10.2	14,653
	17.1	10.2	12,621
	17.0	10.1	11,967
	17.0	10.1	25,563*
4.9	16.1	9.7	8,373*
	16.2	9.7	15,401
	16.5	9.6	29,541*
	16.5	9.6	26,893*
	16.5	9.8	16,637
	16.6	9.8	18,519
4.7	16.1	9.4	19,125*
	16.2	9.4	18,355*
	16.3	9.1	42,335
	16.3	9.1	38,153
	16.3	9.1	25,650*
	16.3	9.1	25,650*

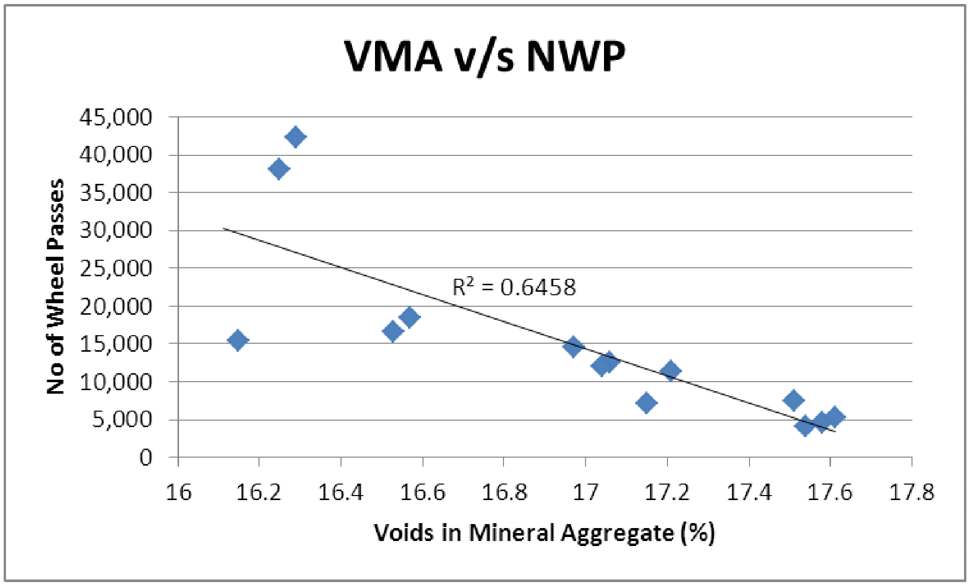


Figure 4-25. Relationship between VMA and number of wheel passes for SR-12.5A mixture.

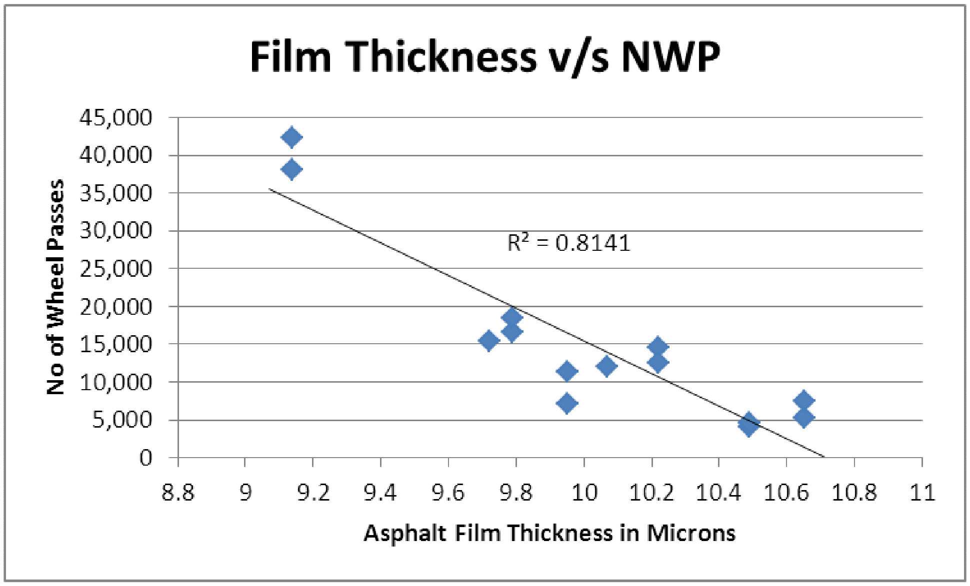


Figure 4-26. Relationship between asphalt film thickness and number of wheel passes for SR-12.5A mixture.

Figure 4-25 shows that as the VMA increases, the number of wheel passes decreases.The value of R^2 is 0.65. Figure 4-26 shows the number of wheel passes decreases as the asphalt film thickness increases. The trendline has a value of 0.81.

4.6 Statistical Analysis

The statistical analysis was done using Statistical Analysis System (SAS) software to identify the relationship between asphalt content, asphalt film thickness, VMA, and performance test results (HWTD and KT-56). However, Pearson correlation coefficients were computed for other variables and are presented in the appendix. The variables include asphalt content(%), air voids(%), VMA(%), VFA(%), film thickness in microns, number of wheel passes, creep slope (passes/mm), stripping slope (passes/mm), stripping inflection point, TSR (%), and dust-to-binder ratio. The value of the Pearson's correlation coefficient is the number between -1 to +1, which measures the degree of association between two variables. For this study, the strength of the relation between the two variables is defined in the following table.

Table 4-21. Interpretation of correlation

Correlation strength	Pearson's correlation coefficient	
	Positive	Negative
strong	0.7 to 1.0	-1.0 to -0.7
weak	0.3 to 0.7	-0.7 to -0.3
None/negligible	0.0 to 0.3	-0.3 to 0.0

Table 4-22. Correlation matrix for SR-9.5A mixture (HWTD)

	Asphalt Content	VMA @ Nf	VFA @ Nf	Film Thickness in Microns	No. of Passes	Creep Slope (Passes/mm)	Stripping Slope (passes/mm)	Stripping Inflection Point	Dust Binder Ratio
Asphalt Content	1	0.97 <.0001	0.94 <.0001	0.97 <.0001	-0.40 0.05	-0.10 0.63	-0.50 0.01	-0.46 0.03	-0.97 <.0001
VMA @ Nf	0.97 <.0001	1	0.95 <.0001	0.99 <.0001	-0.54 0.01	-0.21 0.32	-0.61 0.00	-0.59 0.00	-0.99 <.0001
VFA @ Nf	0.94 <.0001	0.95 <.0001	1	0.98 <.0001	-0.60 0.00	-0.22 0.30	-0.67 0.00	-0.64 0.00	-0.98 <.0001
Film Thickness in Microns	0.97 <.0001	0.99 <.0001	0.98 <.0001	1	-0.58 0.00	-0.22 0.29	-0.65 0.00	-0.62 0.00	-1.00 <.0001
No. of Passes	-0.40 0.05	-0.54 0.01	-0.60 0.00	-0.58 0.00	1	0.46 0.02	0.87 <.0001	0.98 <.0001	0.57 0.00
Creep Slope (Passes/mm)	-0.10 0.63	-0.21 0.32	-0.22 0.30	-0.22 0.29	0.46 0.02	1	0.46 0.03	0.43 0.04	0.21 0.32
Stripping Slope (passes/mm)	-0.50 0.01	-0.61 0.00	-0.67 0.00	-0.65 0.00	0.87 <.0001	0.46 0.03	1	0.89 <.0001	0.64 0.00
Stripping Inflection Point	-0.46 0.03	-0.59 0.00	-0.64 0.00	-0.62 0.00	0.98 <.0001	0.43 0.04	0.89 <.0001	1	0.62 0.00
Dust Binder Ratio	-0.97 <.0001	-0.99 <.0001	-0.98 <.0001	-1.00 <.0001	0.57 0.00	0.21 0.32	0.64 0.00	0.62 0.00	1

Table 4-22 shows that variables VMA and creep slope, and film thickness and creep slope have no correlation with each other. Some have a somewhat negative correlation with each other, such as VMA and number of wheel passes (-0.54), VMA and stripping slope (-0.61), VMA and stripping inflection point (-0.59), film thickness and number of wheel passes (-0.58), film thickness and stripping slope (-0.65), and film thickness and stripping inflection point (-0.62). Some have strong negative correlations with each other such as the variables dust-to-binder ratio and VMA (-0.99), dust-to-binder ratio, and film thickness (-0.99).

Table 4-23. Correlation matrix for SR-12.5A mixture (HWTD)

	Asphalt Content	VMA @ Nf	VFA @ Nf	Film Thickness in Microns	No. of Passes	Creep Slope (Passes/mm)	Stripping Slope (passes/mm)	Stripping Inflection Point	Dust Binder Ratio
Asphalt Content	1	0.96	0.66	0.98	-0.65	-0.69	-0.47	-0.57	-0.77
		<.0001	0.00	<.0001	0.00	0.00	0.02	0.00	<.0001
VMA @ Nf	0.96	1	0.45	0.92	-0.55	-0.57	-0.48	-0.53	-0.61
	<.0001		0.03	<.0001	0.01	0.00	0.02	0.01	0.00
VFA @ Nf	0.66	0.45	1	0.77	-0.58	-0.69	-0.25	-0.41	-0.73
	0.00	0.03		<.0001	0.00	0.00	0.23	0.05	<.0001
Film Thickness in Microns	0.98	0.92	0.77	1	-0.64	-0.71	-0.46	-0.56	-0.75
	<.0001	<.0001	<.0001		0.00	0.00	0.02	0.00	<.0001
No. of Passes	-0.65	-0.55	-0.58	-0.64	1	0.90	0.57	0.75	0.59
	0.00	0.01	0.00	0.00		<.0001	0.00	<.0001	0.00
Creep Slope (Passes/mm)	-0.69	-0.57	-0.69	-0.71	0.90	1	0.50	0.79	0.69
	0.00	0.00	0.00	0.00	<.0001		0.01	<.0001	0.00
Stripping Slope (passes/mm)	-0.47	-0.48	-0.25	-0.46	0.57	0.50	1	0.87	0.11
	0.02	0.02	0.23	0.02	0.00	0.01		<.0001	0.62
Stripping Inflection Point	-0.57	-0.53	-0.41	-0.56	0.75	0.79	0.87	1	0.37
	0.00	0.01	0.05	0.00	<.0001	<.0001	<.0001		0.08
Dust Binder Ratio	-0.77	-0.61	-0.73	-0.75	0.59	0.69	0.11	0.37	1
	<.0001	0.00	<.0001	<.0001	0.00	0.00	0.62	0.08	

Table 4-23 shows variables VMA and number of wheel passes (-0.55), VMA and creep slope (-0.57), VMA and stripping slope (-0.48), VMA and stripping inflection point (-0.53), VMA and dust-to-binder (-0.61), film thickness and number of wheel passes (-0.64), film thickness and stripping slope (-0.46), and film thickness and stripping inflection point (-0.56) have somewhat negative correlations with each other. Some have strong negative correlation with each other like the variables film thickness and creep slope (-0.71), and film thickness and dust-to-binder ratio (-0.75).

Table 4-24. Correlation matrix for SR-19A mixture (HWTD)

	Asphalt Content	VMA @ Nf	VFA @ Nf	Film Thickness in Microns	No. of Passes	Creep Slope (Passes/mm)	Stripping Slope (passes/mm)	Stripping Inflection Point	Dust Binder Ratio
Asphalt Content	1	0.54 0.01	0.79 <.0001	0.76 <.0001	-0.04 0.85	-0.24 0.27	0.13 0.75	0.13 0.73	-0.74 <.0001
VMA @ Nf	0.54 0.01	1	0.61 0.00	0.86 <.0001	0.46 0.02	0.18 0.40	0.53 0.14	0.40 0.28	-0.88 <.0001
VFA @ Nf	0.79 <.0001	0.61 0.00	1	0.93 <.0001	0.20 0.34	-0.08 0.71	-0.01 0.98	0.07 0.85	-0.91 <.0001
Film Thickness in Microns	0.76 <.0001	0.86 <.0001	0.93 <.0001	1	0.35 0.09	0.03 0.89	0.17 0.66	0.20 0.61	-1.00 <.0001
No. of Passes	-0.04 0.85	0.46 0.02	0.20 0.34	0.35 0.09	1	0.71 0.00	0.67 0.05	0.85 0.00	-0.34 0.10
Creep Slope (Passes/mm)	-0.24 0.27	0.18 0.40	-0.08 0.71	0.03 0.89	0.71 0.00	1	0.40 0.29	0.56 0.12	-0.04 0.84
Stripping Slope (passes/mm)	0.13 0.75	0.53 0.14	-0.01 0.98	0.17 0.66	0.67 0.05	0.40 0.29	1	0.79 0.01	-0.16 0.68
Stripping Inflection Point	0.13 0.73	0.40 0.28	0.07 0.85	0.20 0.61	0.85 0.00	0.56 0.12	0.79 0.01	1	-0.17 0.66
Dust Binder Ratio	-0.74 <.0001	-0.88 <.0001	-0.91 <.0001	-1.00 <.0001	-0.34 0.10	-0.04 0.84	-0.16 0.68	-0.17 0.66	1

Table 4-24 shows variables VMA and creep slope, film thickness and creep slope, film thickness and stripping slope, and film thickness and stripping inflection point have no correlation with each other. For all the above mentioned relationships, the value of p is greater than 0.05, so the result from this analysis may not be valid. Variables VMA and number of wheel passes have a somewhat positive correlation with each other. Some of the variables have a weak positive correlation with each other but the value of p is greater than 0.05, such as variables VMA and stripping slope (0.53), VMA and stripping inflection point (0.40), and film thickness and number of wheel passes (0.35). Some have a strong negative correlation with each other like VMA and dust-to-binder ratio (-0.87), and film thickness and dust-to-binder ratio (-0.99).

Table 4-25. Correlation matrix for SR-9.5A mixture (KT-56)

	Asphalt Content (%)	Air Voids (%)	VMA (%)	VFA (%)	Film Thickness in Microns	Dust Binder Ratio	TSR (%)
Asphalt Content (%)	1	-0.43 0.17	0.98 <.0001	0.90 <.0001	0.99 <.0001	-0.87 0.00	0.63 0.03
Air Voids (%)	-0.43 0.17	1	-0.28 0.37	-0.77 0.00	-0.53 0.08	0.67 0.02	-0.05 0.87
VMA (%)	0.98 <.0001	-0.28 0.37	1	0.83 0.00	0.96 <.0001	-0.83 0.00	0.67 0.02
VFA (%)	0.90 <.0001	-0.77 0.00	0.83 0.00	1	0.95 <.0001	-0.94 <.0001	0.48 0.12
Film Thickness in Microns	0.99 <.0001	-0.53 0.08	0.96 <.0001	0.95 <.0001	1	-0.91 <.0001	0.61 0.03
Dust Binder Ratio	-0.87 0.00	0.67 0.02	-0.83 0.00	-0.94 <.0001	-0.91 <.0001	1	-0.36 0.25
TSR (%)	0.63 0.03	-0.05 0.87	0.67 0.02	0.48 0.12	0.61 0.03	-0.36 0.25	1

Table 4-25 shows variables VMA and TSR (0.67), and film thickness and TSR (0.61) have somewhat positive correlation with each other.

Table 4-26. Correlation matrix for SR-12.5A mixture (KT-56)

	Asphalt Content (%)	Air Voids (%)	VMA (%)	VFA (%)	Film Thickness in Microns	Dust Binder Ratio	TSR (%)
Asphalt Content (%)	1	0.39 0.21	0.97 <.0001	0.62 0.03	0.95 <.0001	-0.94 <.0001	-0.64 0.02
Air Voids(%)	0.39 0.21	1	0.51 0.09	-0.44 0.15	0.19 0.55	-0.18 0.58	-0.53 0.08
VMA (%)	0.97 <.0001	0.51 0.09	1	0.54 0.07	0.94 <.0001	-0.93 <.0001	-0.70 0.01
VFA (%)	0.62 0.03	-0.44 0.15	0.54 0.07	1	0.79 0.00	-0.80 0.00	-0.22 0.50
Film Thickness in Microns	0.95 <.0001	0.19 0.55	0.94 <.0001	0.79 0.00	1	-1.00 <.0001	-0.60 0.04
Dust Binder Ratio	-0.94 <.0001	-0.18 0.58	-0.93 <.0001	-0.80 0.00	-1.00 <.0001	1	0.58 0.05
TSR (%)	-0.64 0.02	-0.53 0.08	-0.70 0.01	-0.22 0.50	-0.60 0.04	0.58 0.05	1

Table 4-26 shows variables VMA and TSR (-0.704) have strong negative correlation with each other. The variables film thickness and TSR (-0.6) have a weak negative correlation with each other.

Table 4-27. Correlation matrix for SR-19A mixture (KT-56)

	Asphalt Content (%)	Air Voids (%)	VMA (%)	VFA (%)	Film Thickness in Microns	Dust Binder Ratio	TSR (%)
Asphalt Content (%)	1	-0.35	0.78	0.80	0.90	-0.88	0.12
		0.27	0.00	0.00	<.0001	0.00	0.70
Air Voids (%)	-0.35	1	0.15	-0.77	-0.35	0.41	-0.03
	0.27		0.64	0.00	0.27	0.19	0.93
VMA (%)	0.78	0.15	1	0.51	0.87	-0.84	0.09
	0.00	0.64		0.09	0.00	0.00	0.78
VFA (%)	0.80	-0.77	0.51	1	0.86	-0.89	0.08
	0.00	0.00	0.09		0.00	<.0001	0.81
Film Thickness in Microns	0.90	-0.35	0.87	0.86	1	-1.00	0.10
	<.0001	0.27	0.00	0.00		<.0001	0.76
Dust Binder Ratio	-0.88	0.41	-0.84	-0.89	-1.00	1	-0.07
	0.00	0.19	0.00	<.0001	<.0001		0.83
TSR (%)	0.12	-0.03	0.09	0.08	0.10	-0.07	1
	0.70	0.93	0.78	0.81	0.76	0.83	

Table 4-27 shows variables VMA and TSR, and film thickness and TSR have no correlation with each other. But the value of p is greater than 0.05 and hence, the analysis is not valid.

Chapter 5 - Effects of Lower Asphalt Content on Quality Control/ Quality Assurance of HMA Mixes

5.1 Introduction

The impetus to develop statistics-oriented specifications for the highway industry similar to those used in the manufacturing industry was began in the early 1960s with the initiative led by the Bureau of Public Roads. This resulted in development and implementation of Portland cement concrete specifications in 1973, followed by their evaluation in 1979 (Diwan, 2003). In order to get a satisfactory product, quality control/quality assurance (QC/QA) programs are important. It is the combination of end-result specifications, and materials and methods specifications. The specifications that may be applicable for asphalt pavement construction can be classified as material-related specifications (MRS), end-result specifications (ERS), and performance-related specifications (PRS). Many highway agencies are now striving for PRS. In recent years many states have adopted statistical QC/QA programs to obtain a quality hot-mix asphalt construction. In Kansas, the contractor is responsible for QC and KDOT is responsible for QA. QA specification has become important for overall quality management.

5.2 Terminology (Glossary of Highway Quality Assurance Terms, TRB 2002)

- Quality control: "*Those QA actions and considerations necessary to assess and adjust production and construction processes so as to control the level of quality being produced in the end product.*"
- Quality assurance: "*All those planned and systematic actions necessary to provide confidence that a product or facility will perform satisfactorily in service.*"
- Specification limit(s): "*The limiting value(s) placed on a quality characteristic, established preferably by statistical analysis, for evaluating material or construction within the specification requirements. The term can refer to either an individual upper or lower specification limit, USL or LSL, called a single specification limit; or to USL and LSL together, called double specification limits.*"
- USL: "*Upper Specification Limit is the upper boundary below which a sample (an average of samples) may deviate from the target value*."

- LSL: "*Lower Specification Limit is the lower boundary above which a sample (an average of samples) may deviate from the target value*."
- Quality index (Q): Used to estimate the PWL. The Q value along with the PWL table is used to determine the estimated PWL.
- Percent within limits (PWL): "*The percentage of the lot falling above the LSL, beneath the USL, or between the USL and LSL*"

  $PWL = (PWL_U + PWL_L) - 100$

5.3 QC/QA Program of the Kansas Department of Transportation

It should be noted that definitions of QC/QA differ from industry to industry. KDOT's QC/QA definition is similar to the one mentioned in TRB's glossary of terms (Gedafa et al., 2011). Figures 5-1 and 5-2 illustrate QC specifications for materials and properties that are to be achieved by the contractor for different Superpave HMA during production.

The current QC/QA program of KDOT pays incentives/disincentives for air voids and in-place density (density pay adjustment P_D and air void pay adjustment P_V). The pay factors were calculated as follows:

I. Density pay adjustment P_D

   a) Density pay adjustment for HMA overlay

      The density pay factors are presented in the Figure 5-3. Calculation for density pay factor A1, A2 and A3:

      $A1 = [100 + 4(\% \text{ of lot } G_{mm} - 92.0)]/100$

      $A2 = [84 + 16 (\% \text{ of lot } G_{mm} - 90.0)]/100$

      $A3 = [84 + 16 (\% \text{ of lot } G_{mm} - 89.0)]/100$

      Density pay adjustment factor (P_D)* = Density pay factor – 1.000

      *P_D shall be rounded to the nearest thousandth.

   b) Density pay adjustment for HMA Surface, HMA Base and HMA Pavement:

      $P_D = (PWL_{LD} * 0.004) - 0.360$

TABLE 602-1: COMBINED AGGREGATE REQUIREMENTS										
Nom. Max. Size Mix Designation	Percent Retained – Square Mesh Sieves								Min. VMA (%)	D/B Ratio
	1"	3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 200		
SM-4.75A			0	0-5	0-10		40-70	88-94	16.0	0.9 – 2.0
SM-9.5A SR-9.5A			0	0-10	10 min.	33-53		90-98	15.0	0.6 – 1.2
SM-9.5B SR-9.5B			0	0-10	10 min.	53-68		90-98	15.0	0.8 – 1.6
SM-9.5T SR-9.5T			0	0-10	10 min.	53-68		90-98	15.0	0.8 – 1.6
SM-12.5A SR-12.5A		0	0-10	10 min.		42-61		90-98	14.0	0.6 – 1.2
SM-12.5B SR-12.5B		0	0-10	10 min.		61-72		90-98	14.0	0.8 – 1.6
SM-19A SR-19A	0	0-10	10 min.			51-65		92-98	13.0	0.6 – 1.2
SM-19B SR-19B	0	0-10	10 min.			65-77		92-98	13.0	0.8 – 1.6

Figure 5-1. Combined aggregate requirements (KDOT, Division 600 Flexible Pavement).

TABLE 602-12: SPECIFICATION WORKING RANGES (QC/QA)				
Mix Characteristic	Tolerance from JMF			
	Single Test Value	Plot	4 Point Moving Average Value	Plot
Binder Content	±0.6%	*	±0.3%	*
Mix Characteristic	Tolerance for Specification Limits			
	Single Test Value	Plot	4 Point Moving Average Value	Plot
Gradation (applicable sieves in TABLE 602-1)	N/A	*	zero tolerance	*
Air Voids @ N_{des} gyrations	±2.0%	*	N/A	
Voids in Mineral Aggregate (VMA)	1.0% below min.	*	zero tolerance	*
Voids Filled with Asphalt (VFA)	N/A		zero tolerance	*
Course Aggregate Angularity (CAA)	zero tolerance		N/A	
Sand Equivalent (SE)	zero tolerance		N/A	
Fine Aggregate Uncompacted Voids (FAA)	zero tolerance		N/A	
%Tensile Strength Ratio (%TSR)	zero tolerance	*	N/A	
Density @ N_{ini} and N_{max}	N/A		zero tolerance	
Dust to Effective Binder (D/B) Ratio	zero tolerance	*	N/A	*

Figure 5-2. Specification working ranges (QC/QA) (KDOT, Division 600 Flexible Pavement).

TABLE 602-15: DENSITY PAY FACTORS FOR SPECIFIED THICKNESS[4]			
Specified Thickness →	≥ 2"	≥ 1½"	
	All	Continuous Action[5]	No Continuous Action[6]
% of G_{mm} Average of 10 Density Tests[1]	Pay Factor[2]		Pay Factor[2]
93.0% or greater	1.040		1.040
92.0 to 92.9%	A1		A1
91.0 to 91.9%	1.000		1.000
90.0 to 90.9%	A2		1.000
89.0 to 89.9%	0.840 or Remove[3]		A3
less than 89.0%	0.840 or Remove[3]		0.840 or Remove[3]

Figure 5-3. Density pay factors for specified thickness (KDOT, Division 600 Flexible Pavement).

II. Air void pay adjustment P_V

For passing t-tests:

a) Calculate $Q_{UV} = \frac{(USL - \bar{X})}{S}$ and $Q_{LV} = \frac{(\bar{X} - LSL)}{S}$

where $\bar{X}$ is the average measured V_a of all samples within the lot,

USL is the upper specification limit for V_a (5%),
LSL is the lower specification limit for V_a (3%), and
S is the standard deviation of the measured V_a for all samples within a lot.

b) $P_V = ((PWL_{UV} + PWL_{LV} - 100)(0.0030)) - 0.270$

where PWL_{UV} = upper Percent Within Limits value for V_a, and

PWL_{LV} = lower Percent Within Limits value for V_a.

c) For failing t-test: Values from the Table 602-16 of the KDOT specifications (shown below) are used to calculate the P_V.

TABLE 602-16: Statistical Values for Air Voids Pay Adjustment for Failing t-Test		
Term	**Definition**	**Value**
$\bar{X}$	Average or Mean	KDOT's test result for the lot
S	Standard Deviation	0.50
USL	Upper Specification Limit	5.50%
LSL	Lower Specification Limit	2.50%
N	Sample Size	3

Figure 5-4. Statistical parameters for air voids pay adjustment for failing t-tests (KDOT, Division 600 Flexible Pavement).

5.4 Determination of PWLs and Expected Life for SR Mixtures in This Study

A recent study on the QA/QC data analysis of KDOT Superpave HMA projects by Gedafa et al. (2011) developed practical performance models and composite index. In this study, the model proposed by Gedafa et al. evaluated by incorporating mixture characteristics of the two Superpave mixtures (SR-12.5A and SR-9.5A). The expected life (EL) of the pavement was determined by substituting PWLs of air voids, density, asphalt content, and voids in mineral aggregate in the equation 5.1 developed by Gedafa et al. (2011):

$$EL = e^{-5.450+0.287PWL_{VA}^{0.5}+0.219PWL_{DEN}^{0.5}+0.173PWL_{AC}^{0.5}+0.138PWL_{VMA}^{0.5}} \quad (5.1)$$

where

Va= air voids,
DEN = in-place density,

AC= asphalt content, and
VMA= voids in mineral aggregate.

To determine the effect of HMA mixtures produced at asphalt contents lower than the design asphalt content, cylindrical Superpave specimens were prepared in the laboratory for SR-9.5A and SR-12.5A mixtures at varying asphalt contents, starting from design asphalt content and moving on to the drier side. At each asphalt content, two plugs were compacted using the Superpave gyratory compactor. Air voids at N_{design} and VMA at N_{design} were calculated for both plugs and the average of the two values was taken for PWL calculations. Tables 5-1 and 5-2 list these computed parameters for SR-9.5A and SR-12.5A, respectively.

Table 5-1. Volumetric properties for SR-9.5A mixture

Asphalt content (%)	Air voids (%) @ Ndes	VMA @Ndes
5.54(design asphalt content)	3.4	14.6
5.34	3.9	14.6
5.14	3.8	13.9
4.94	5.1	14.5

Table 5-2. Volumetric properties for SR-12.5A mixture

Asphalt content (%)	Air voids (%) @ Ndes	VMA @Ndes
6.44	4.4	17.7
6.24	4.7	17.9
6.04	4.4	17.2
5.84	5.2	17.3

The steps followed to determine the PWL are described below:

1. The mean '$\bar{X}$' and standard deviation 'S' were found.

2. The upper quality index value 'Q_u' was calculated by the equation
$$Q_u = \frac{(USL - \bar{X})}{S}$$

3. The lower quality index value 'Q_L' was calculated by the equation
$$Q_L = \frac{(\bar{X} - LSL)}{S}$$

4. The percentage falling below the USL (PWL_U) was estimated using the computed Q_u value and using a table of corresponding PWL values.

5. The percentage falling above the LSL (PWL_L) was estimated using the computed Q_L value and using a table of corresponding PWL values.

6. The percent within limit (PWL) was determined by the equation

   $PWL= (PWL_U- PWL_L)-100.$

The PWL values were calculated by making some assumptions for the standard deviation 'S' for various variables. The standard deviation for asphalt content, air voids, and VMA is 0.2, 0.5, and 1.0 respectively. The computed PWLs are shown in Tables 5-3 and 5-4 for SR-9.5A and SR-12.5A, respectively.

Table 5-3. Percent within limits for SR-9.5A mixture

Asphalt content	Air voids	VMA	Density*
50.0	100	66.8	100
83.3	100	68.7	100
83.3	100	47.3	100
50.0	77.6	65.5	100

Table 5-4. Percent within limits for SR-12.5A mixture

Asphalt content	Air voids	VMA	Density*
50.0	100	100	100
83.3	100	100	100
83.3	100	100	100
50.0	68.0	100	100

Density *- The PWL for density was taken as 100%, assuming density will be always above the LSL.

5.5 Expected Life

The expected lives of the pavements incorporating these mixtures with varying levels of PWL were computed using Equation 5-1. Table 5.5 tabulates the results. From this table, it is evident the mixtures performed worse at asphalt contents on the drier side, i.e. at asphalt contents that were lower than the design asphalt content. Expected lives for the driest mixtures (design binder content – 0.6%) studied were 2.55 years and 2.1 years for SR-12.5A and SR-9.5A mixtures, respectively. Although there seemed to be increased life for the Superpave mixtures that were somewhat drier (design binder content – 0.4%), there was a drastic decrease in life after that.

Table 5-5. Expected Life for SR-9.5A and SR-12.5A mixtures

SR-9.5A		SR-12.5A	
Asphalt content	Expected life	Asphalt content	Expected life
5.54	4.0	6.44	6.4
5.34	7.4	6.24	11.4
5.14	5.5	6.04	11.4
4.94	2.1	5.84	2.6

Computed PWLs and expected lives indicate the expected life model proposed by Gedafa et al. (2011) is sensitive to all volumetric properties (VMA, air voids) and asphalt content). Specific observations are as below:

Asphalt Content: For the SR-12.5A mixture, the expected life increased from 6.4 years to 11.4 years when PWL for the asphalt content was increased from 50 to 83.3%, while keeping air voids, VMA, and density PWLs at 100%. This shows the model was responsive to the changes in asphalt content.

Air Voids: For the SR-9.5A mixture, the expected life decreased from 4.0 to 2.1 years when PWL of air voids was decreased from 100 to 77.6%, keeping asphalt content (PWL=50%), VMA (PWL=66%), and density (PWL=100%) constant. This shows the model was responsive to changes in air voids as well.

VMA: For the SR-9.5A mixture, the expected life decreased from 7.4 to 5.5 years when PWL of VMA was decreased from 68.7 to 47.3%, keeping asphalt content (PWL=83.3%), air voids, and density (PWL=100%) constant. This shows the model was responsive to changes in VMA.

Thus the model developed with air voids, VMA, asphalt content, and in-place density seemed logical as results ranged from a maximum expected life of 11.4 years for good quality (PWL's for VMA=100%, air voids=100%, asphalt content = 83.3%, respectively) and expected life of 2.6 years for poor quality (PWL's for VMA=100%, air voids=68%, asphalt content = 50%, respectively). However, the model needs to be validated by correlating it with actual field performance data.

Chapter 6 - Conclusions and Recommendations

6.1 Conclusions

The main objective of this research study was to investigate the moisture resistance of Superpave HMA mixtures with varying asphalt content. The other objective was to investigate effects of voids in mineral aggregate and asphalt film thickness on the performance of the mixes. To allow for inherent material and production variability, KDOT's QC/QA program allows ±0.6% (single test value) or ±0.3% (4-point moving average value) variation from the design asphalt content mentioned in the job-mix formula. But some contractors are taking this as an advantage and producing drier mixtures. To investigate the effect of these drier mixes, two performance tests, Hamburg Wheel Tracking Device (HWTD) test and KT-56 tests were selected. Based on results obtained from the two tests, the following conclusions can be drawn:

- The amount of asphalt content in the mixture significantly affected the rutting and moisture resistance of HMA mixtures.
- For SR-9.5A and SR-12.5A mixtures, the number of wheel passes, creep slope, and stripping inflection point was higher at the dry side of optimum asphalt content.
- For SR-9.5A and SR-12.5A mixtures, the number of wheel passes, creep slope, and stripping inflection point increased as asphalt content decreased.
- For the SR-9.5A mixture, highest tensile strength ratio was observed at design asphalt content and for the SR-12.5A mixture, highest tensile strength ratio was observed at the dry side of the design asphalt content.
- As the asphalt content in the mixture decreased, the unconditioned tensile strength of both the mixes increased, while the conditioned tensile strength of the SR-9.5A mixture decreased and conditioned tensile strength of SR-12.5A increased.
- All mixtures produced in the laboratory met the tensile strength ratio (TSR) criteria specified by the Kansas Department of Transportation, except the SR-12.5A mixture at design asphalt content.
- From the QC/QA analysis of drier mixtures, the expected life was worst for drier mixtures. The expected life is as high as 11.4 years to a least of 2.6 years. The model developed seemed reasonable from the results generated.
- From the statistical analysis (correlation table), the following conclusions can be made:

i. There is a weak negative correlation (-0.7< Pearson coefficient <-0.3) between the variable voids in mineral aggregate/film thickness and HWTD output parameters i.e., as the VMA/film thickness increases there is a decrease in the number of passes of HWTD parameters and vice-versa.

ii. For the SR-9.5A mixture, there is a weak positive correlation (0.7< Pearson coefficient <0.3) between the variable voids in mineral aggregate/film thickness and tensile strength ratios i.e., as the VMA/film thickness increases there is a decrease in the tensile strength ratios and vice-versa.

iii. For the SR-12.5A mixture, there is a strong negative correlation (-0.7< Pearson coefficient <-1.0) between voids in mineral aggregate and tensile strength ratios, and a weak negative correlation (-0.7< Pearson coefficient <-0.3) between film thickness and tensile strength ratios.

In summary, as there are weak correlations between the voids in mineral aggregate/film thickness and performance test results, the study is nonconclusive from a durability point of view. However, performance simulations using a theoretical model show that asphalt pavements with dry mixes are likely to have shorter performance lives.

6.2 Recommendations

In order to know the effect of drier mixtures on the performance of the pavement, the Hamburg Wheel Tracking Device and KT-56 tests should be conducted on the Superpave mixtures containing the reclaimed asphalt pavement.

a) Further tests are recommended on Superpave mixtures (without reclaimed asphalt pavement) to determine the exact behavior of the drier mixtures.

b) Further study is recommended on Superpave mixtures using a crack simulation test like the Texas overlay tester that may evaluate true behavior of the drier mixtures.

c) The low-temperature behavior of the drier mixes should also be evaluated.

d) The practical performance model should be evaluated with more Superpave mixtures and correlated with actual field data.

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Appendix A - Volumetric Properties and Test Results

Table A-1 Volumetric properties and HWTD test results for SR-9.5A mixture.

Asphalt Content (%)	Air Voids @Ndesign (%)	VMA @Ndesign (%)	Film Thickness in Microns	No. of Passes To Reach a Rut Depth of 20mm	Creep slope (Passes/ mm)	Stripping slope (passes/m m)	Stripping Inflection Point	Post Compaction (@1000 passes)
5.54	7.92	18.44	11.35	5,759	740	185	3,100	2.5
5.54	8.02	18.5	11.35	4,789	282	NA	NA	3.2
5.54	7.79	18.33	11.39	2,250	85	NA	NA	6.2
5.54	7.82	18.36	11.39	3,700	314	122	2,360	4
5.54	7.9	18.39	11.35	7,433	807	232	3,750	2.2
5.54	7.89	18.38	11.35	6,589	909	185	3,970	2.5
5.34	8.1	17.8	10.41	20,855	4750	381	15,250	2
5.34	8.18	17.86	10.41	24,187	5625	447	17,200	1.5
5.34	8.02	18.03	10.77	8,367	1232	225	5,150	2.5
5.34	8.06	17.95	10.65	9,091	1236	273	5,200	2
5.34	8.12	17.91	10.53	9,450	1471	262	6,000	2
5.34	8.12	17.91	10.53	11,621	1714	338	6,700	2
5.14	8.11	17.63	10.2	8,867	1500	237	5,700	2.2
5.14	8.04	17.53	10.16	11,689	1714	364	7,800	2
5.14	7.98	17.44	10.12	12,679	2360	286	9,100	1.7
5.14	7.9	17.37	10.12	13,033	2700	286	9,400	1.8
5.14	7.96	17.52	10.24	9,217	1763	240	5,760	1.6
5.14	7.9	17.47	10.24	9,649	1533	240	6,200	2
4.94	8.11	17.28	9.79	11,547	2360	259	8,200	1.6
4.94	8.18	17.34	9.79	11,049	1970	274	7,520	1.9
4.94	8.07	17.33	9.89	9,550	1450	253	6,300	1.9
4.94	8.08	17.34	9.89	10,903	2200	238	7,650	1.8
4.94	8.15	17.4	9.89	12,091	1288	320	9,000	2
4.94	8.09	17.27	9.79	16,023	3650	387	10,100	1.5

Table A-2 Volumetric properties and KT-56 test results for SR-9.5A mixture.

Asphlat Content (%)	Conditioned / Unconditioned	Air Voids (%) @Ndes	VMA (%) @Ndes	Film Thickness in Microns	Tensile Strength (KPa)	Avg Conditioned Tensile Strength (KPa)	Avg Unconditioned Tensile Strength (KPa)	Avg Tensile Strength Ratio
5.54	Conditioned	7.96	18.37	11.27	690	706	732	96.5
	Unconditioned	8.01	18.43	11.27	802			
	Conditioned	8.05	18.46	11.27	751			
	Unconditioned	8	18.58	11.47	712			
	Conditioned	7.9	18.5	11.47	678			
	Unconditioned	7.91	18.5	11.47	682			
5.34	Conditioned	8.32	18.36	10.85	695	679	692	98.2
	Unconditioned	8.46	18.49	10.85	707			
	Conditioned	8.47	18.33	10.65	682			
	Unconditioned	8.48	18.5	10.85	661			
	Conditioned	8.35	18.21	10.65	660			
	Unconditioned	8.42	18.28	10.65	707			
5.14	Conditioned	8.16	17.81	10.36	620	591	737	80.2
	Unconditioned	8.18	17.83	10.36	771			
	Conditioned	8.27	17.91	10.36	577			
	Unconditioned	8.11	17.83	10.44	721			
	Conditioned	7.89	17.63	10.44	576			
	Unconditioned	8.02	17.75	10.44	718			
4.94	Conditioned	8.2	17.57	10.03	671	667	784	85.1
	Unconditioned	8.23	17.6	10.03	833			
	Conditioned	8.2	17.56	10.03	726			
	Unconditioned	8.32	17.47	9.79	772			
	Conditioned	8.5	17.64	10.03	603			
	Unconditioned	8.38	17.53	9.79	746			

Table A-0-3 Volumetric properties and HWTD test results for SR-12.5A mixture.

Asphalt Content (%)	Air Voids @Ndes (%)	VMA @Ndes (%)	Film Thickness in Microns	No. of Passes to Reach a Rut Depth of 20 mm	Creep Slope (Passes/mm)	Stripping Slope (passes/mm)	Stripping Inflection Point	Post Compaction (@1000 passes)
5.3	7.6	18.22	10.77	15,723	1090.9	NA	NA	2.1
5.3	7.6	18.25	10.77	26,211	1725	NA	NA	2
5.3	7.9	18.21	10.49	4,113	283	NA	NA	4.8
5.3	7.9	18.25	10.49	4,583	279.09	NA	NA	4.9
5.3	7.8	18.27	10.65	5,291	340	NA	NA	4.3
5.3	7.7	18.18	10.65	7,533	488.88	NA	NA	3.5
5.1	8	17.82	9.95	7,127	771.42	225	4,080	2.4
5.1	8	17.89	9.95	11,347	1300	330	6,900	2.2
5.1	7.5	17.63	10.22	14,653	1657.1	340	10,500	2
5.1	7.6	17.72	10.22	12,621	981.25	430	8,050	2.2
5.1	7.7	17.72	10.07	11,967	825	NA	NA	2.3
5.1	7.7	17.72	10.07	25,563	3107.1	704.5	16,750	2
4.9	7	16.72	9.72	8,373	476	NA	NA	3.1
4.9	7	16.76	9.72	15,401	1953.1	425.92	10,800	2.2
4.9	7.5	17.08	9.56	29,541	3900	761.5	19,500	1.8
4.9	7.5	17.08	9.56	26,893	4030	685	17,600	2
4.9	7.4	17.15	9.79	16,637	1675	493.33	10,550	1.7
4.9	7.4	17.19	9.79	18,519	2500	411.66	13,000	1.8
4.7	7.4	16.82	9.37	19,125	2833.33	381.81	13,750	1.6
4.7	7.5	16.87	9.37	18,355	2760	483.33	11,400	1.8
4.7	7.8	16.97	9.14	42,335	8875	668.18	31,250	1.2
4.7	7.8	16.93	9.14	38,153	10000	466.66	30,050	1.2
4.7	7.9	16.95	9.07	25,650	2678.57	NA	NA	1.4
4.7	7.9	16.96	9.07	25,650	6000	NA	NA	1.4

Table A-4 Volumetric properties and KT-56 test results for SR-12.5A mixture.

Asphlat Content (%)	Conditioned / Unconditioned	Air Voids Ndes (%)	VMA Ndes (%)	Film Thickness in Microns	Tensile Strength (KPa)	Average Conditioned Tensile Strength (KPa)	Average Unconditioned Tensile Strength (KPa)	Average Tensile Strength Ratio
5.3	Conditioned	8.06	18.45	10.57	538.69	586	774	75.8
	Unconditioned	7.91	18.38	10.65	775.44			
	Conditioned	7.61	18.05	10.57	543.94			
	Unconditioned	7.74	18.16	10.57	743.32			
	Conditioned	7.86	18.35	10.65	675.94			
	Unconditioned	7.86	18.35	10.65	802.09			
5.1	Conditioned	8	17.95	10.07	525.26	589	734	80.2
	Unconditioned	7.73	17.92	10.3	744.37			
	Conditioned	8.04	17.99	10.07	577.74			
	Unconditioned	8.04	17.99	10.07	749.25			
	Conditioned	7.65	17.54	10.3	663.12			
	Unconditioned	7.73	17.92	10.3	708.6			
4.9	Conditioned	7.65	17.57	9.99	684.76	788	898	87.7
	Unconditioned	7.6	17.53	9.99	907.23			
	Conditioned	7.23	17.2	9.99	803.77			
	Unconditioned	7.49	17.05	9.56	873.48			
	Conditioned	7.66	17.2	9.56	876.1			
	Unconditioned	7.66	17.2	9.56	914.44			
4.7	Conditioned	7.37	16.98	9.6	779.78	757	800	94.7
	Unconditioned	7.41	17.01	9.6	827.21			
	Conditioned	7.82	16.87	9.03	809.39			
	Unconditioned	7.54	17.12	9.6	755.65			
	Conditioned	7.94	16.98	9.03	682.08			
	Unconditioned	8.02	17.05	9.03	815.65			

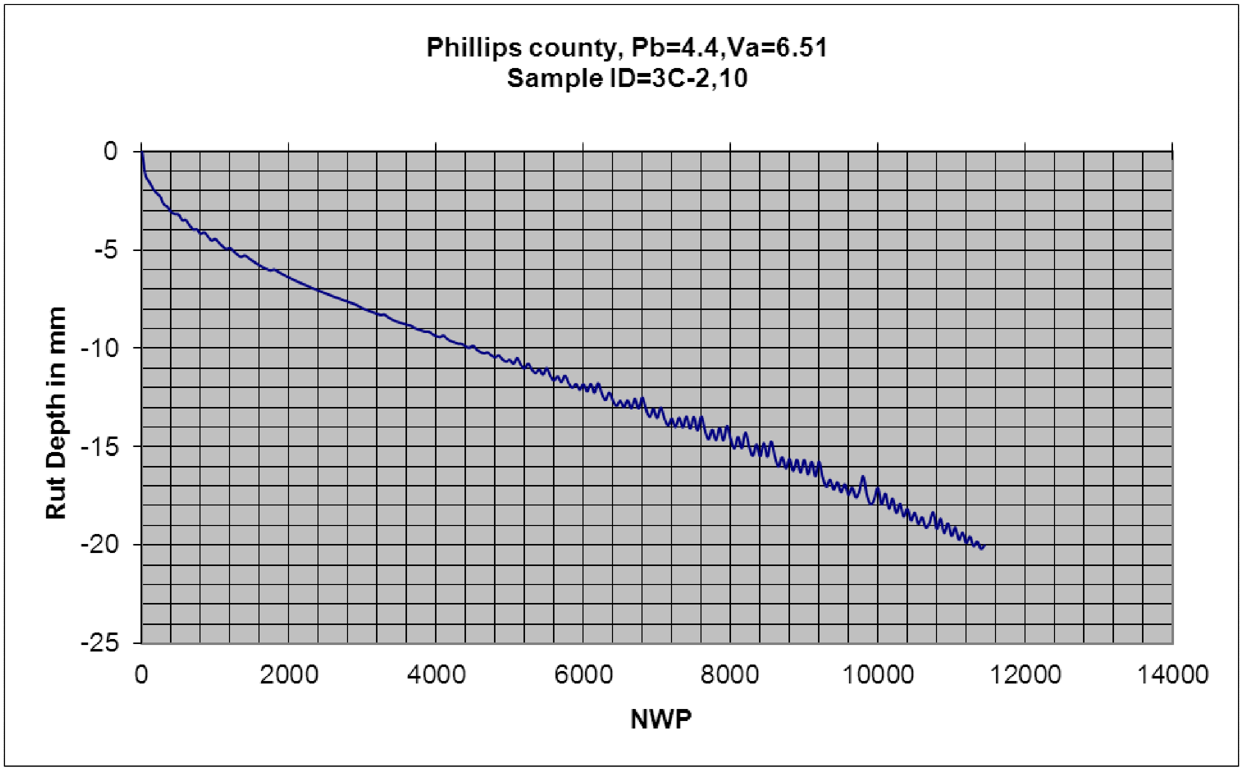
Table A-5 Volumetric properties and HWTD test results for SR-19Amixture.

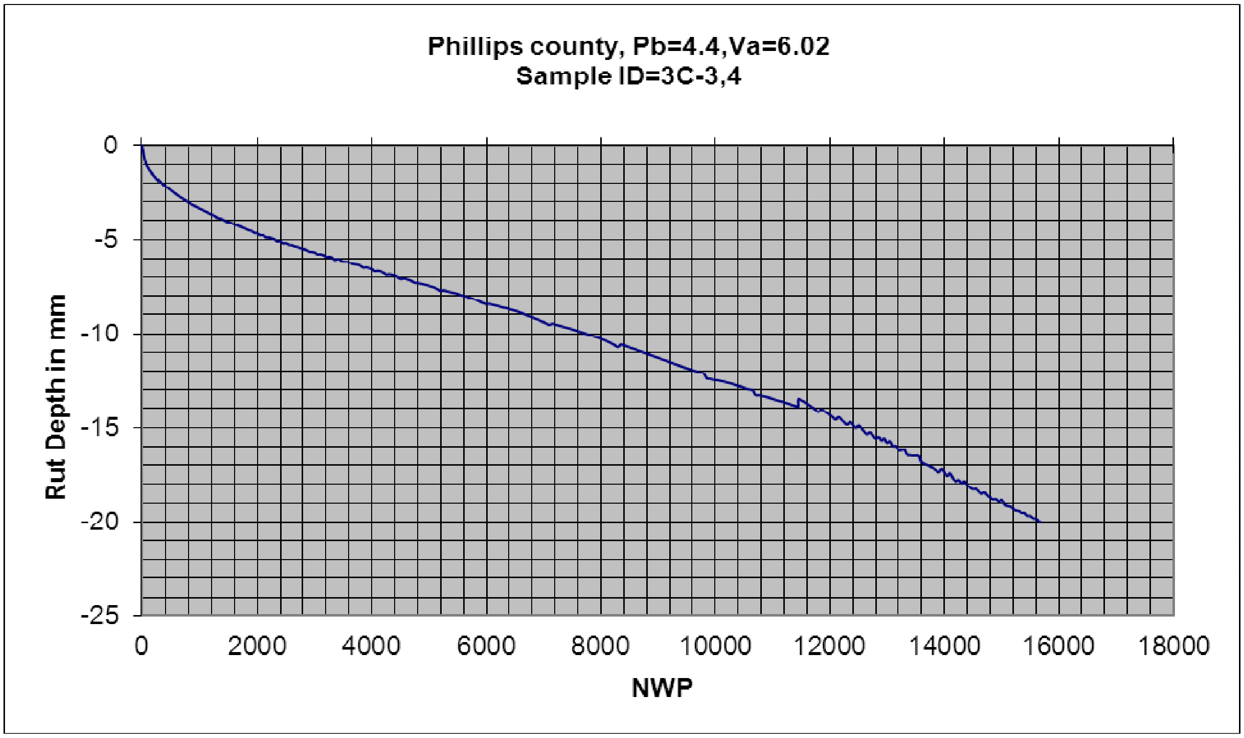
Asphalt Content (%)	Air Voids (%)	VMA (%)	Film Thickness in Microns	No. of passes	Rut Depth in mm	Creep Slope (Passes/mm)	Stripping Slope (passes/mm)	Stripping Inflection Point	Post Compaction (@1000 passes)
5.75	6.77	15.4	11.59	15,683	20	1033	600	12600	3.5
5.75	6.28	15.8	11.51	11,447	20	700	NA	NA	4.5
5.75	7.23	16.16	11.44	16,250	20	957	NA	NA	3.9
5.75	7.25	16.18	11.44	21,975	20	1414	NA	NA	3.8
5.75	6.44	15.97	12.19	21,050	20	1529	861	13500	2.8
5.75	6.57	16.09	12.19	35,241	20	2500	1571	22500	2.7
5.55	6.9	15.29	10.65	20,000	12.8	3225	875	15400	2.3
5.55	7.09	15.47	10.65	20,000	13.1	3684	1080	18000	2.6
5.55	7.15	14.89	9.77	13,565	20	888	NA	NA	4.3
5.55	7.12	14.87	9.77	11,959	20	800	NA	NA	3.8
5.55	7.07	14.68	9.58	4,950	20	325	NA	NA	7.1
5.55	6.89	14.51	9.58	8,200	20	722	NA	NA	4.9
5.35	7.62	15.69	10.28	20,000	14.9	2364	938	14250	2.2
5.35	7.39	15.48	10.28	20,000	11.1	2525	1563	16500	2.5
5.35	6.98	15.23	10.45	8,938	20	509	NA	NA	5.0
5.35	7.13	15.36	10.45	8,038	20	500	NA	NA	4.7
5.35	7.42	15.21	9.86	20,000	14.42	2050	NA	NA	3.5
5.35	7.49	15.27	9.86	20,000	16.99	1625	NA	NA	3.2
5.15	7.78	15.53	9.86	20,000	11.6	2958	912	17000	2.4
5.15	7.44	15.22	9.86	20,000	17	1900	644	13900	2.5
5.15	7.62	15.45	9.95	20,000	16.47	1442	NA	NA	2.4
5.15	7.59	15.43	9.95	20,000	13.95	1875	NA	NA	2.9
5.15	7.38	14.88	9.47	20,000	14.81	1714	NA	NA	2.8
5.15	7.58	15.07	9.47	20,000	11.69	2031	NA	NA	2.7

Table A-6 Volumetric properties and KT-56 test results for SR-19A mixture.

Virgin Asphalt Added(%)	Conditioned / Unconditioned	Air Voids (%)	VMA (%)	Film Thickness in Microns	Tensile Strength (KPa)	Average Wet Tensile Strength (KPa)	Average Dry Tensile Strength (KPa)	Average Tensile Strength Ratio
4.4	Conditioned	7.18	16.29	11.69	430.81	450.09	462.90	97.23
	Unconditioned	6.76	16.57	12.64	456.96			
	Conditioned	7.38	16.47	11.69	469.4			
	Unconditioned	7.2	16.97	12.64	463.88			
	Conditioned	7.26	17.02	11.69	450.07			
	Unconditioned	7.51	16.59	12.64	467.87			
4.2	Conditioned	6.82	15.4	10.89	455.68	481.23	511.47	94.09
	Unconditioned	6.86	15.47	10.94	527.99			
	Conditioned	7.18	15.73	10.89	465.36			
	Unconditioned	7.28	15.89	10.89	467.73			
	Conditioned	6.8	15.41	10.94	522.66			
	Unconditioned	6.83	15.45	10.94	538.69			
4	Conditioned	7.62	15.21	9.62	756.05	479.73	635.76	75.46
	Unconditioned	7.64	15.55	10.06	678.25			
	Conditioned	7.27	15.52	10.49	474.84			
	Unconditioned	6.87	15.16	10.49	496.54			
	Conditioned	7.34	15.59	10.49	484.62			
	Unconditioned	7.64	15.54	10.06	732.5			
3.8	Conditioned	6.99	15.16	10.34	493.53	442.74	469.59	94.28
	Unconditioned	7.4	15.54	10.34	493.67			
	Conditioned	8.01	15.92	10.1	426.93			
	Unconditioned	7.9	15.82	10.1	436.32			
	Conditioned	8.05	15.95	10.1	407.76			
	Unconditioned	7.55	15.67	10.34	478.79			

Appendix B - Hamburg Wheel Tracking Device Test Results (plots)

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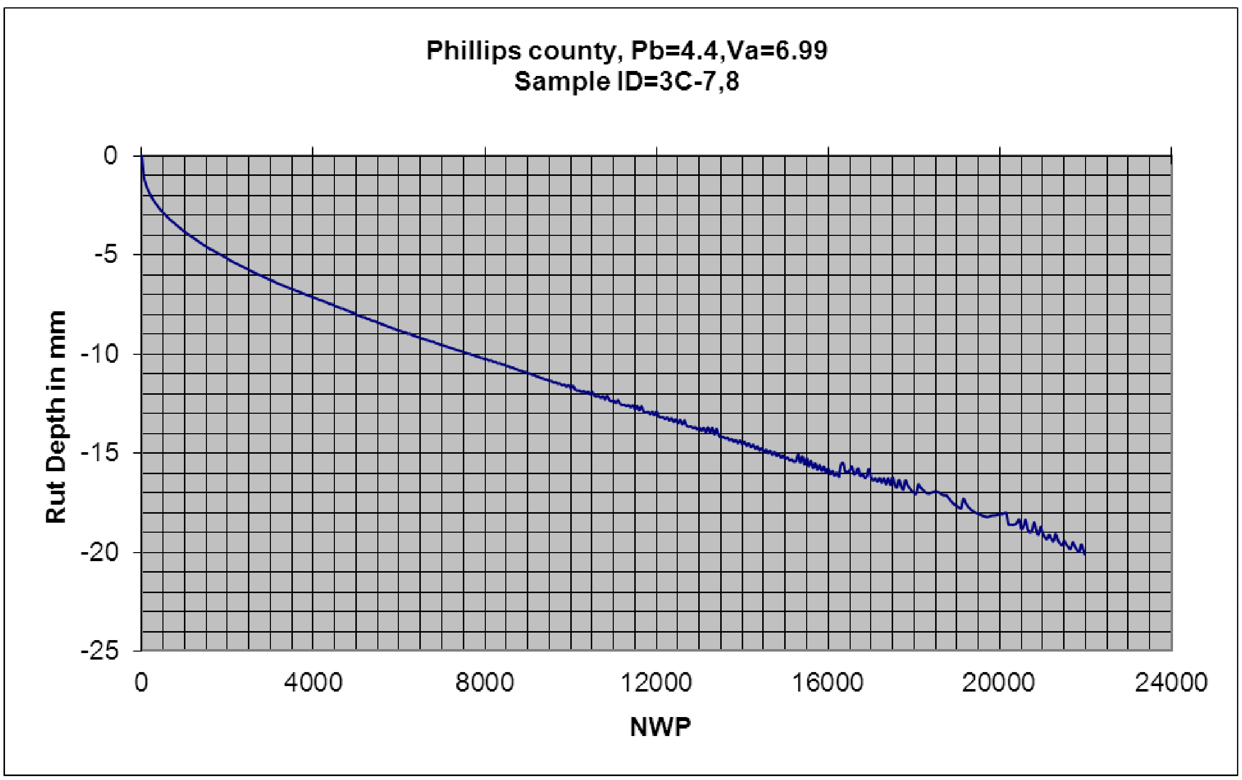
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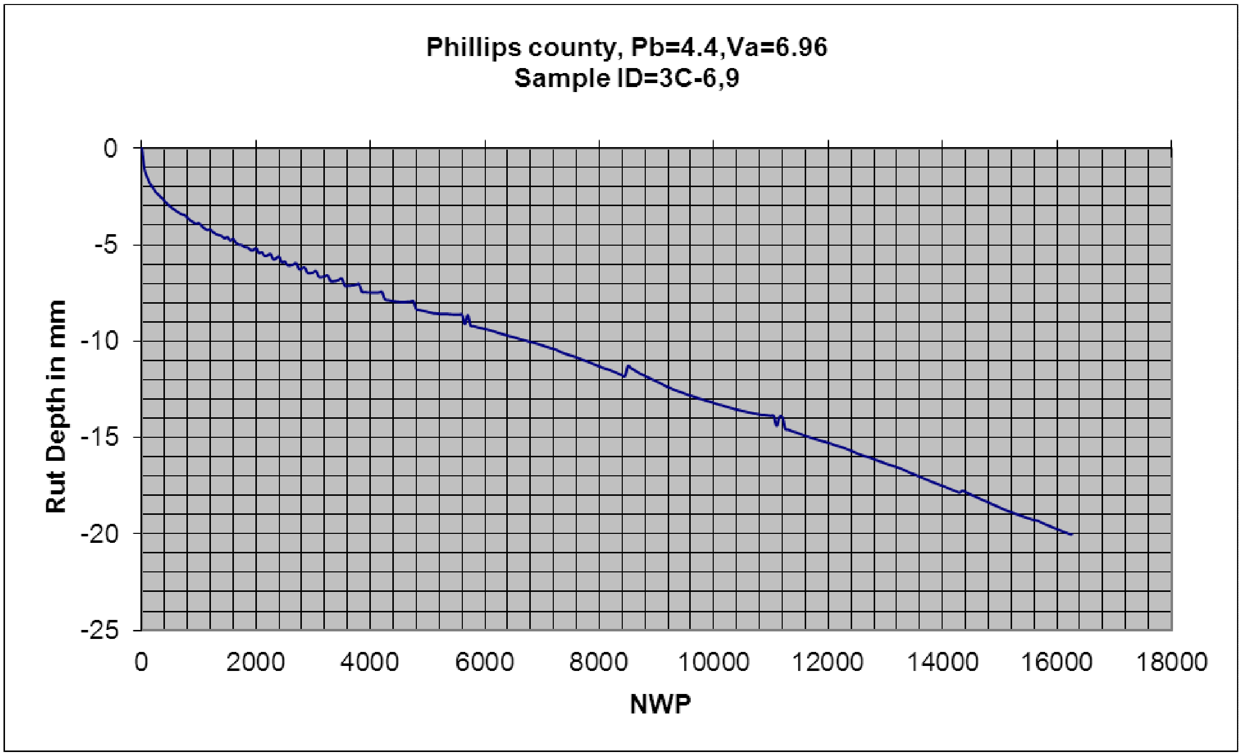
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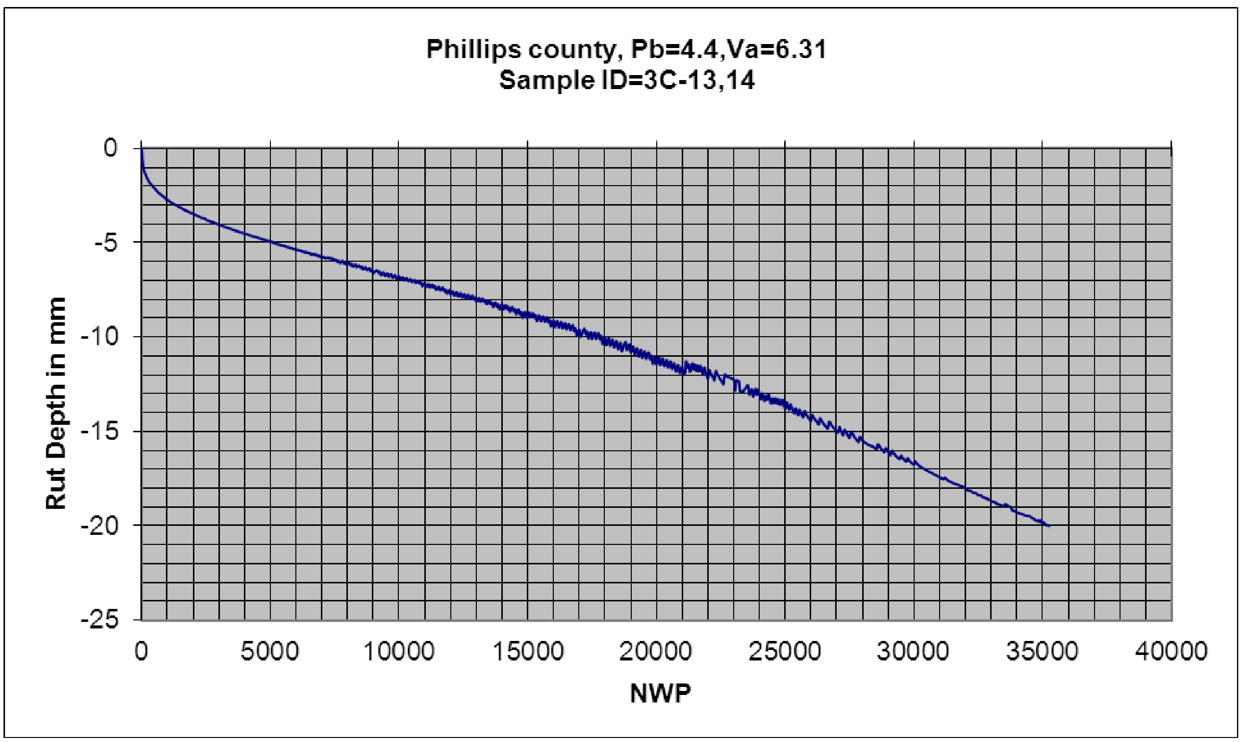
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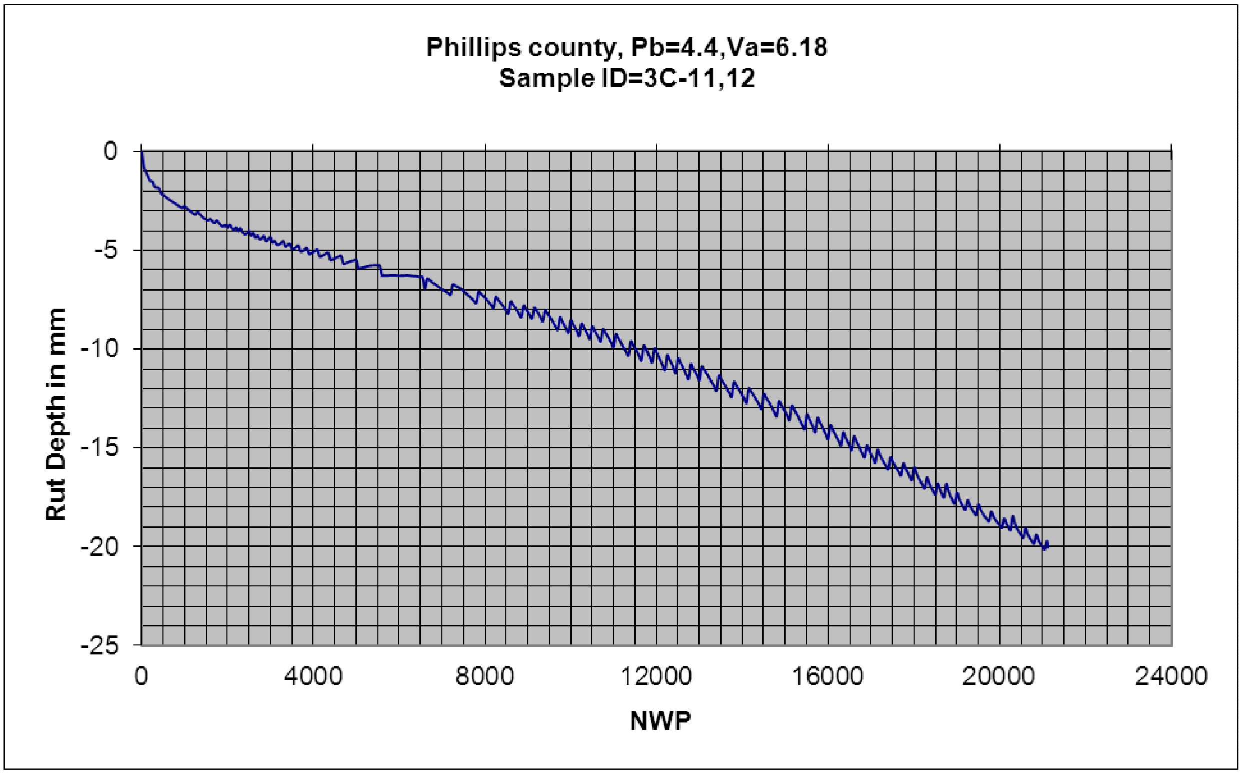
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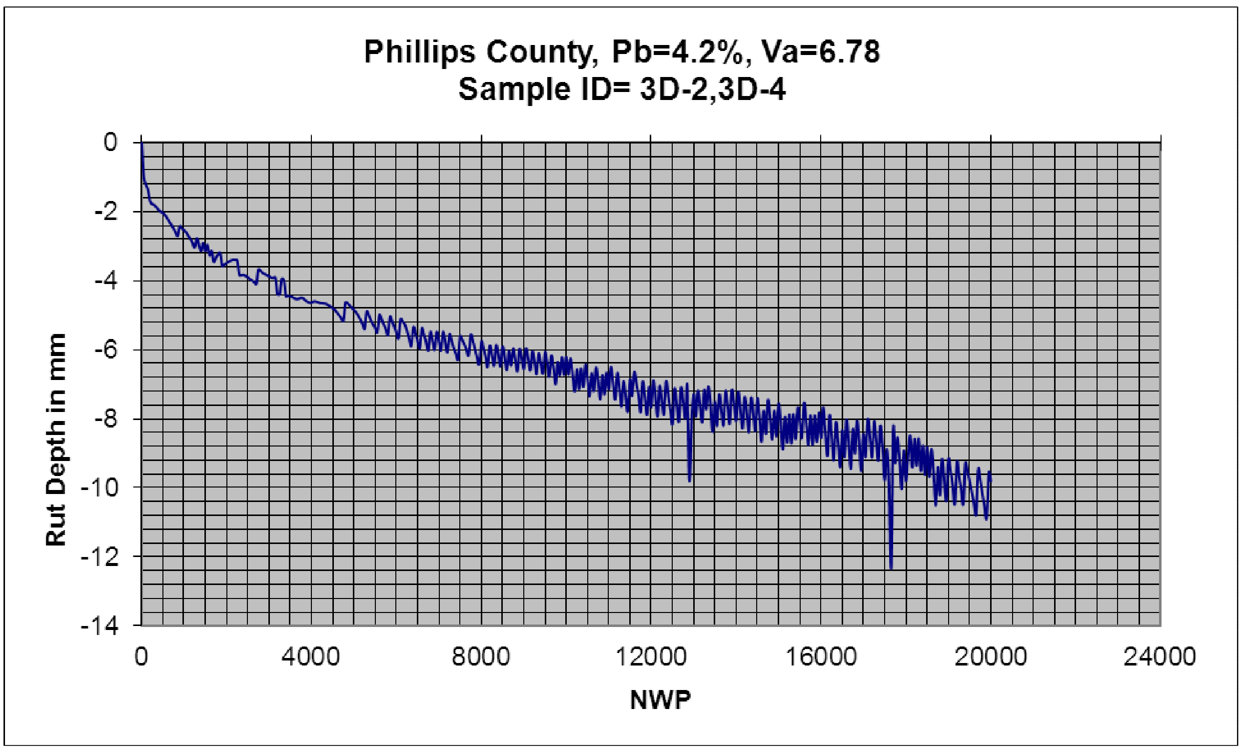
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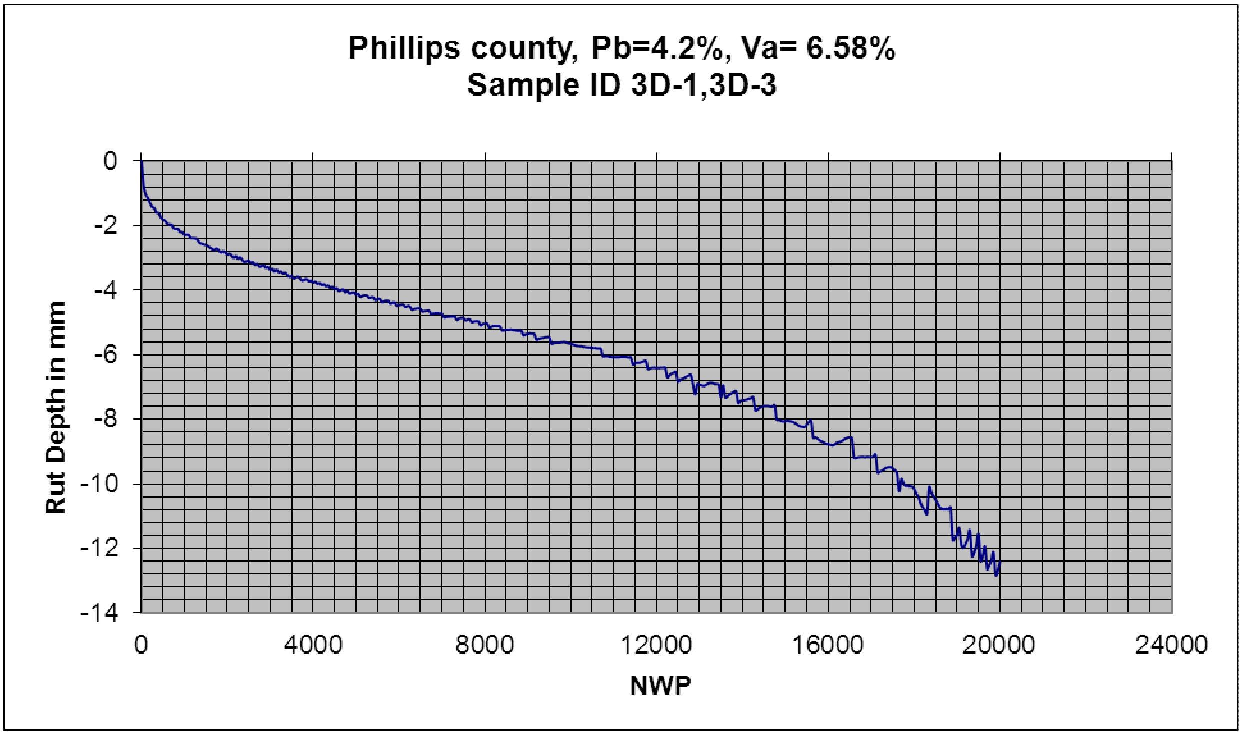
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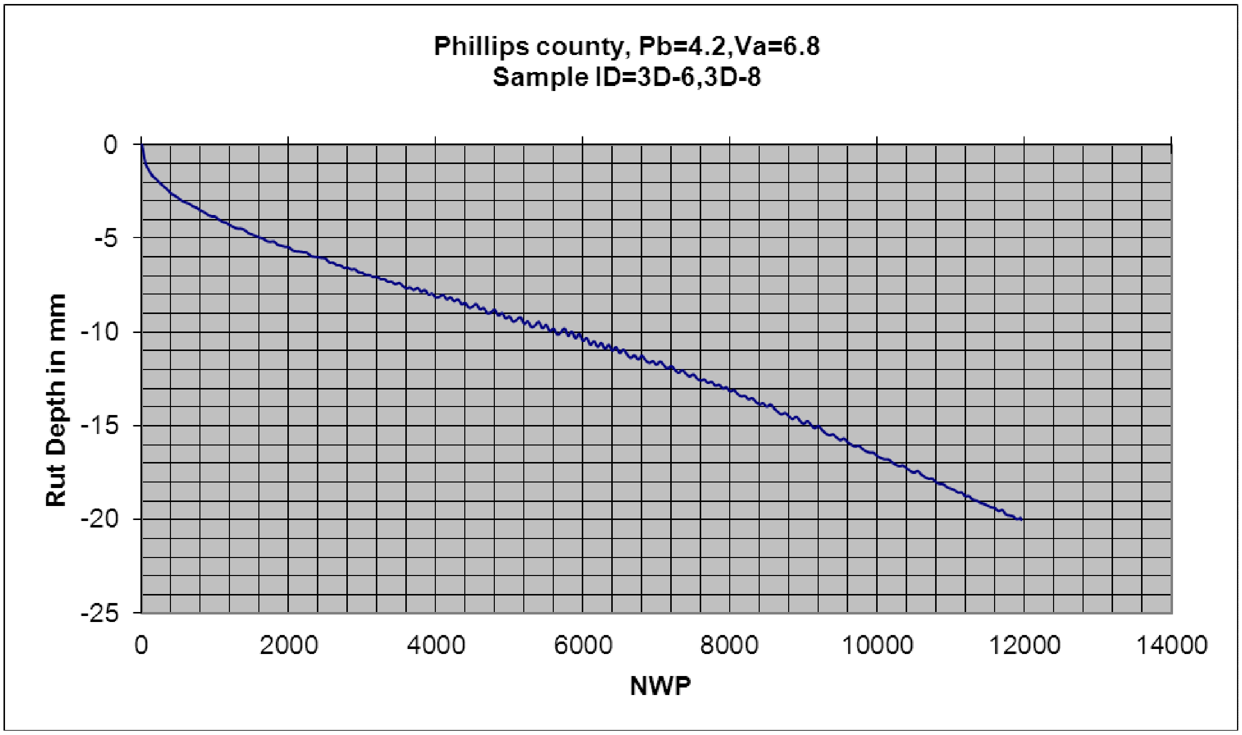
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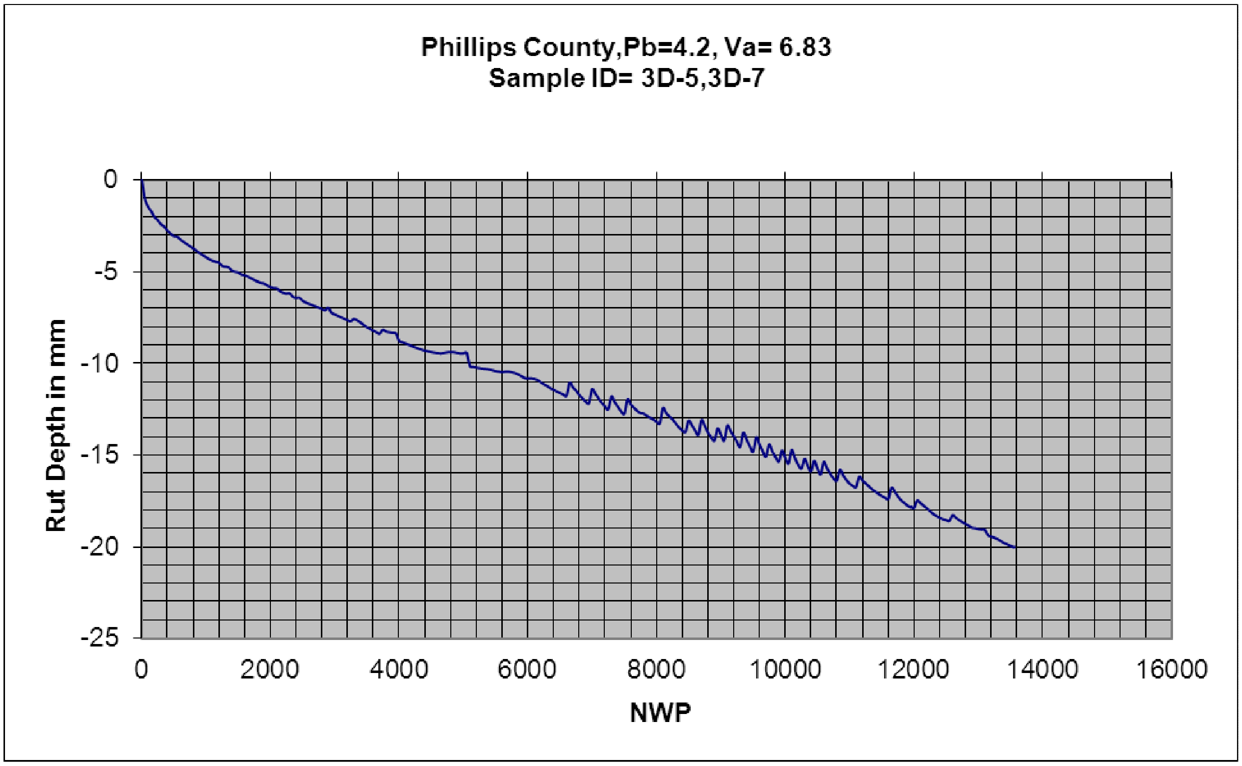
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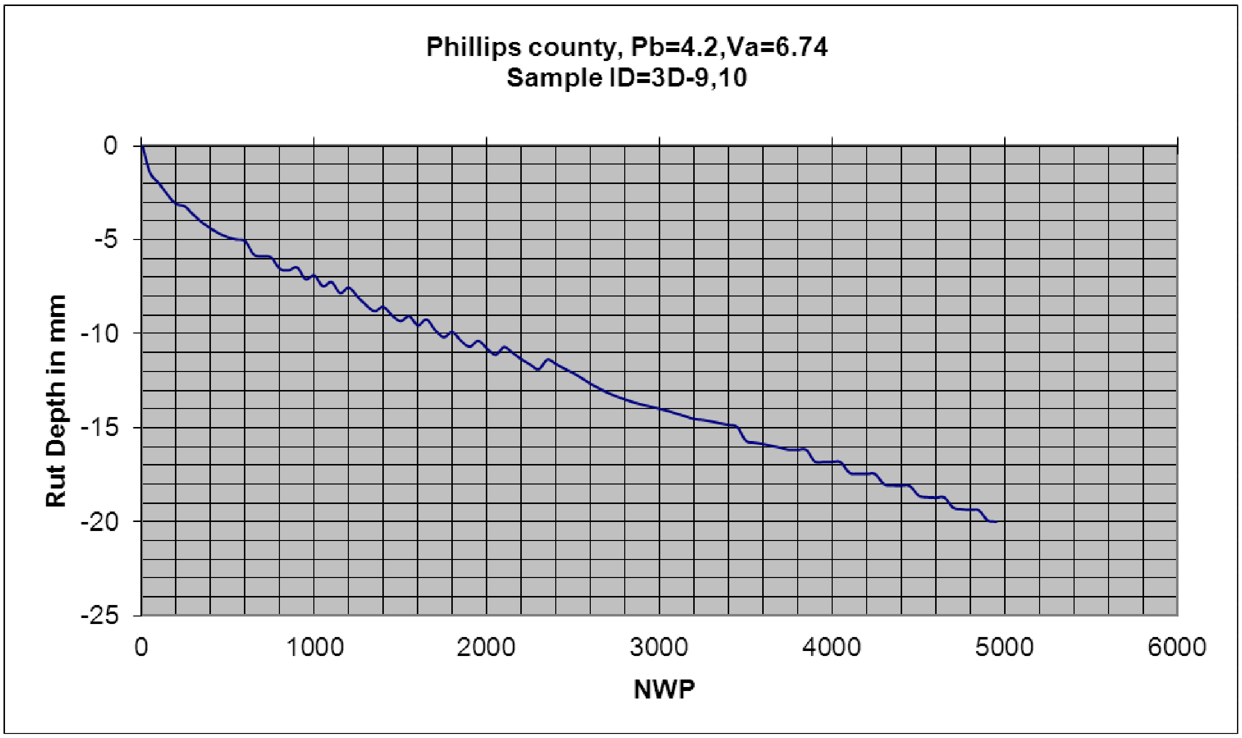
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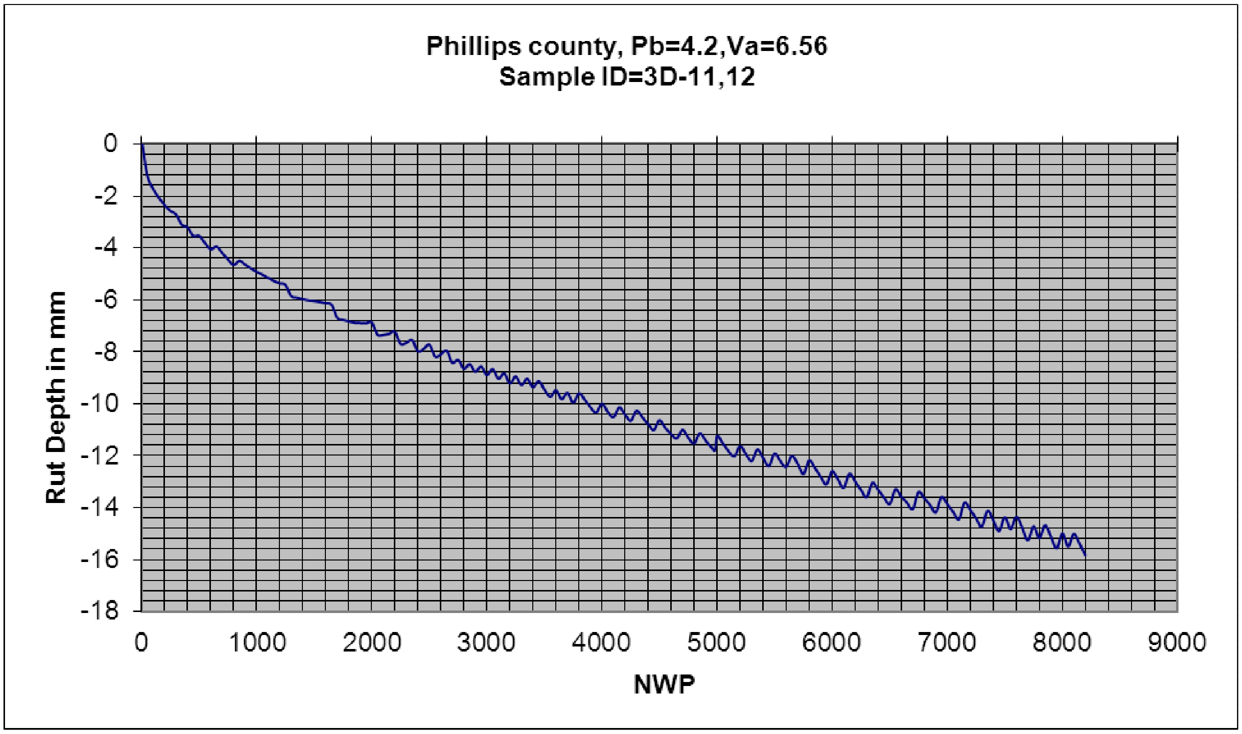

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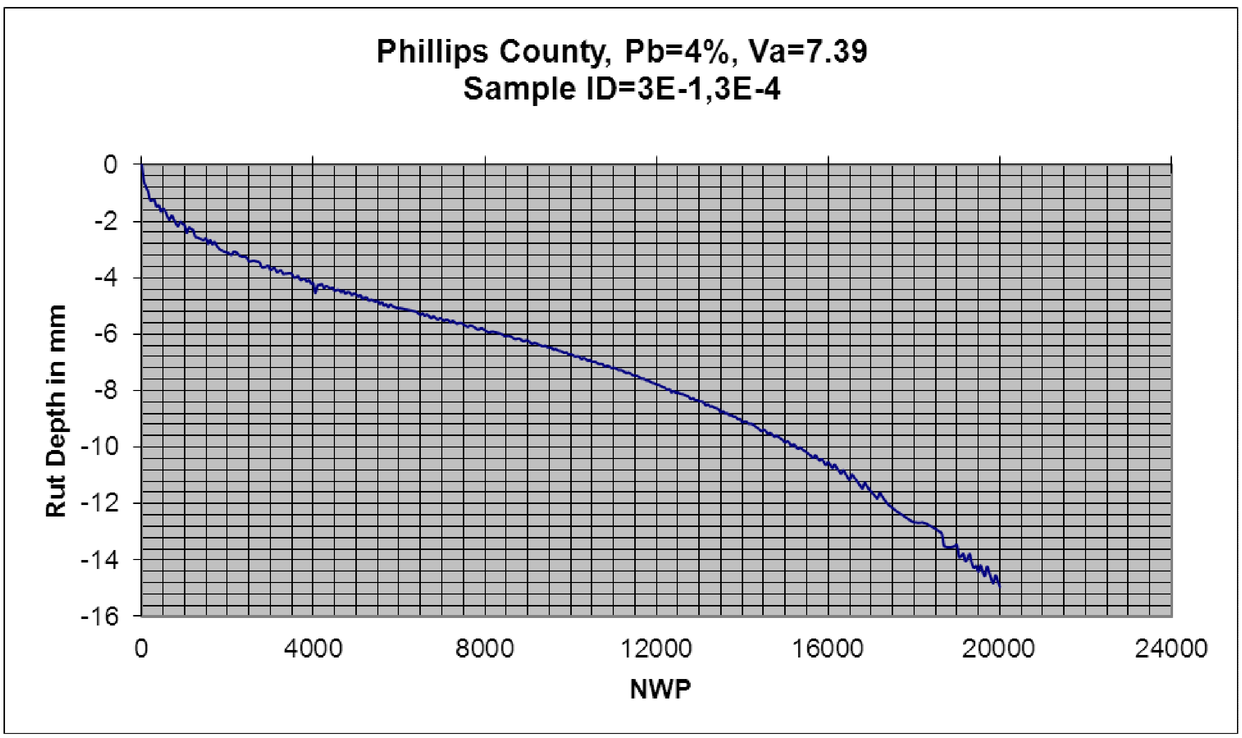

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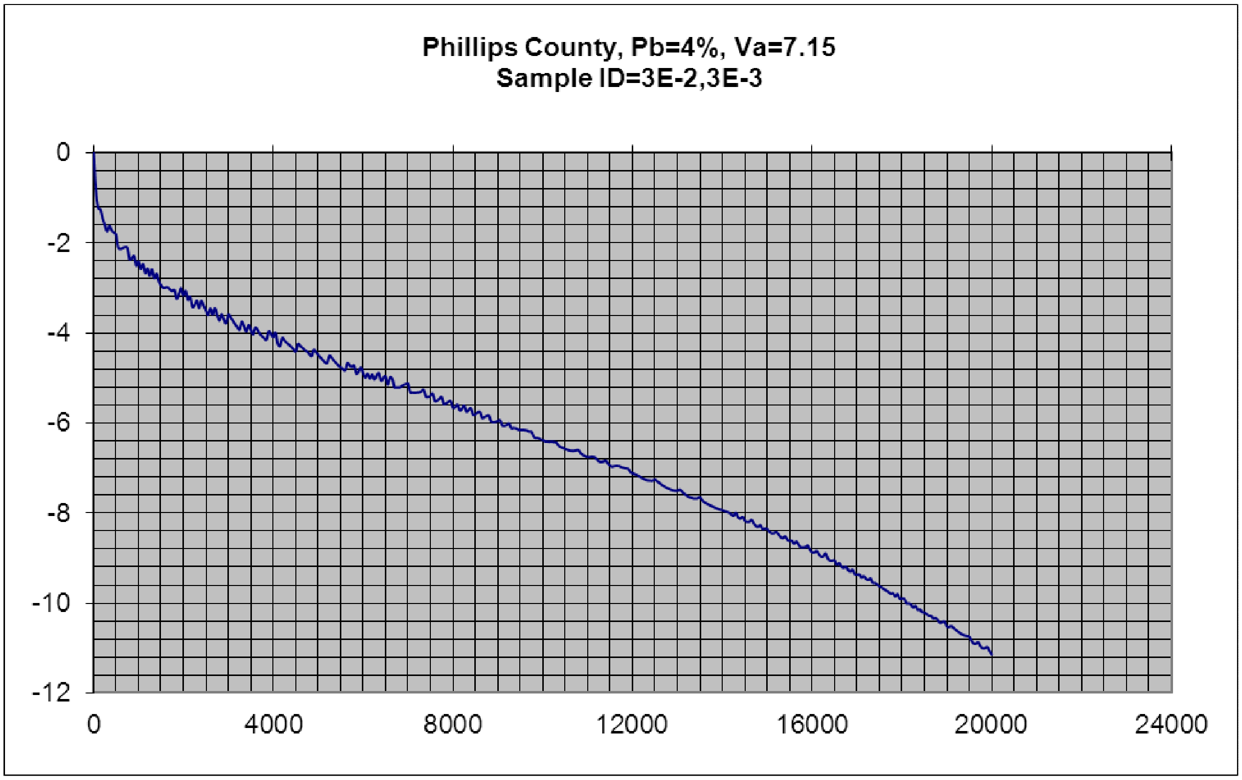

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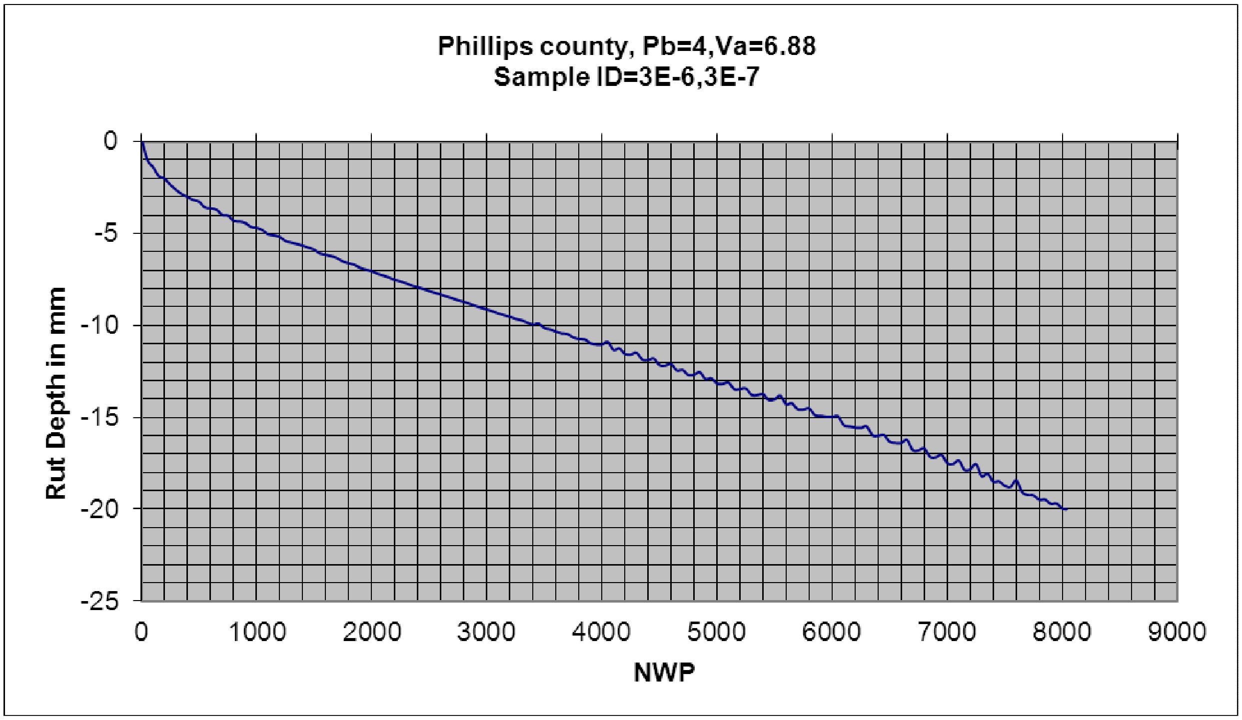

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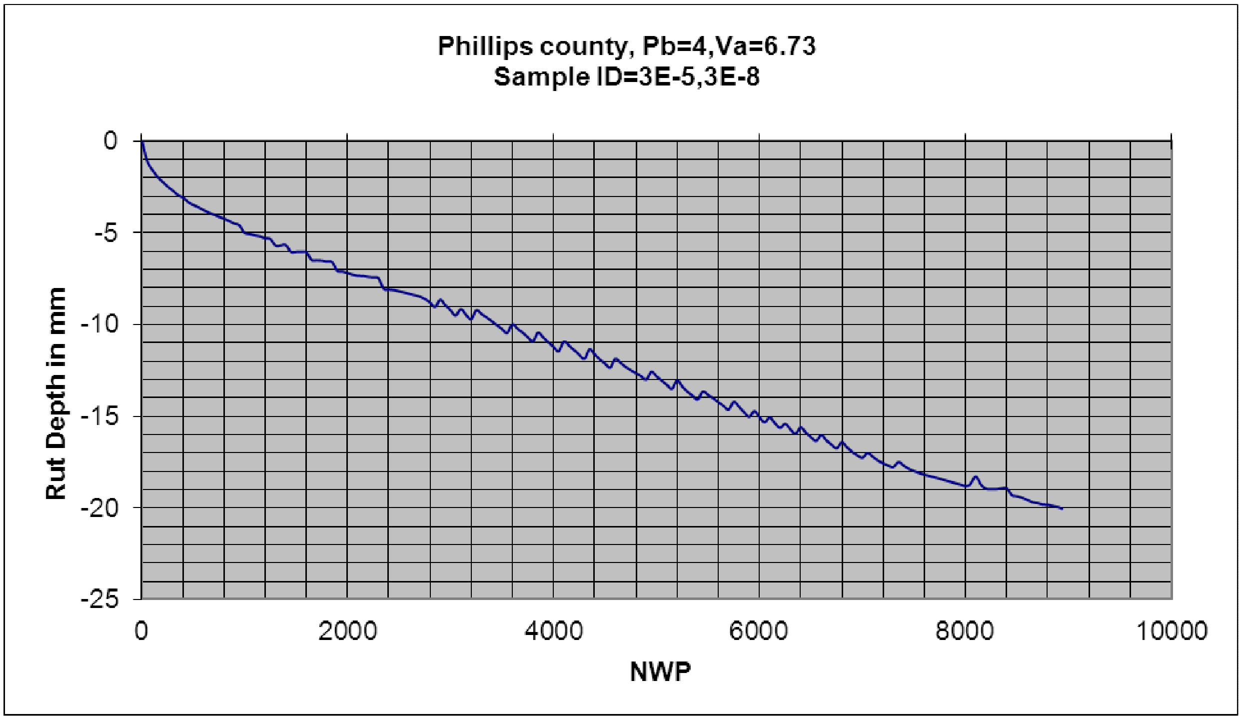
NWP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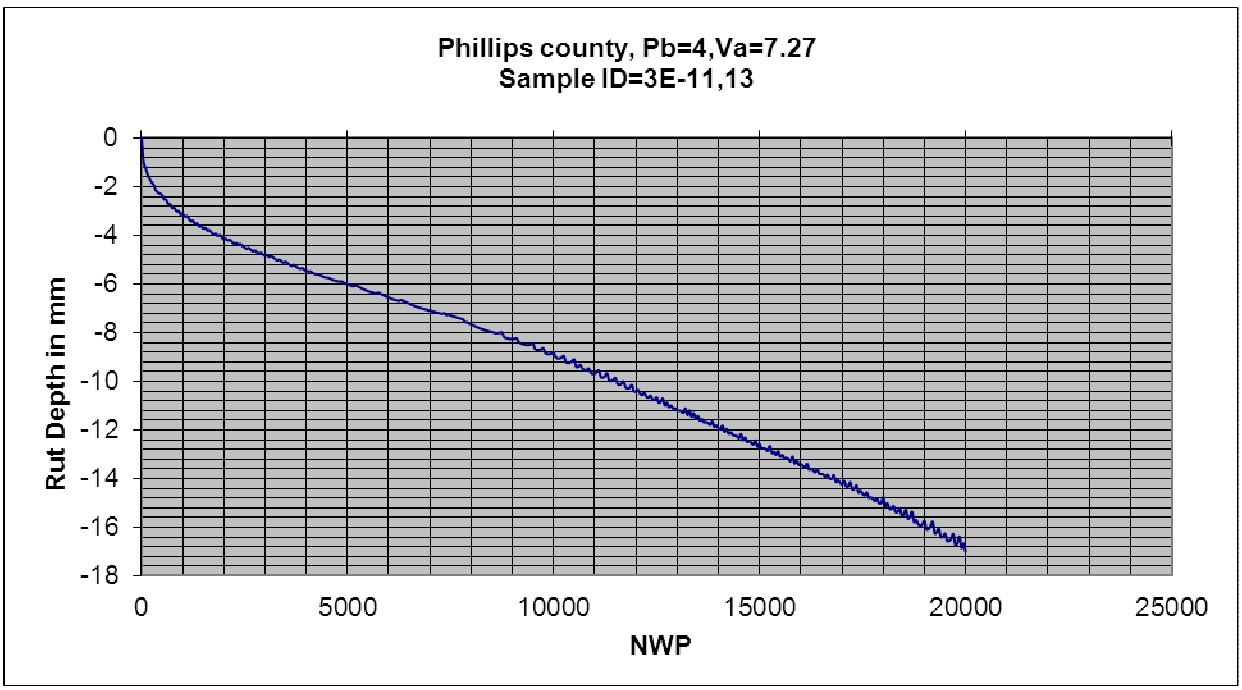
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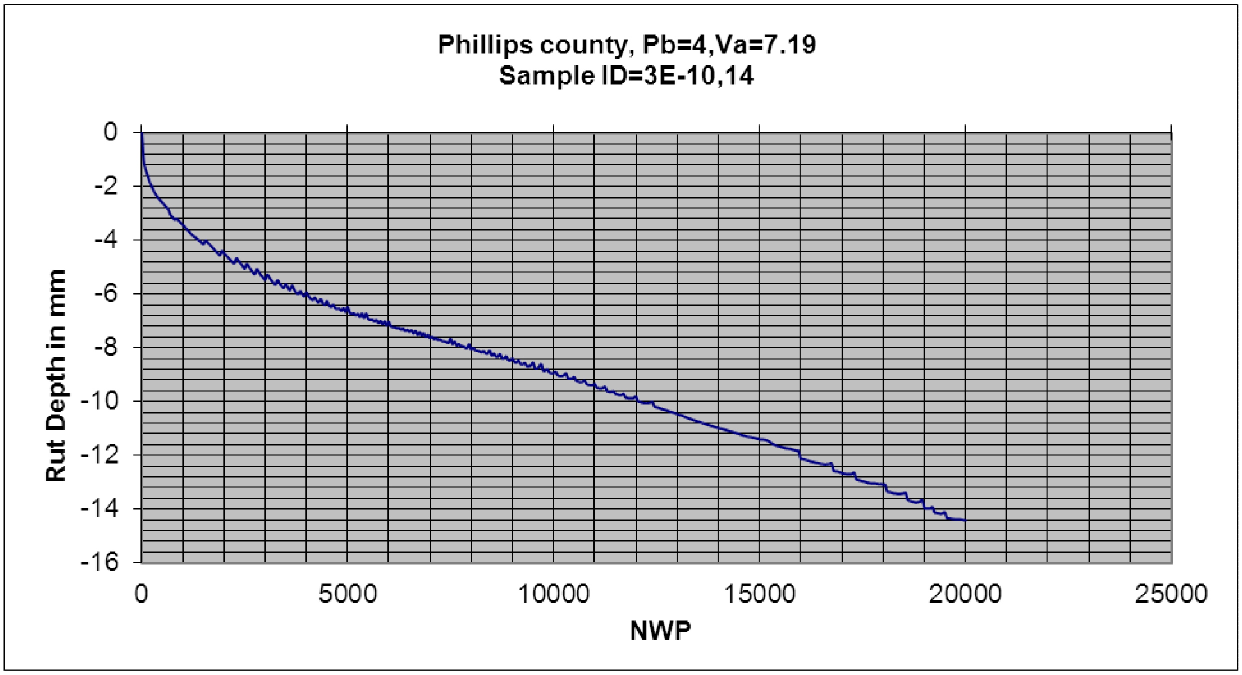
25000
NWP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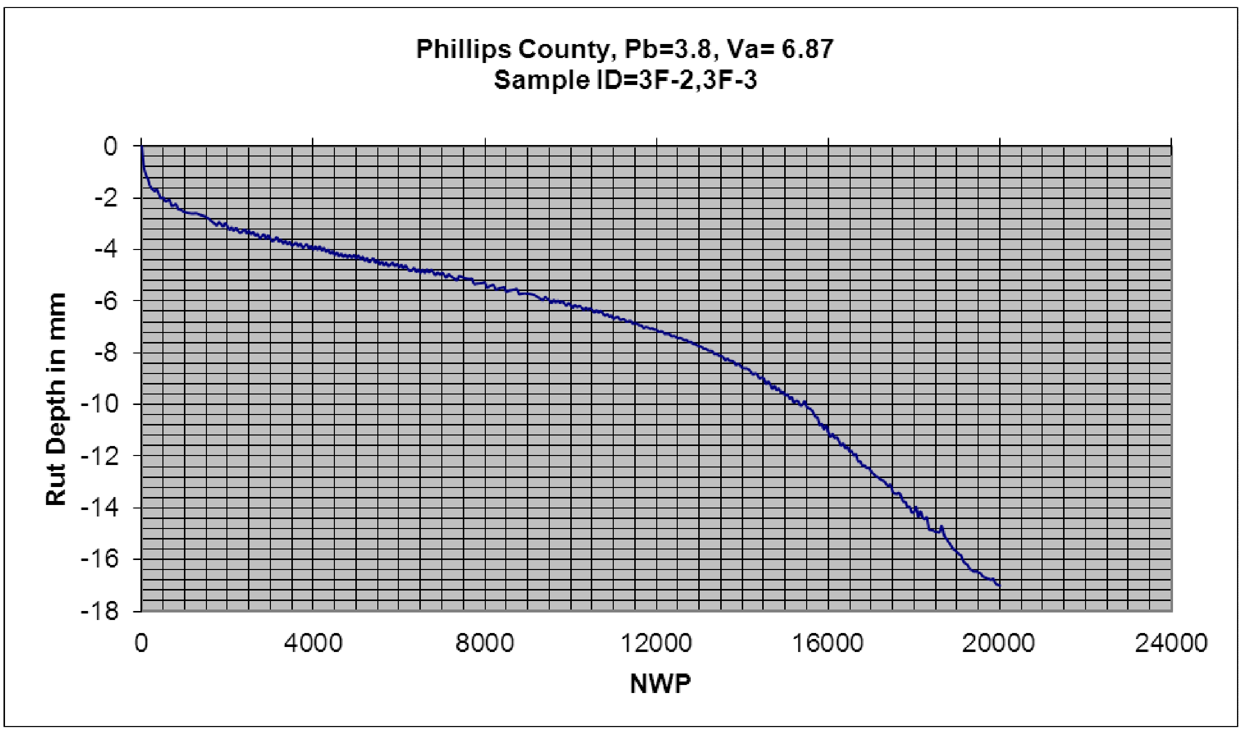
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24000
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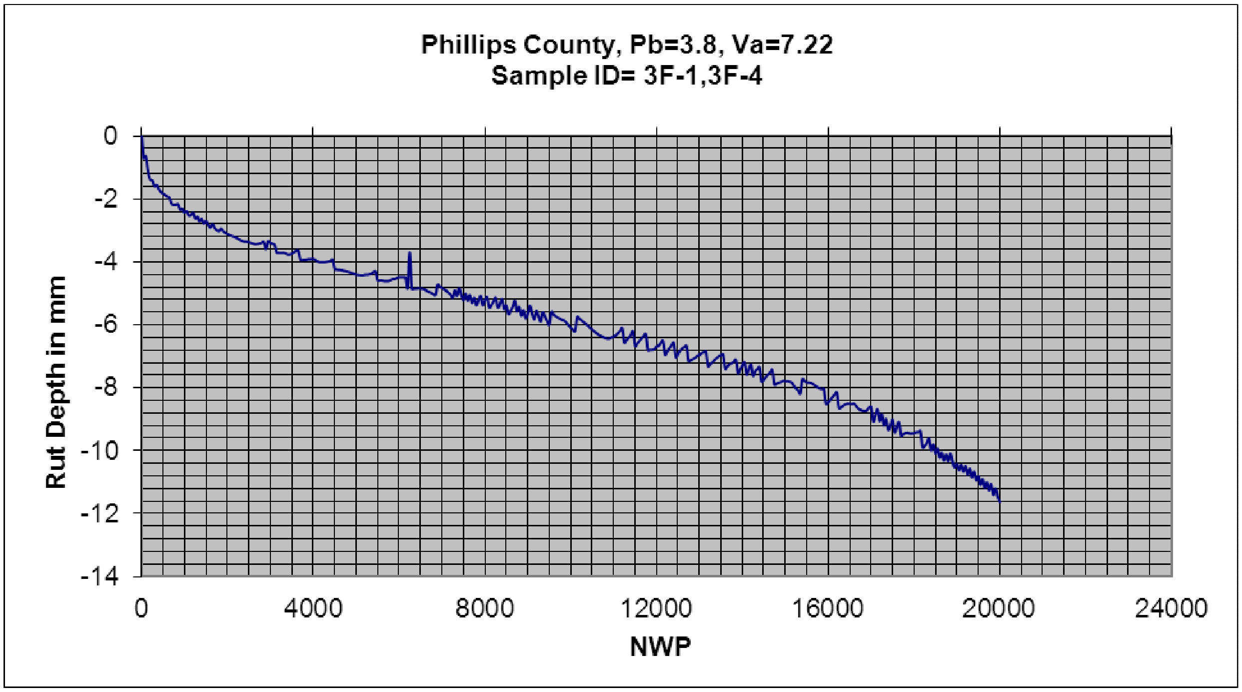
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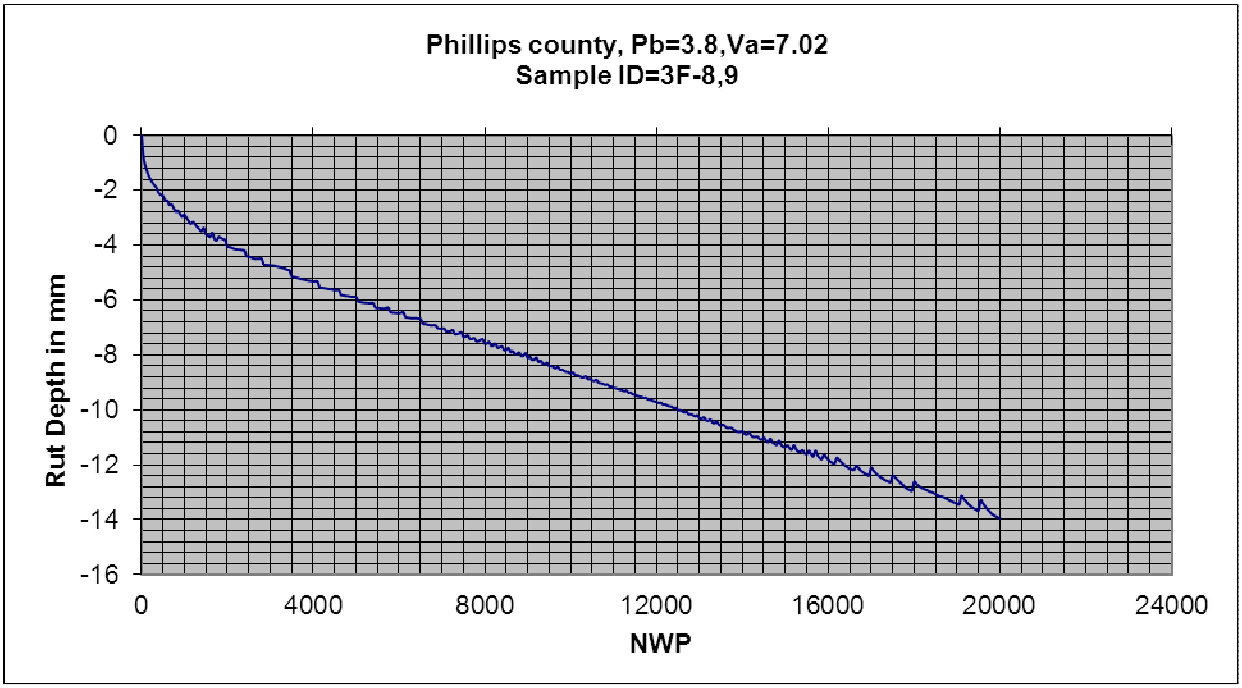
Phillips county, Pb=3.8,Va=7.02
Sample ID=3F-8,9
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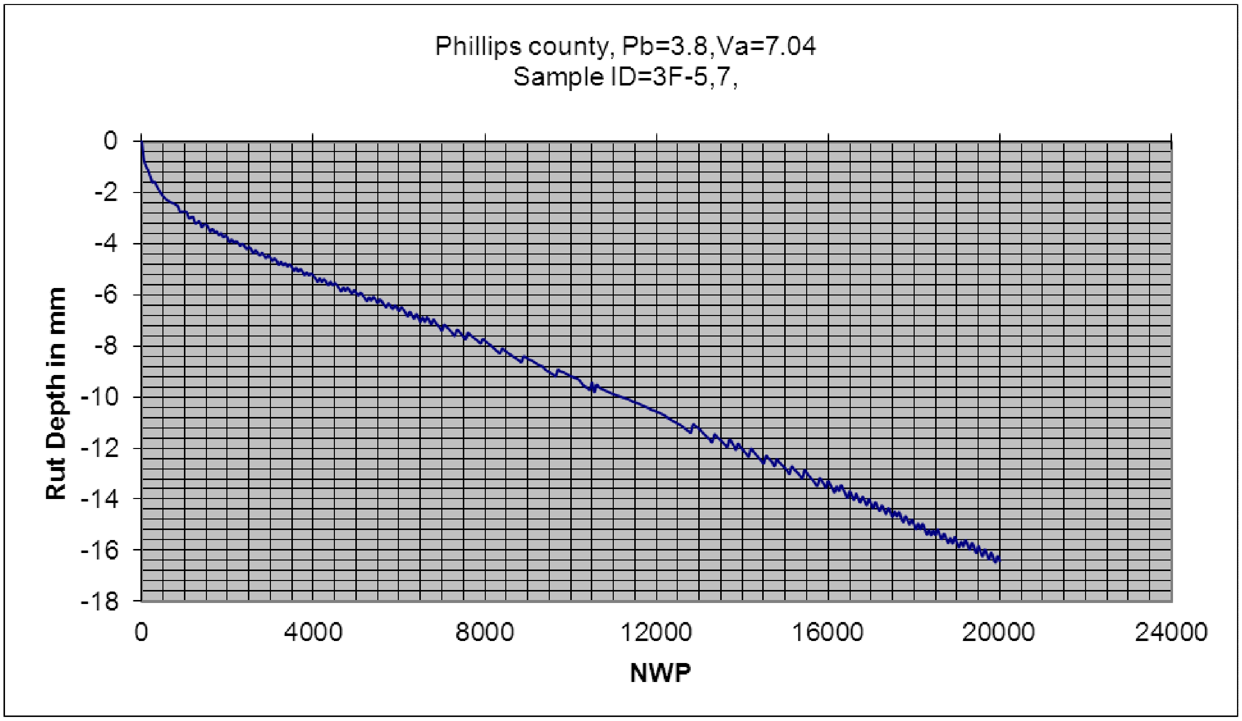
Phillips county, Pb=3.8,Va=7.04
Sample ID=3F-5,7,
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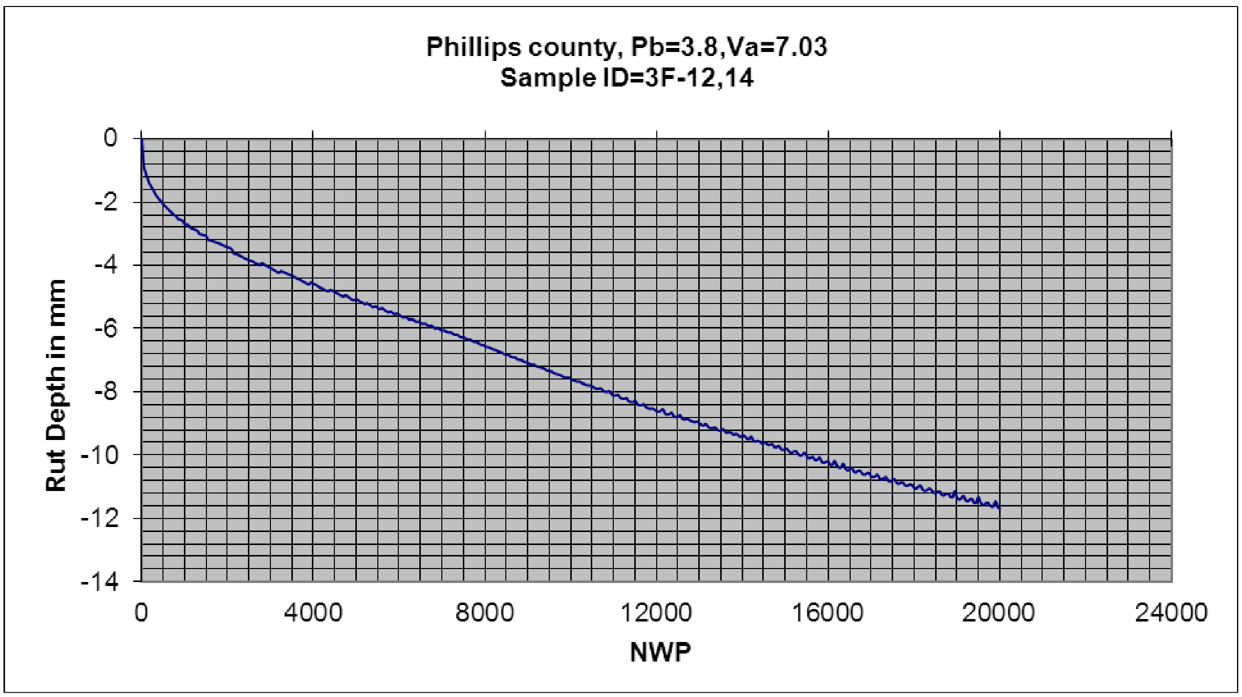
Phillips county, Pb=3.8,Va=7.03
Sample ID=3F-12,14
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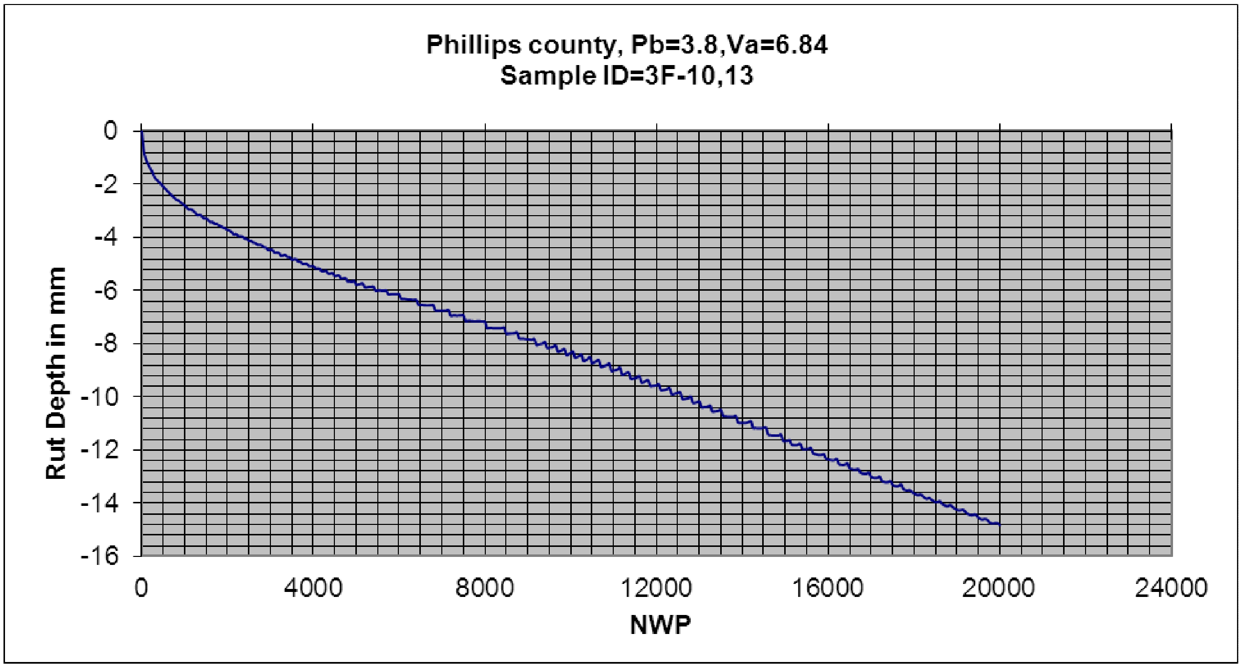
Phillips county, Pb=3.8,Va=6.84
Sample ID=3F-10,13
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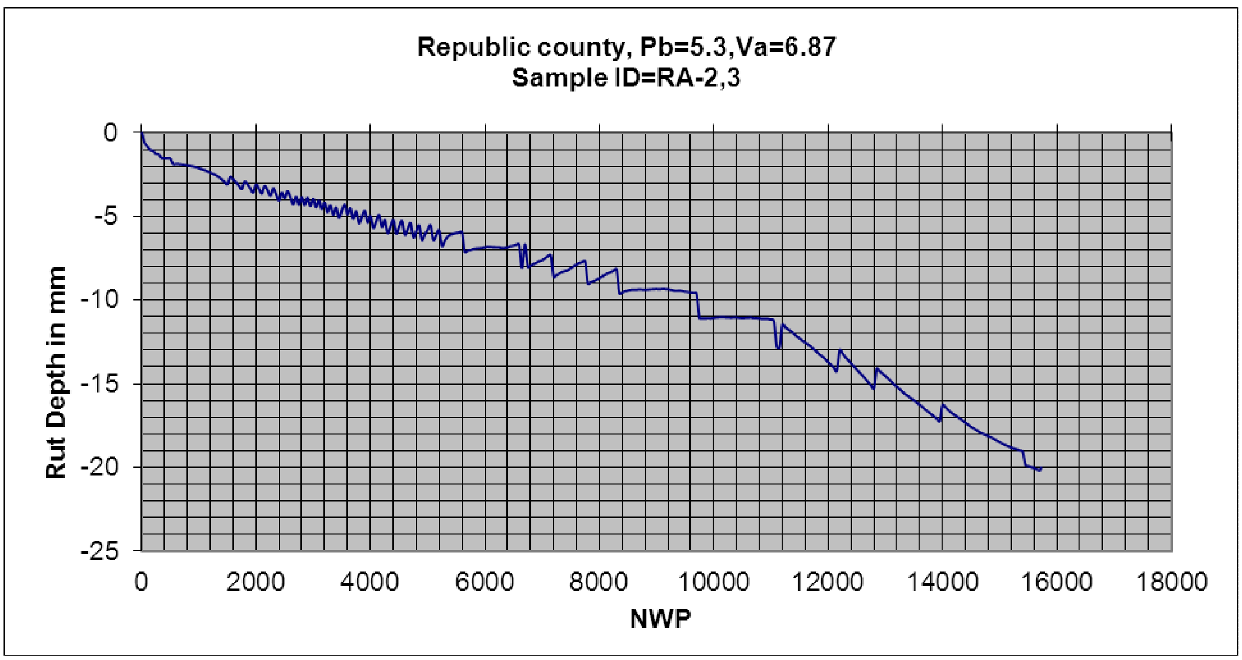
Republic county, Pb=5.3,Va=6.87
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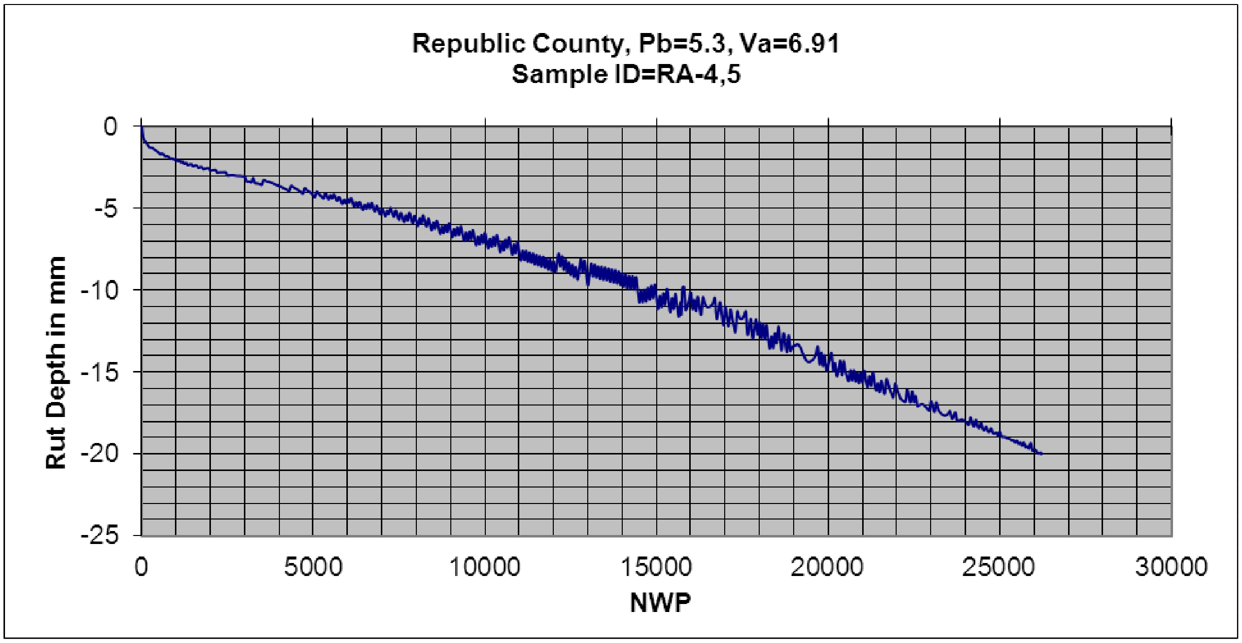
Republic County, Pb=5.3, Va=6.91
Sample ID=RA-4,5
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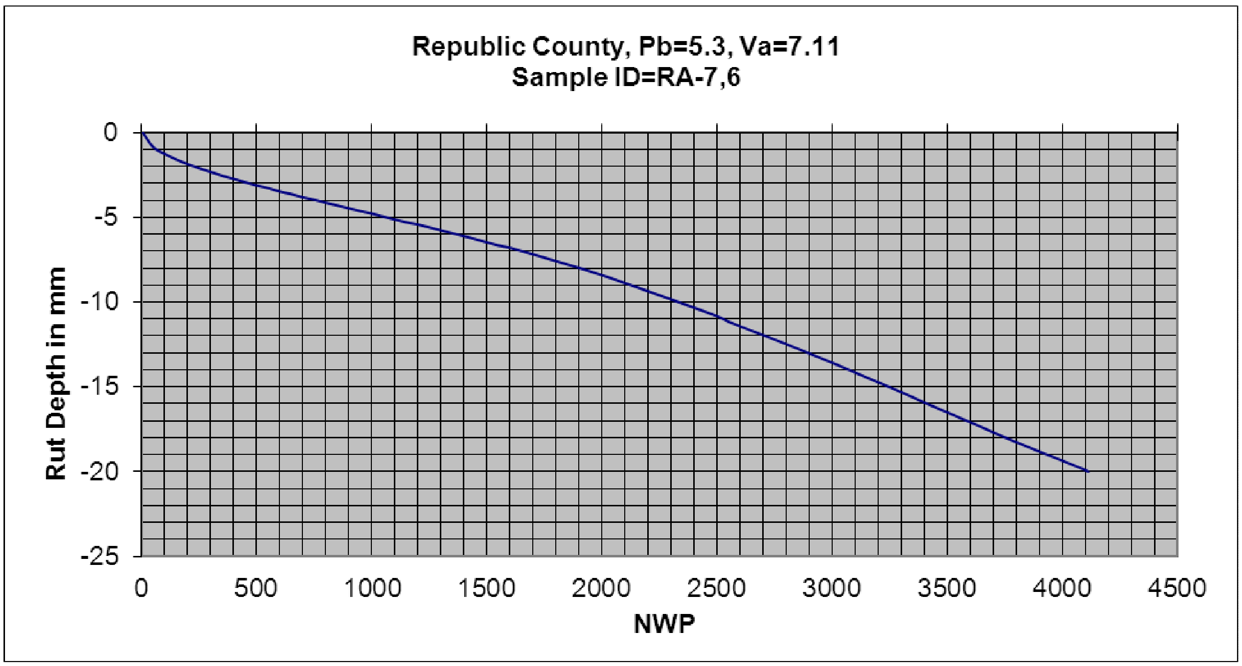
Republic County, Pb=5.3, Va=7.11
Sample ID=RA-7,6
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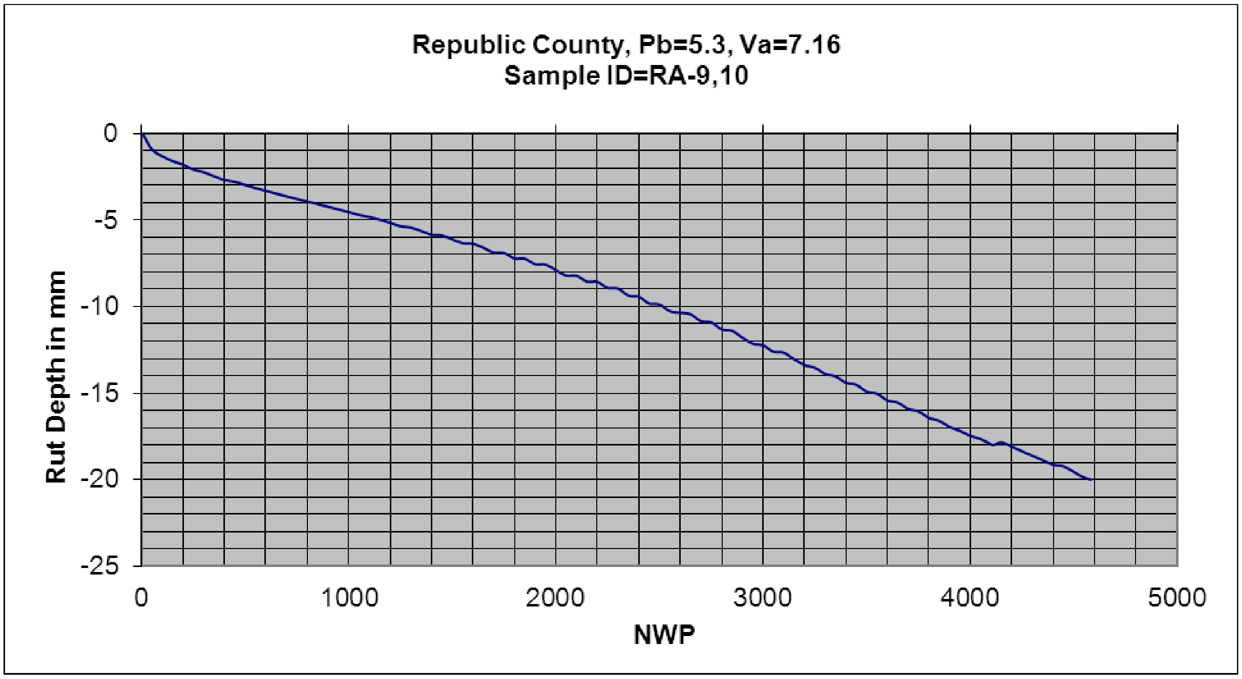
Republic County, Pb=5.3, Va=7.16
Sample ID=RA-9,10
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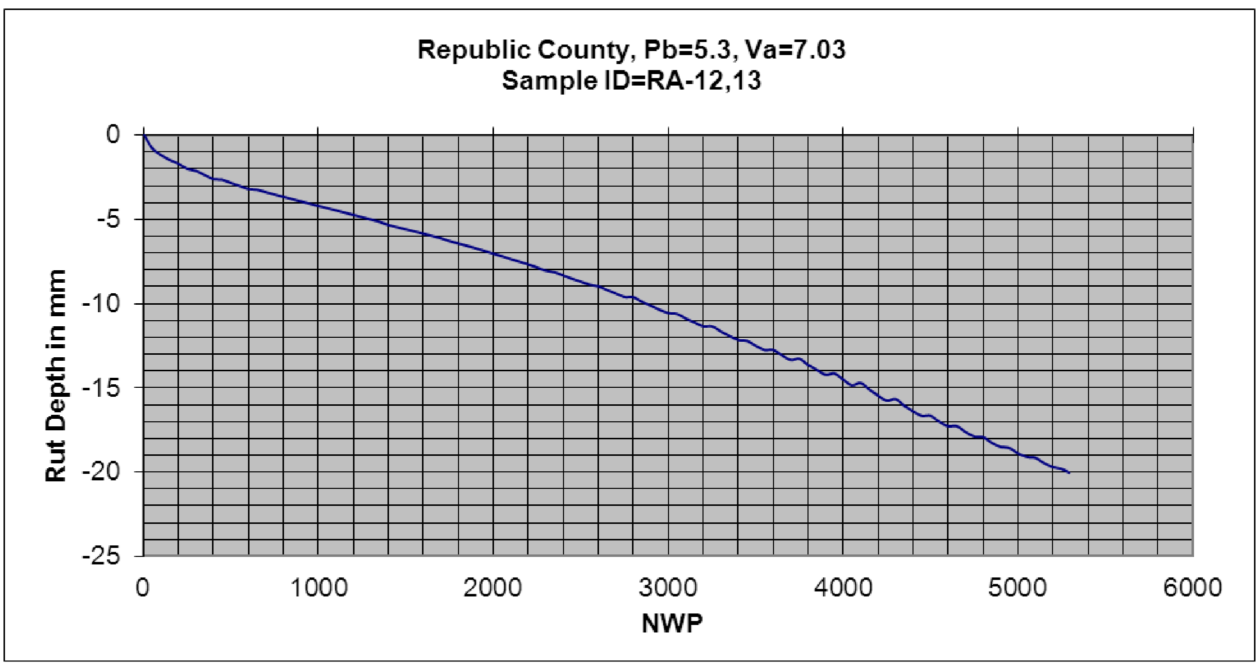
Republic County, Pb=5.3, Va=7.03
Sample ID=RA-12,13
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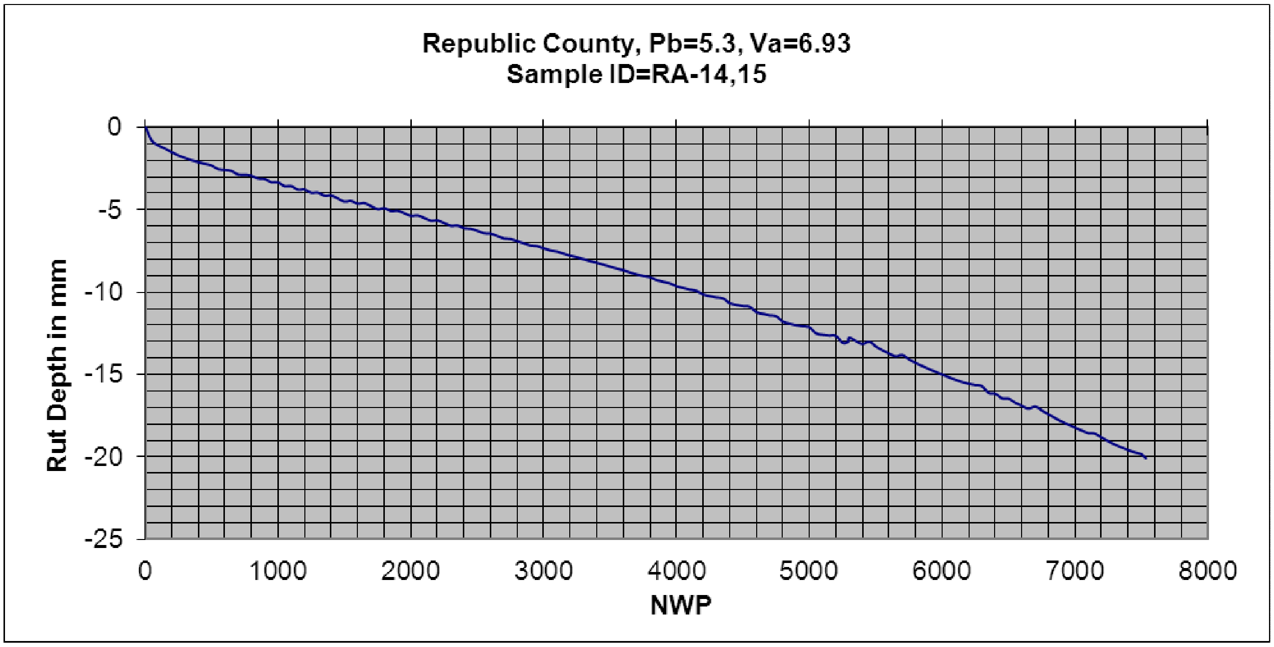
Republic County, Pb=5.3, Va=6.93
Sample ID=RA-14,15
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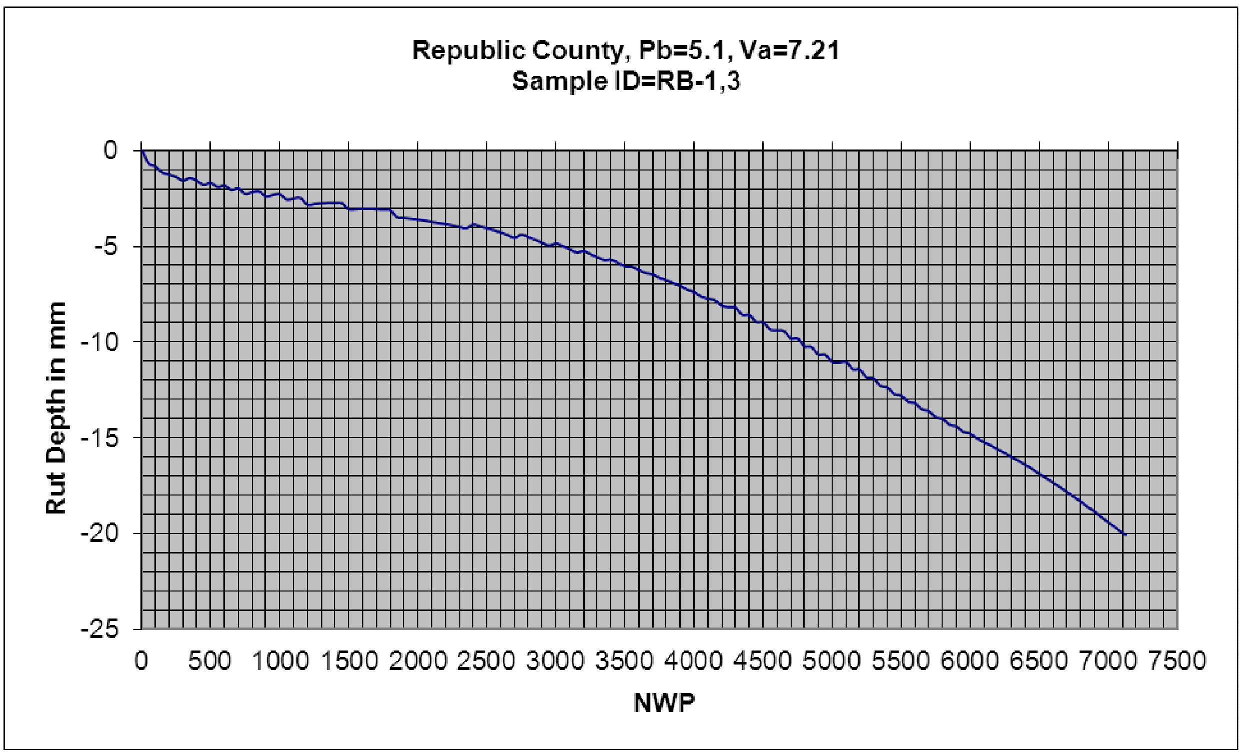
Republic County, Pb=5.1, Va=7.21
Sample ID=RB-1,3
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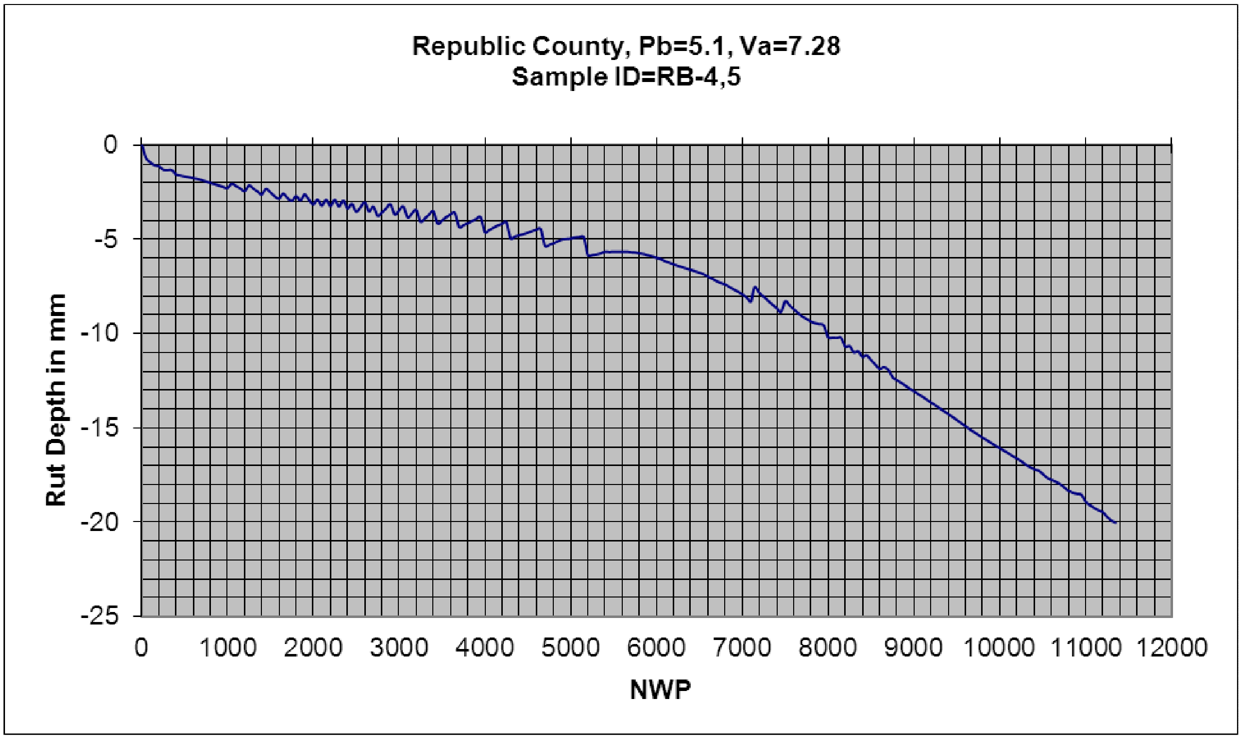
Republic County, Pb=5.1, Va=7.28
Sample ID=RB-4,5
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11000
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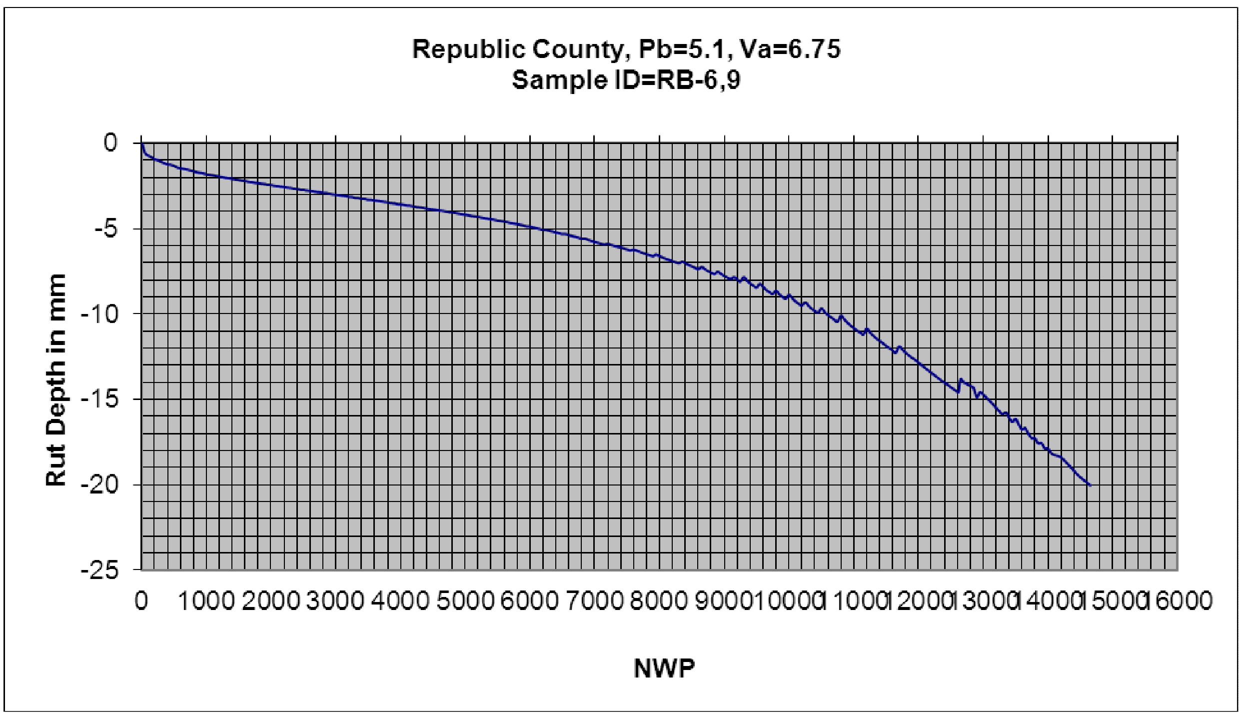
Republic County, Pb=5.1, Va=6.75
Sample ID=RB-6,9
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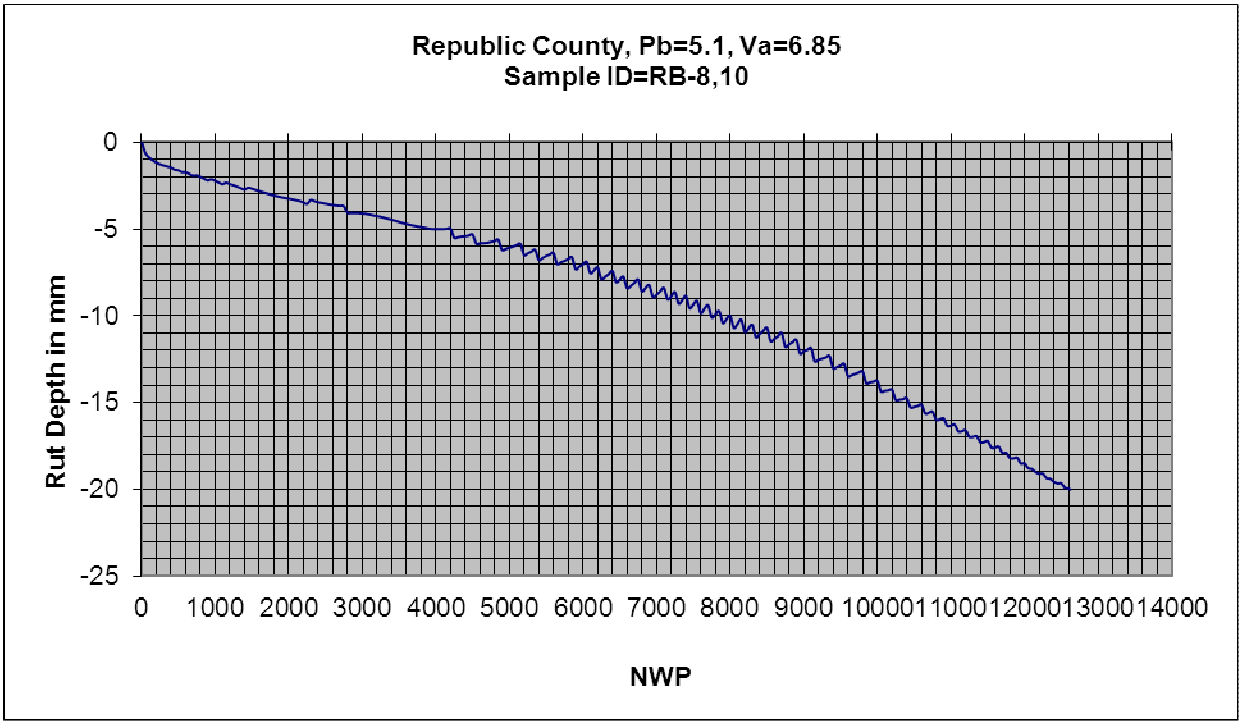

Republic County, Pb=5.1, Va=6.85
Sample ID=RB-8,10
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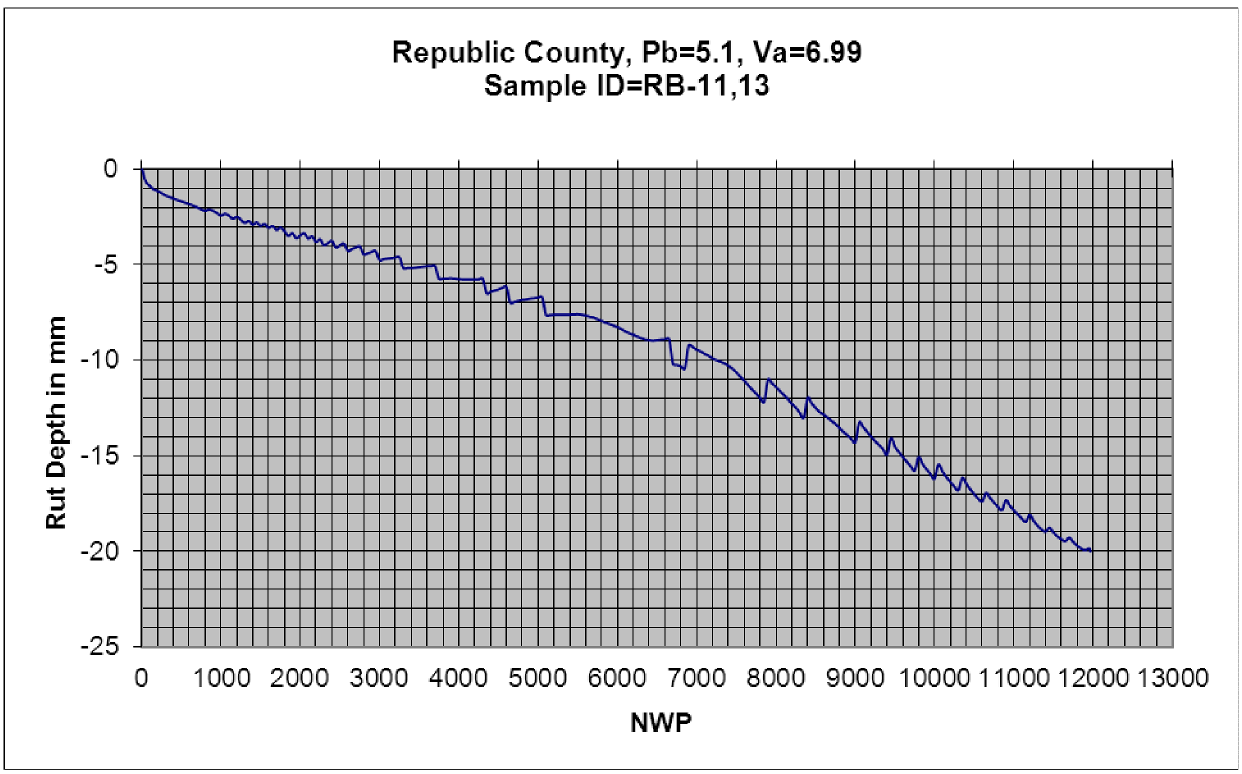

Republic County, Pb=5.1, Va=6.99
Sample ID=RB-11,13
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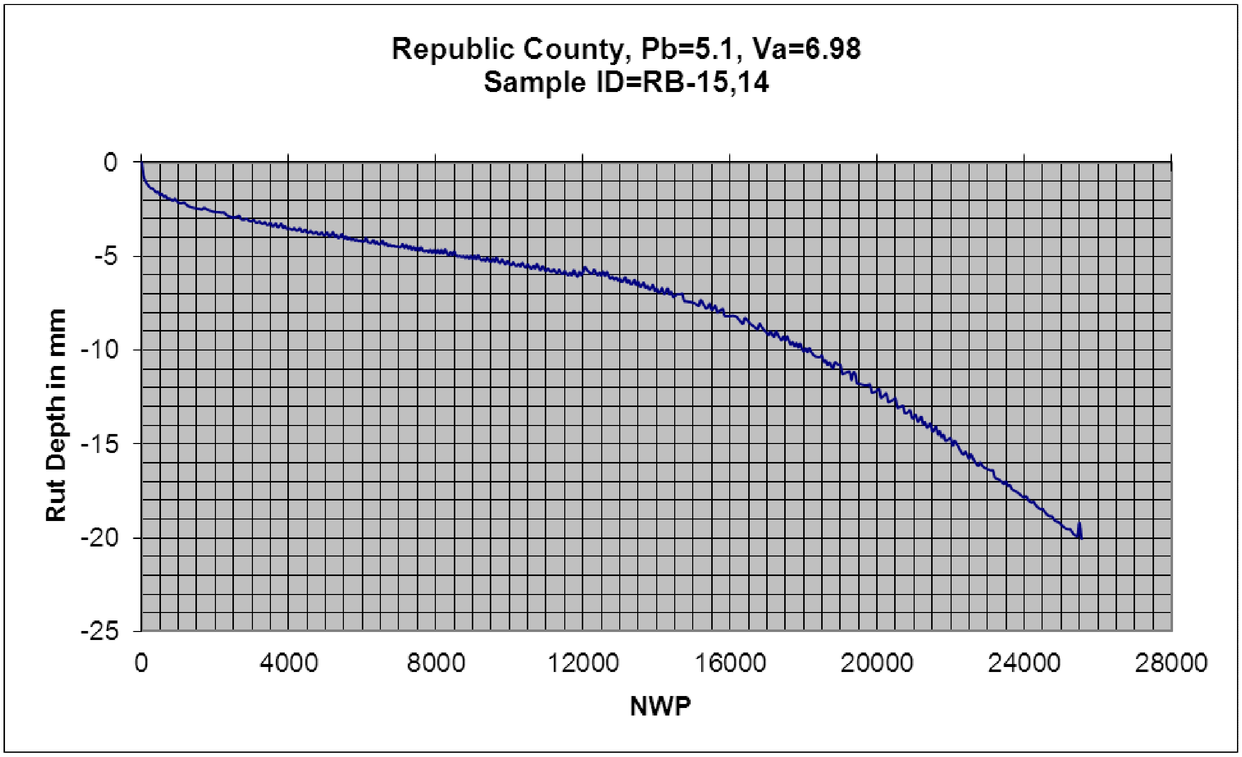
Republic County, Pb=5.1, Va=6.98
Sample ID=RB-15,14
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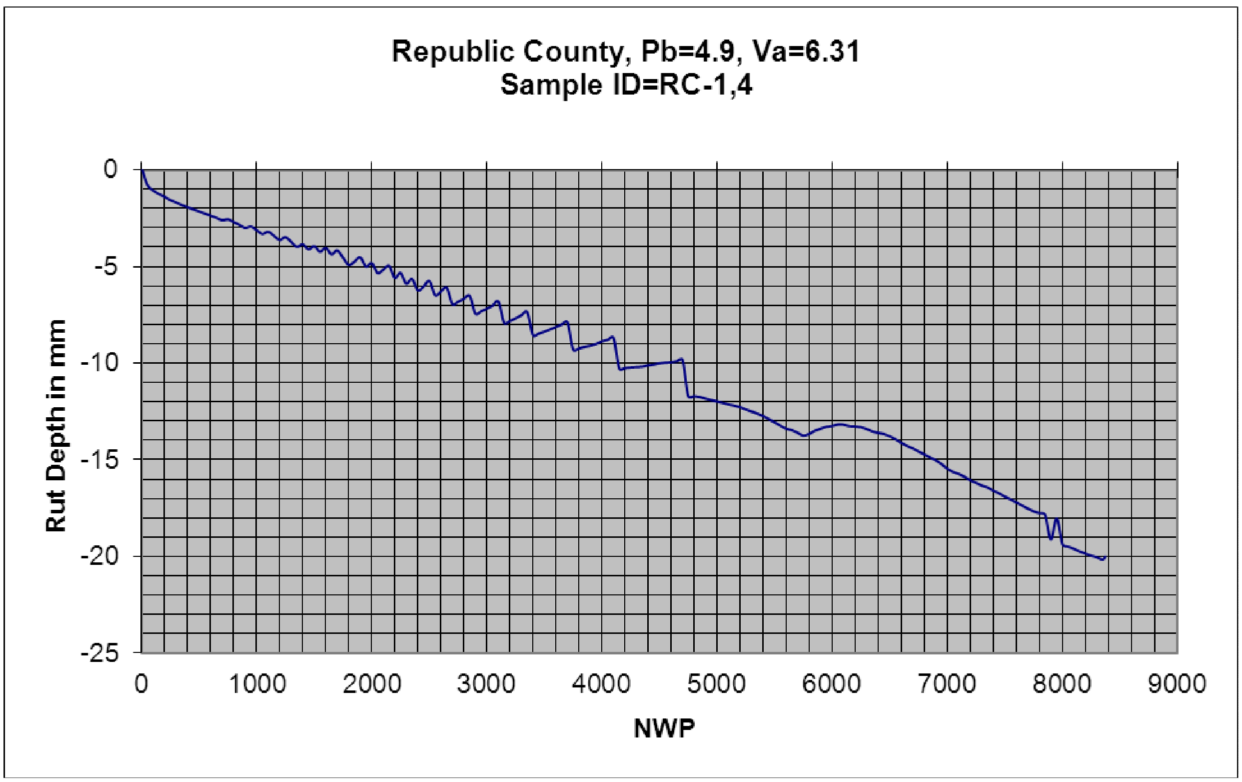
Republic County, Pb=4.9, Va=6.31
Sample ID=RC-1,4
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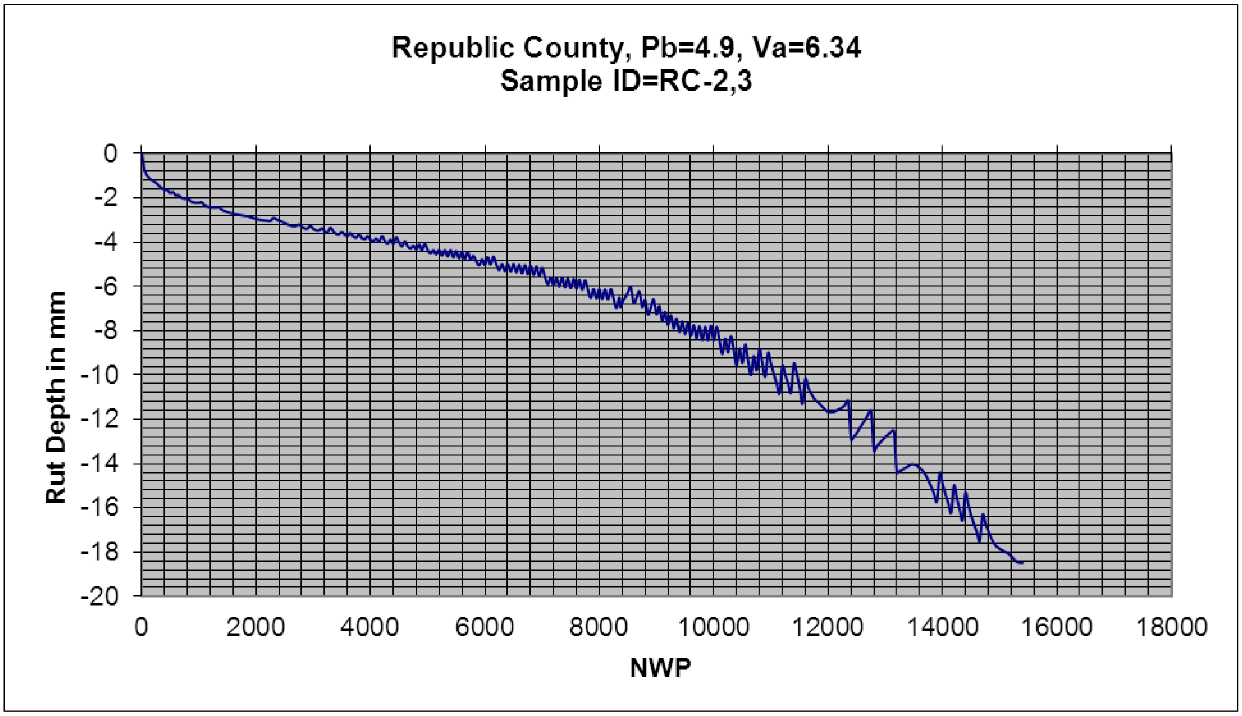
Republic County, Pb=4.9, Va=6.34
Sample ID=RC-2,3
Rut Depth in mm
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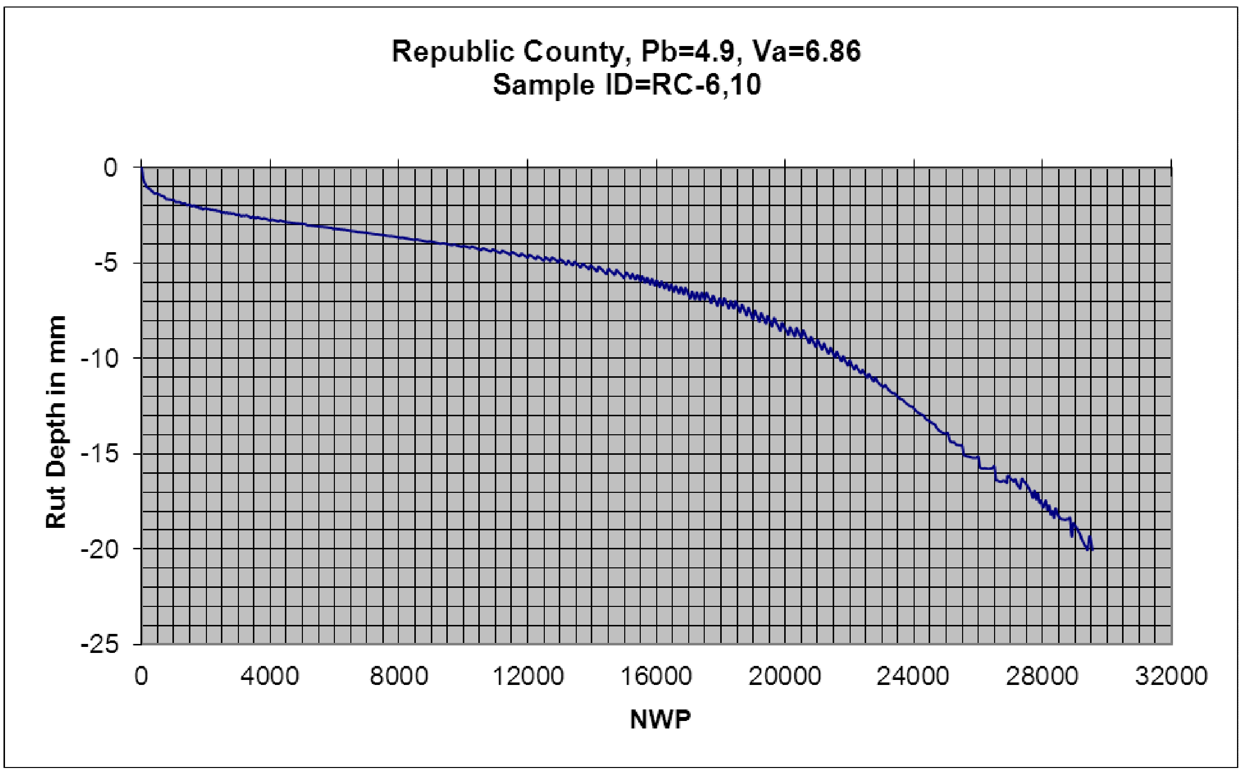
Republic County, Pb=4.9, Va=6.86
Sample ID=RC-6,10
Rut Depth in mm
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24000
28000
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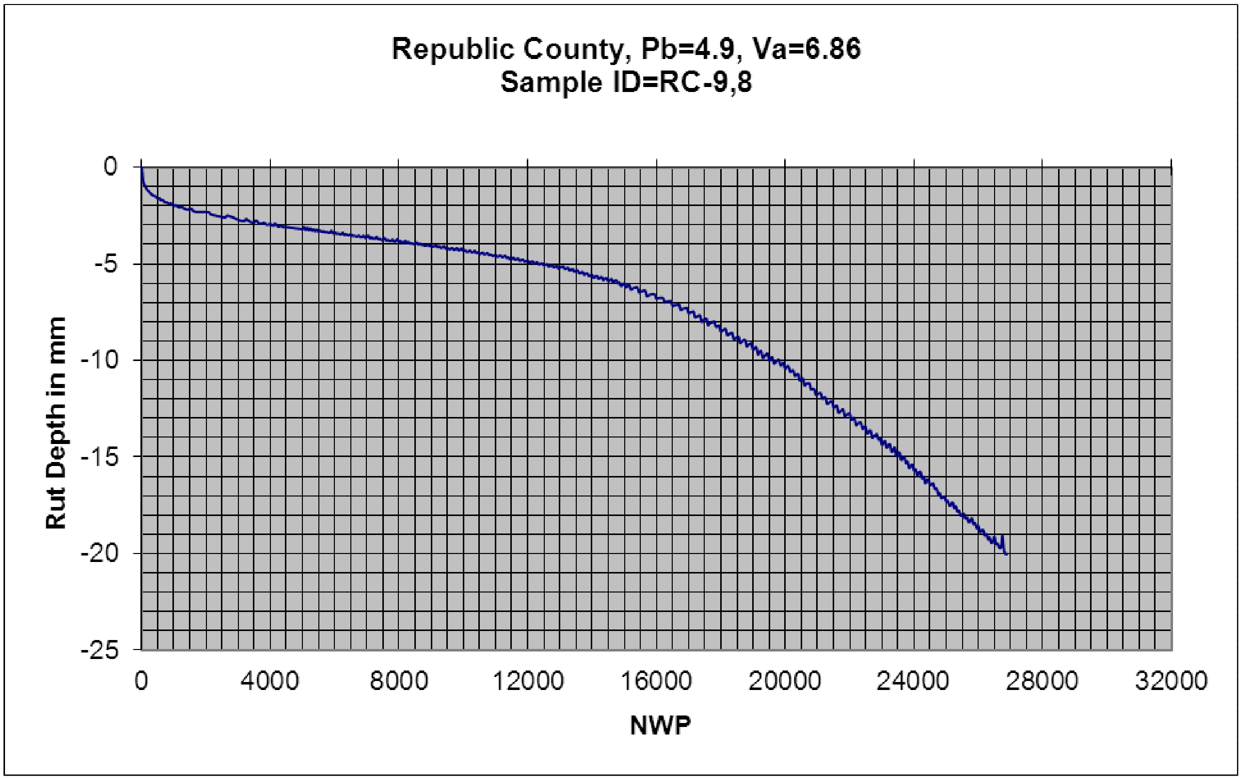
Republic County, Pb=4.9, Va=6.86
Sample ID=RC-9,8
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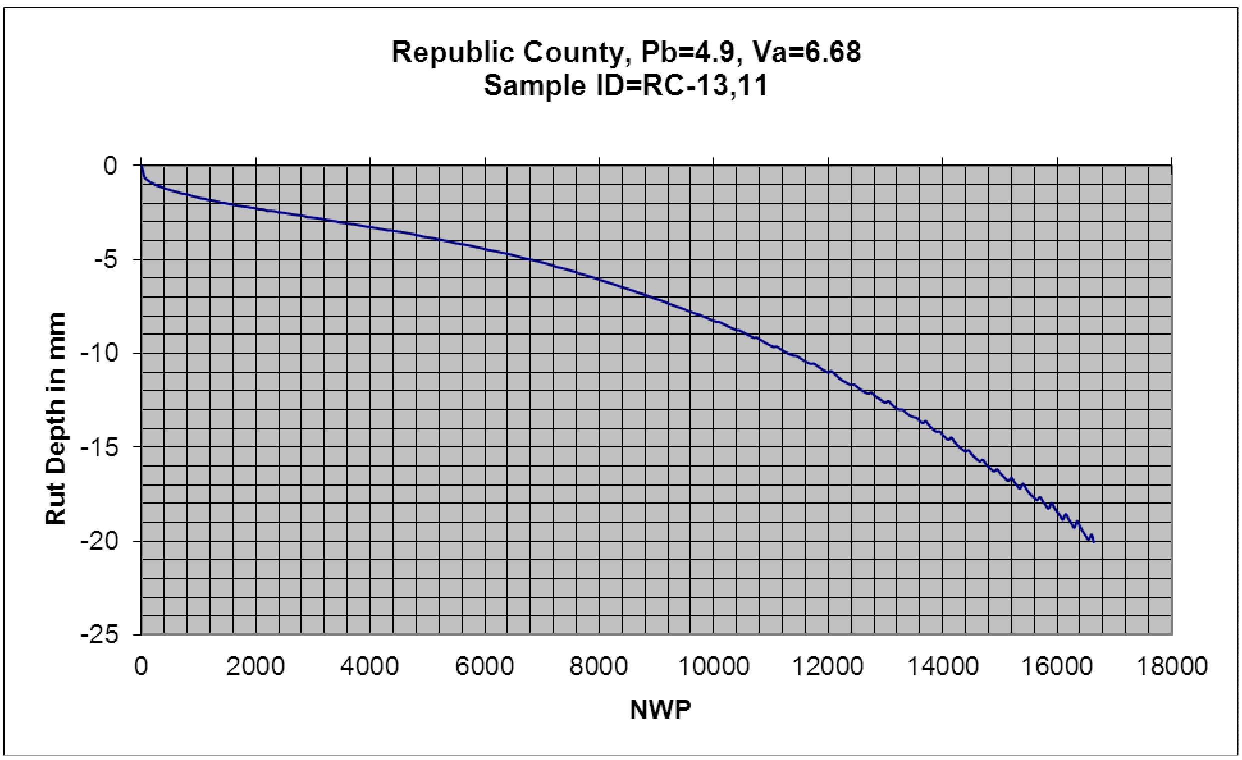
Republic County, Pb=4.9, Va=6.68
Sample ID=RC-13,11
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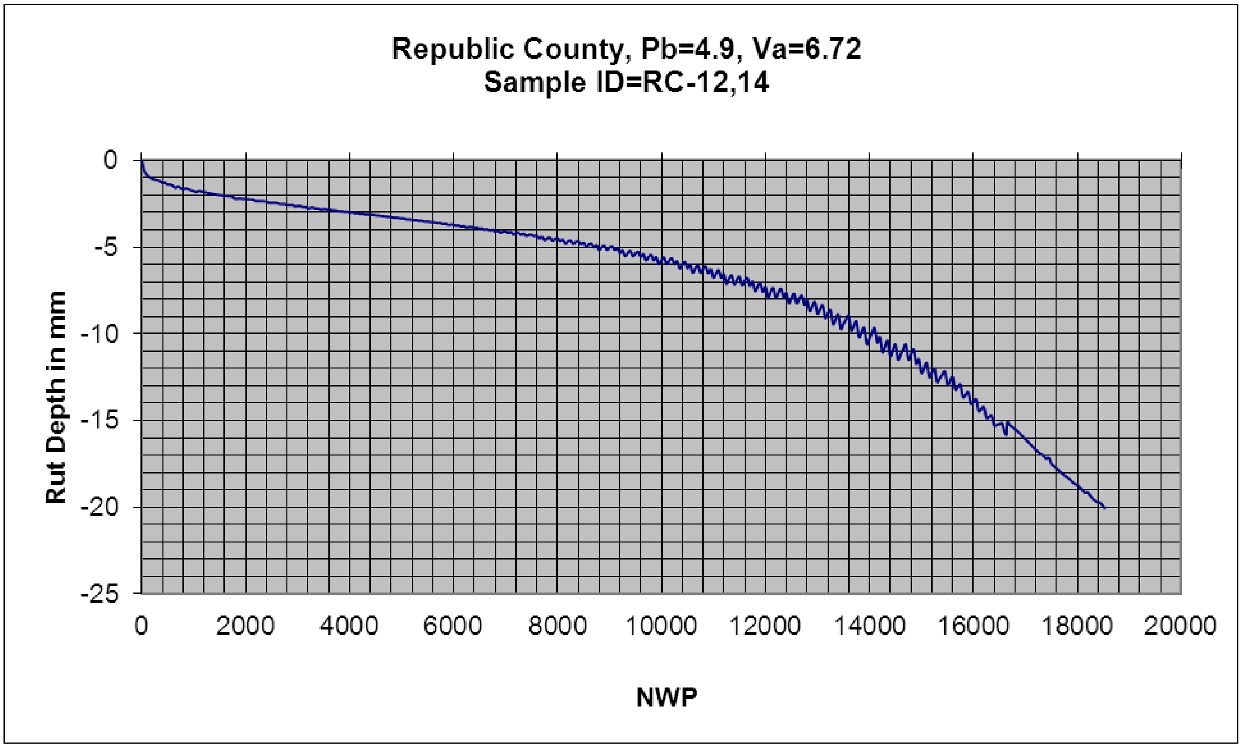
Republic County, Pb=4.9, Va=6.72
Sample ID=RC-12,14
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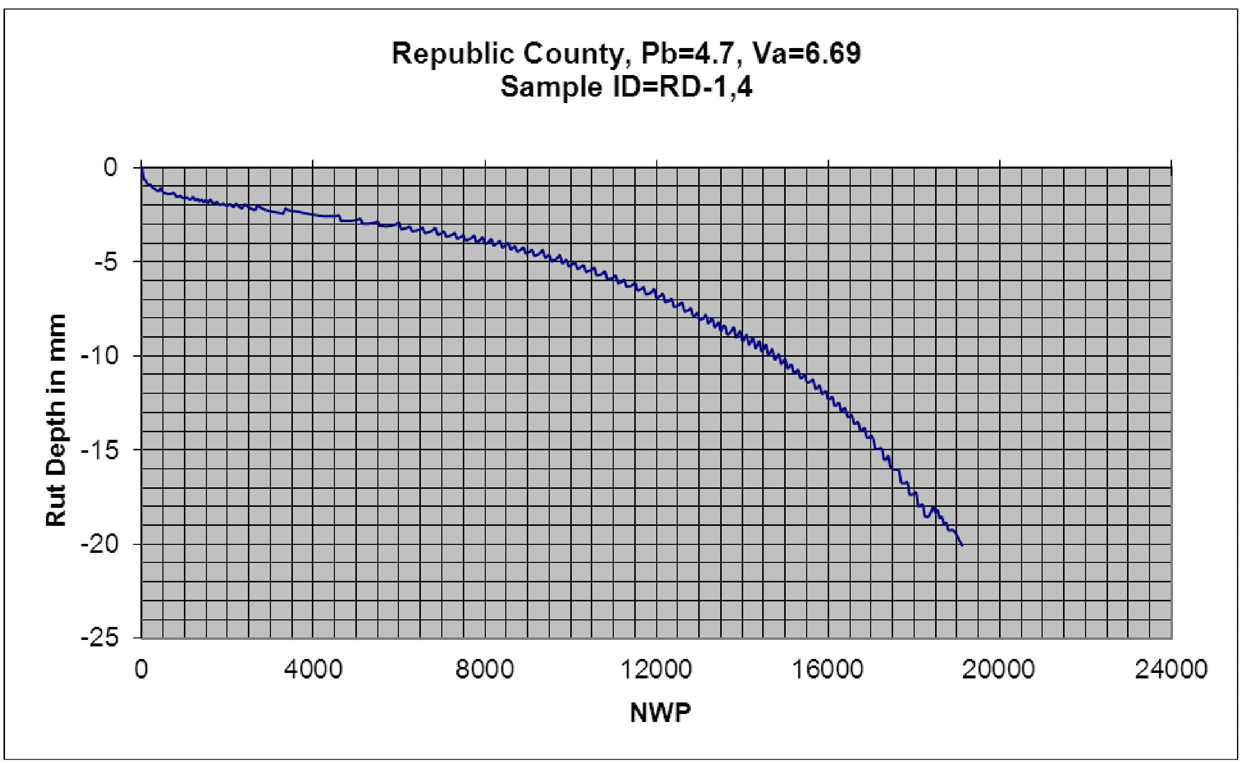
Republic County, Pb=4.7, Va=6.69
Sample ID=RD-1,4
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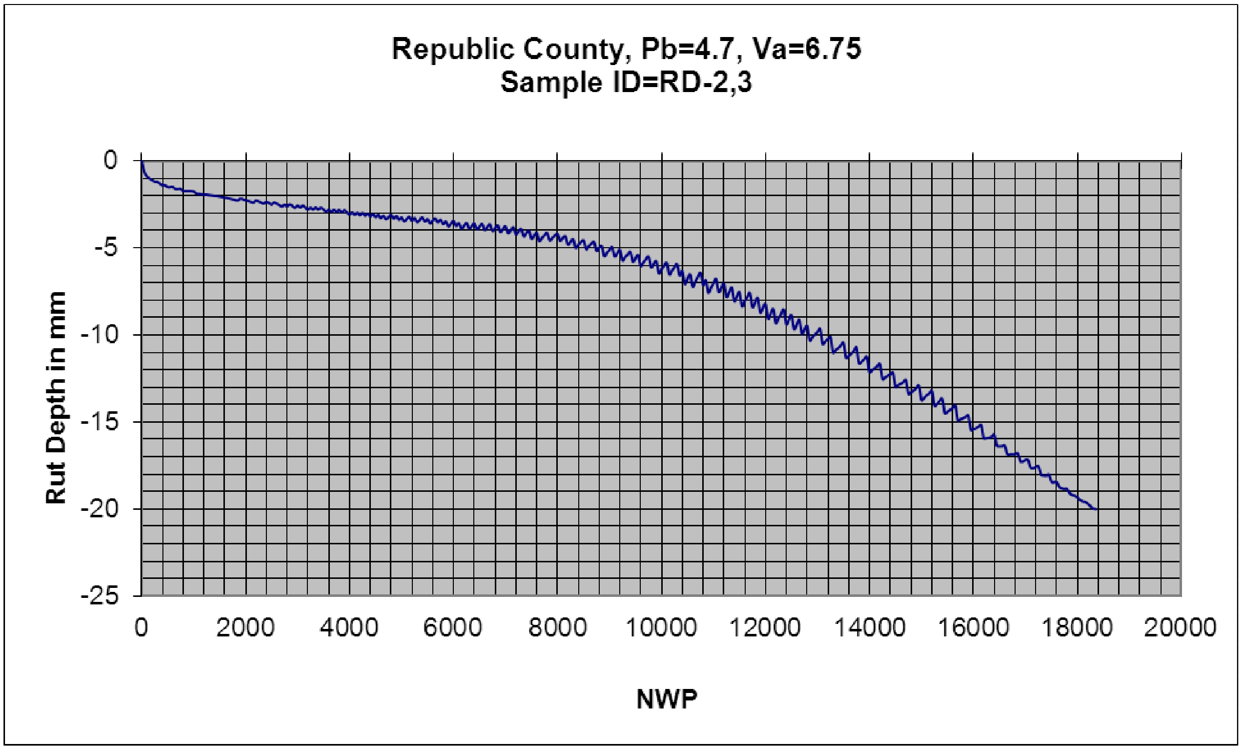
Republic County, Pb=4.7, Va=6.75
Sample ID=RD-2,3
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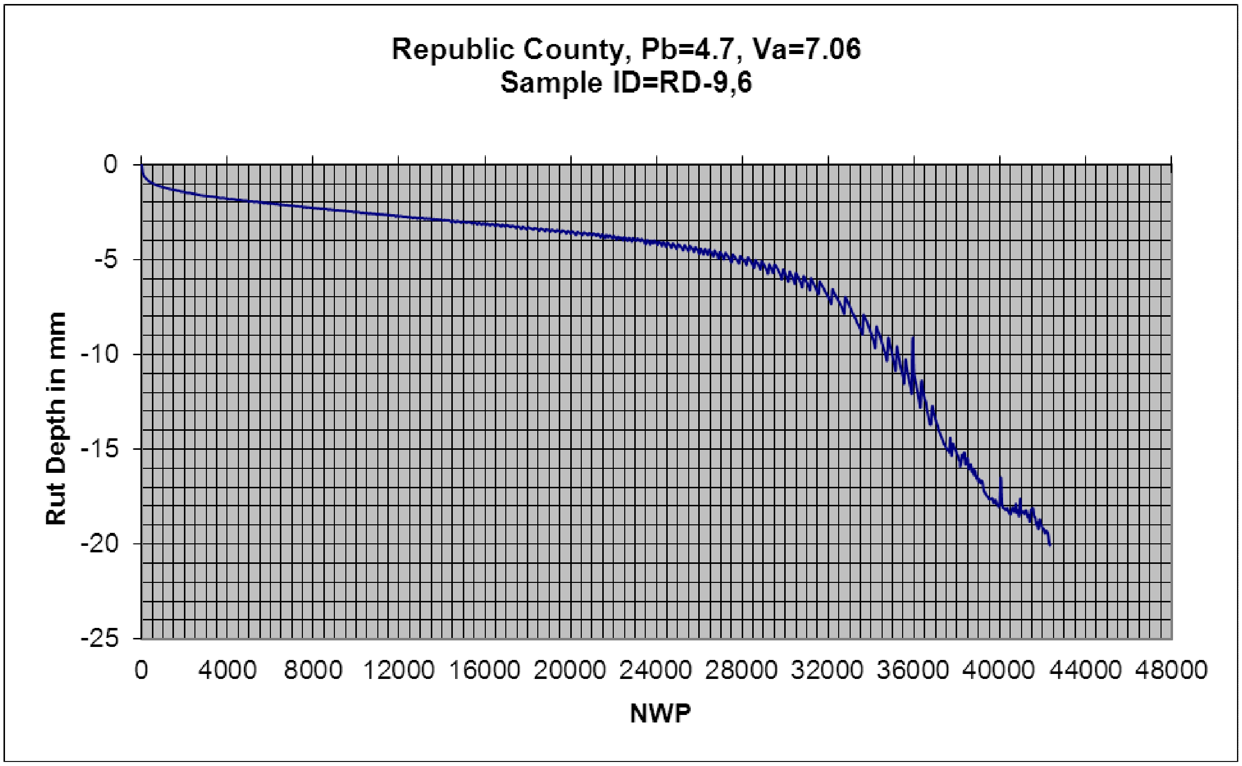
Republic County, Pb=4.7, Va=7.06
Sample ID=RD-9,6
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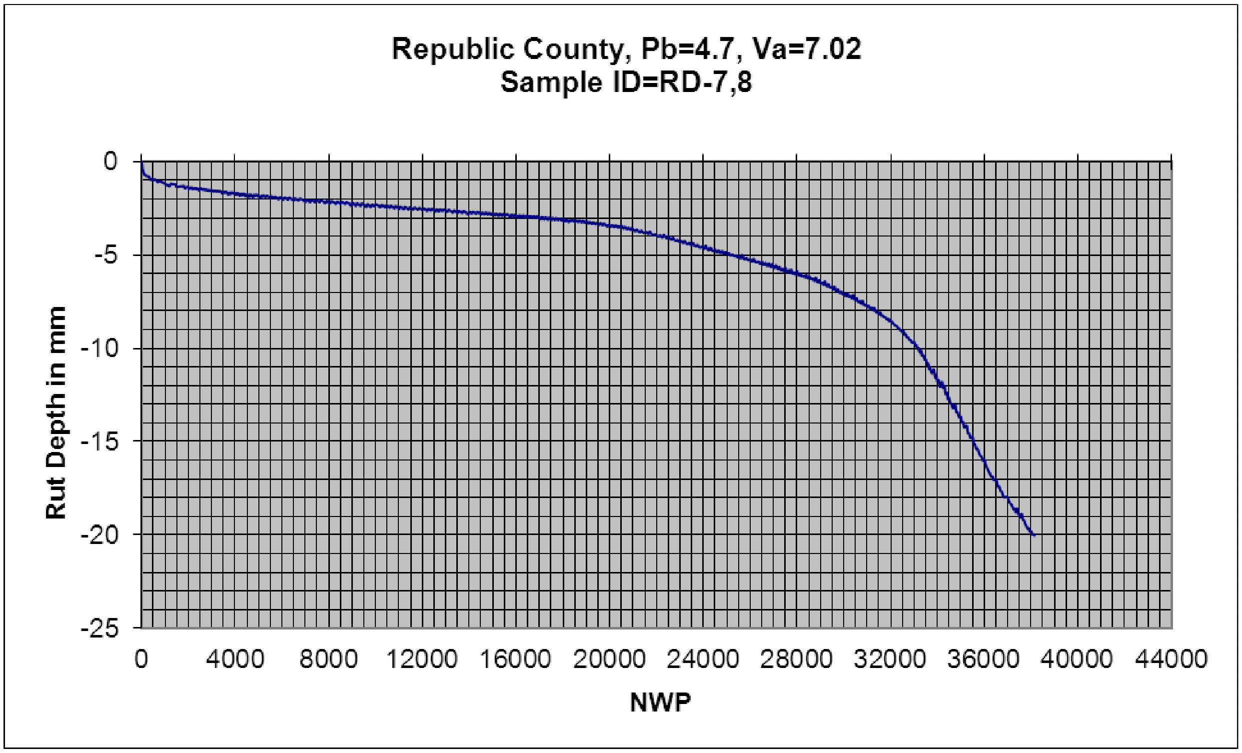
Republic County, Pb=4.7, Va=7.02
Sample ID=RD-7,8
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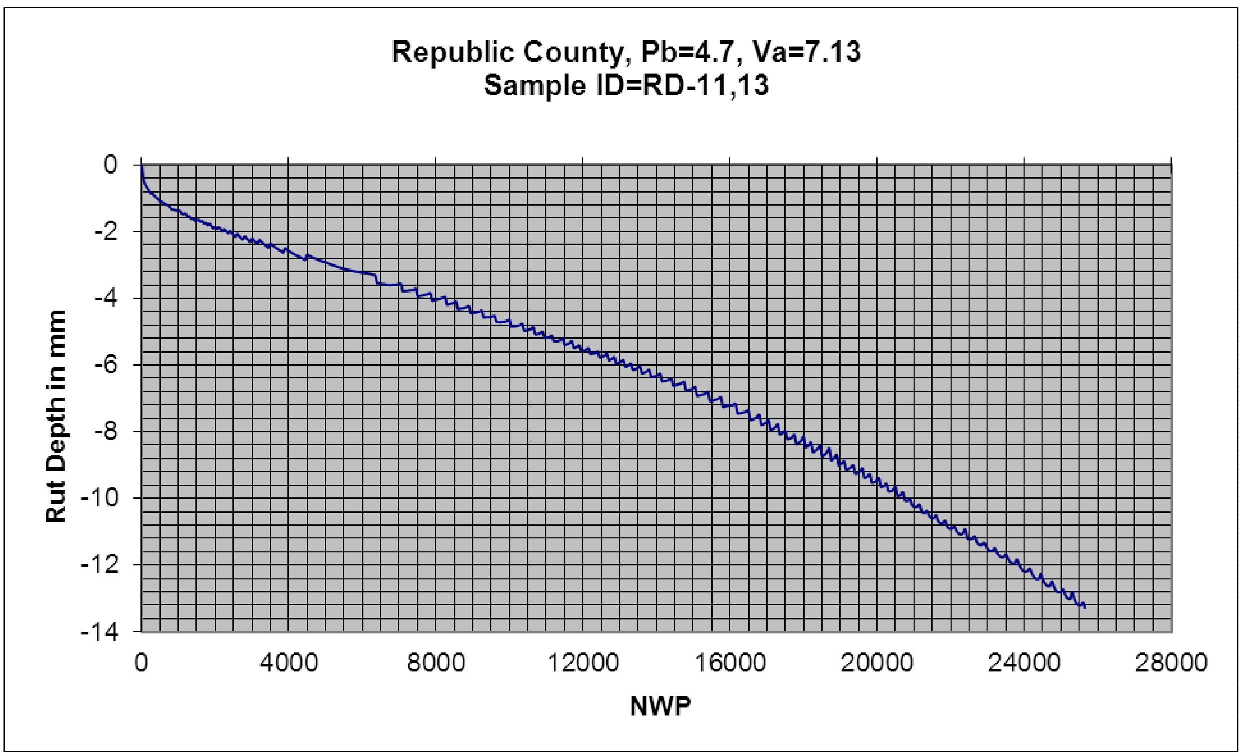
Republic County, Pb=4.7, Va=7.13
Sample ID=RD-11,13
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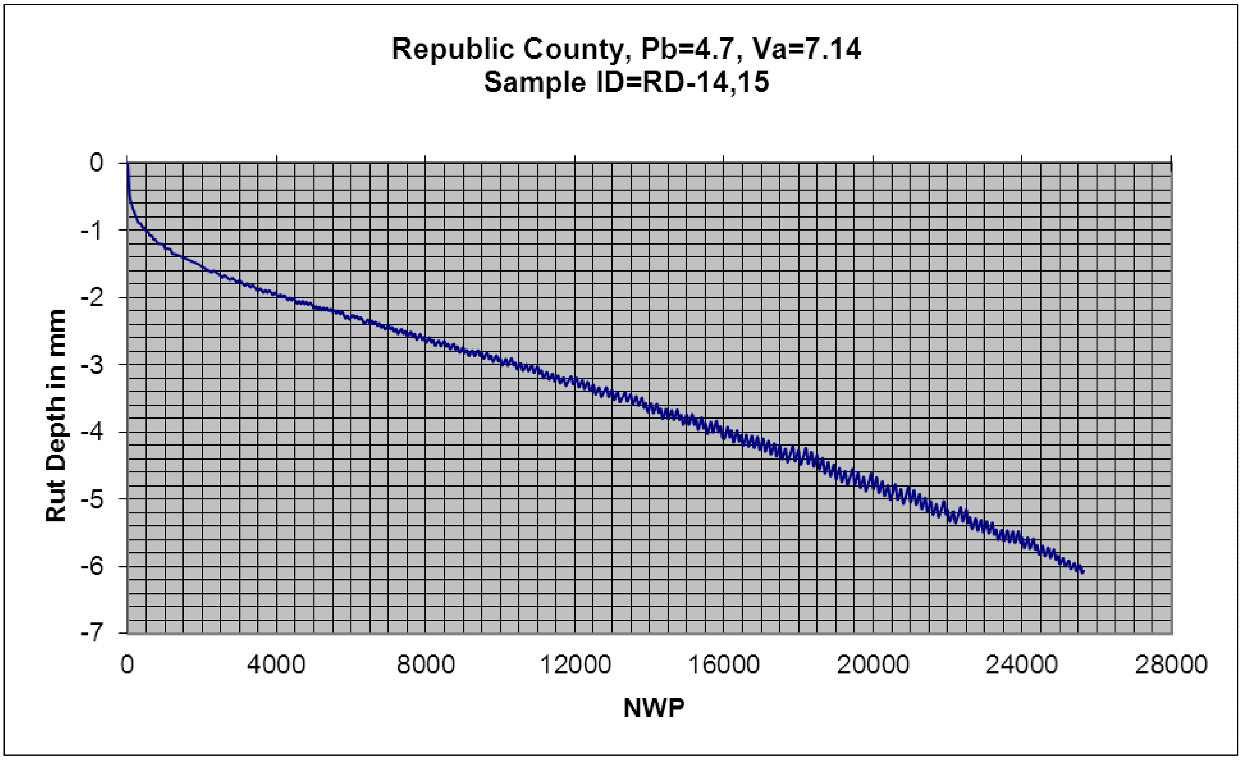
Republic County, Pb=4.7, Va=7.14
Sample ID=RD-14,15
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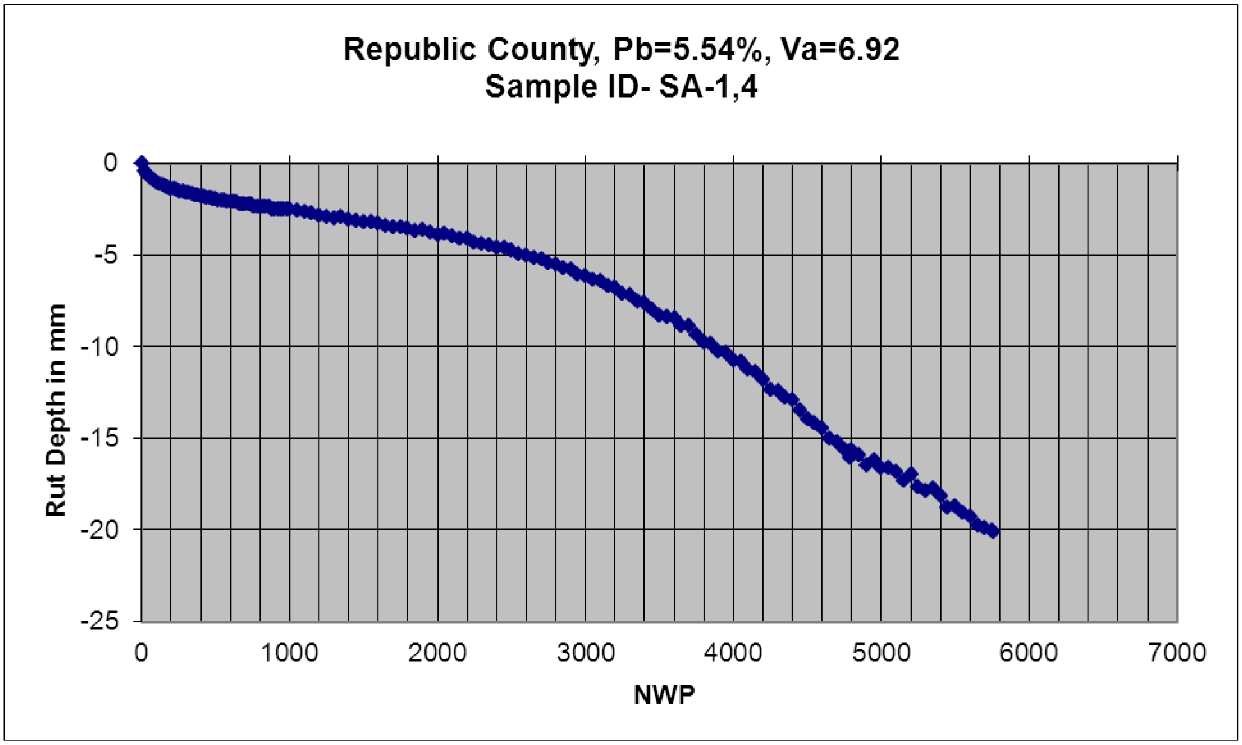
Republic County, Pb=5.54%, Va=6.92
Sample ID- SA-1,4
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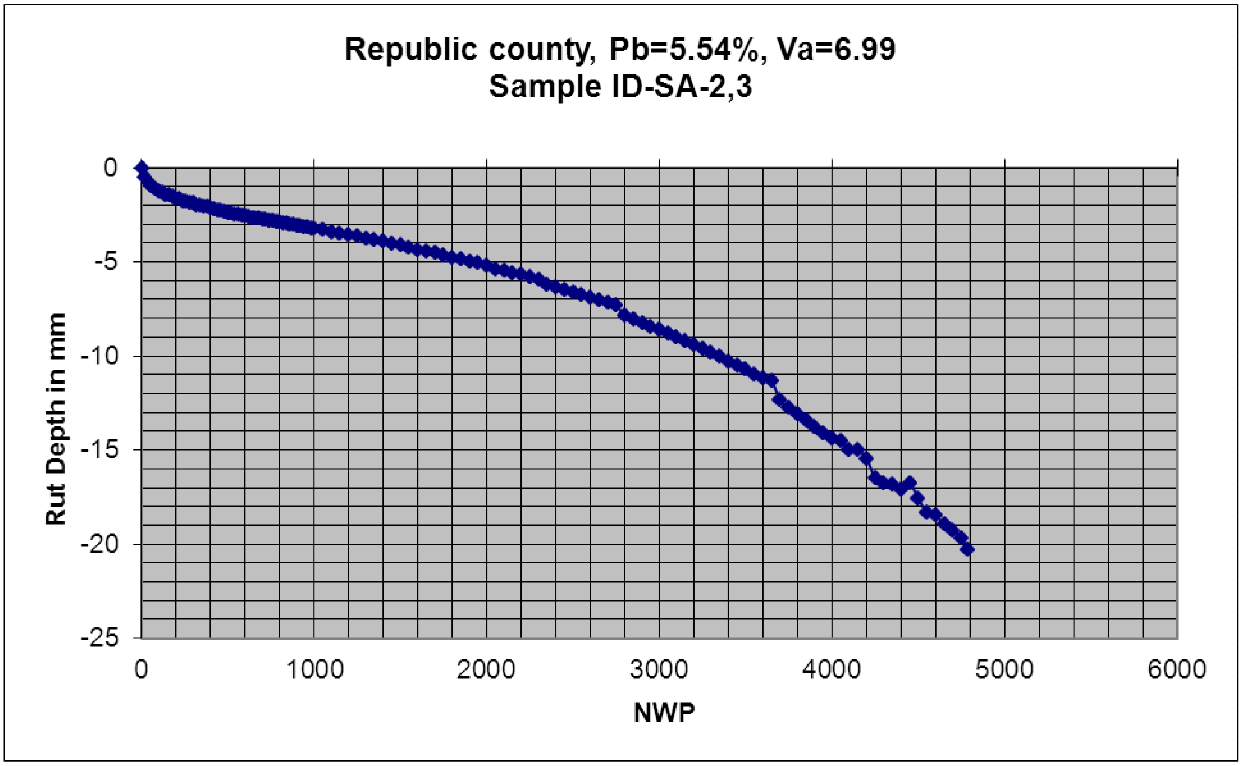
Republic county, Pb=5.54%, Va=6.99
Sample ID-SA-2,3
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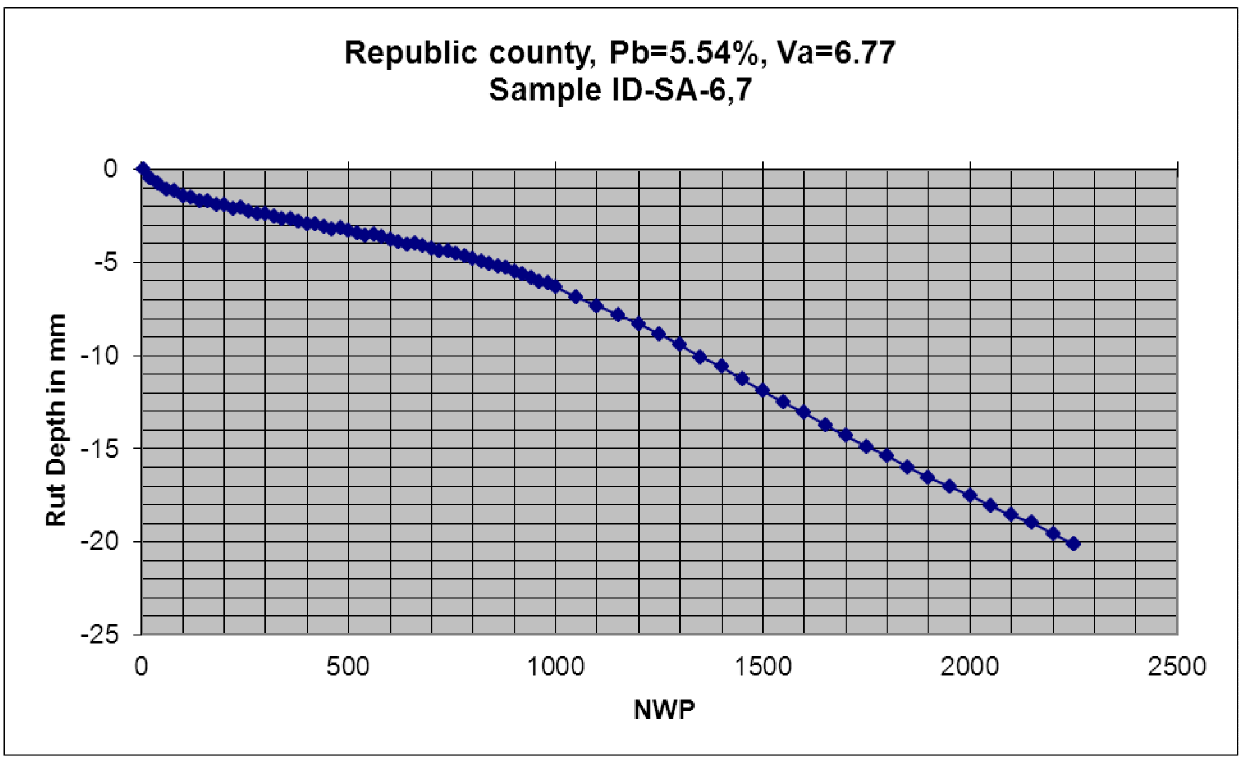
Republic county, Pb=5.54%, Va=6.77
Sample ID-SA-6,7
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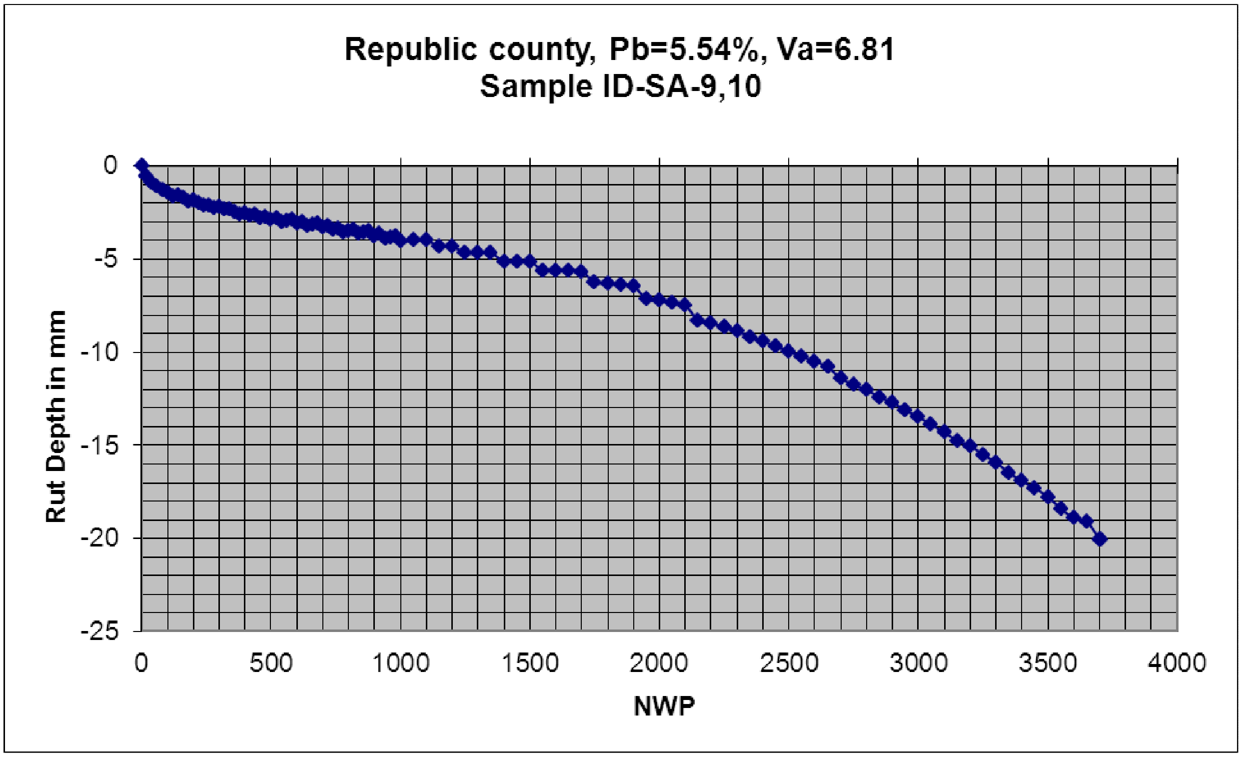
Republic county, Pb=5.54%, Va=6.81
Sample ID-SA-9,10
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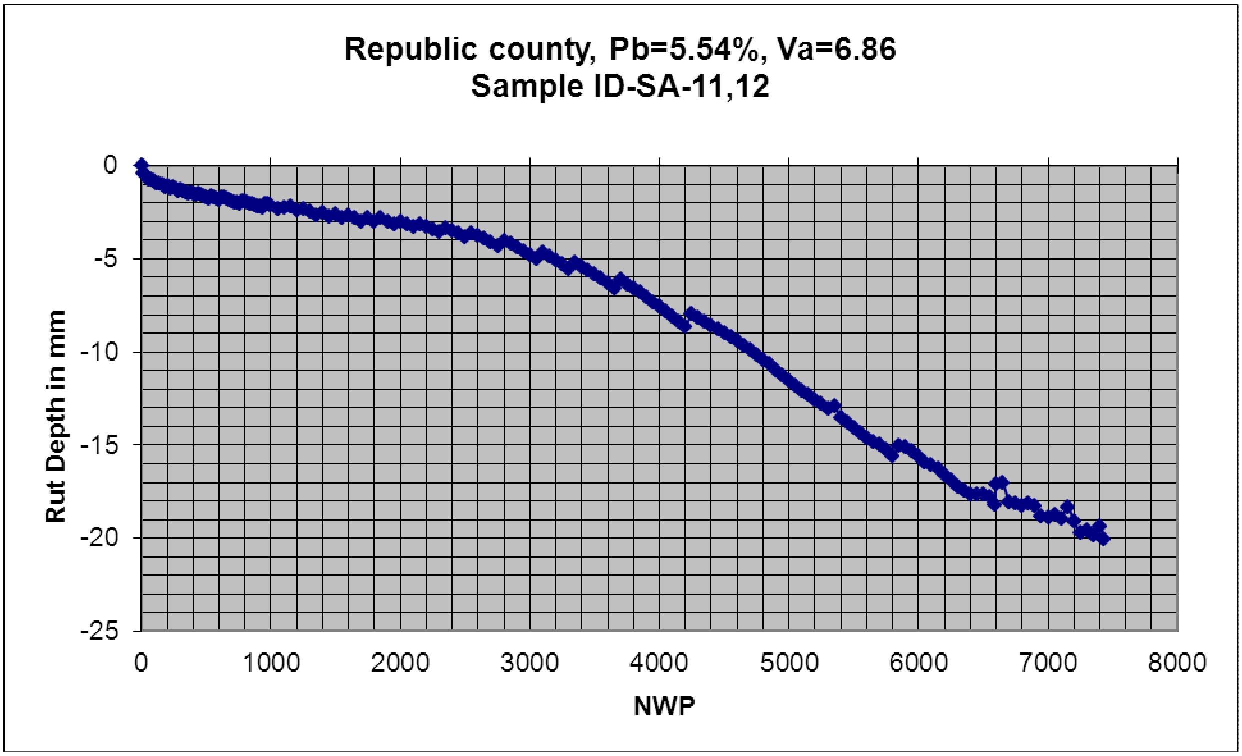
Republic county, Pb=5.54%, Va=6.86
Sample ID-SA-11,12
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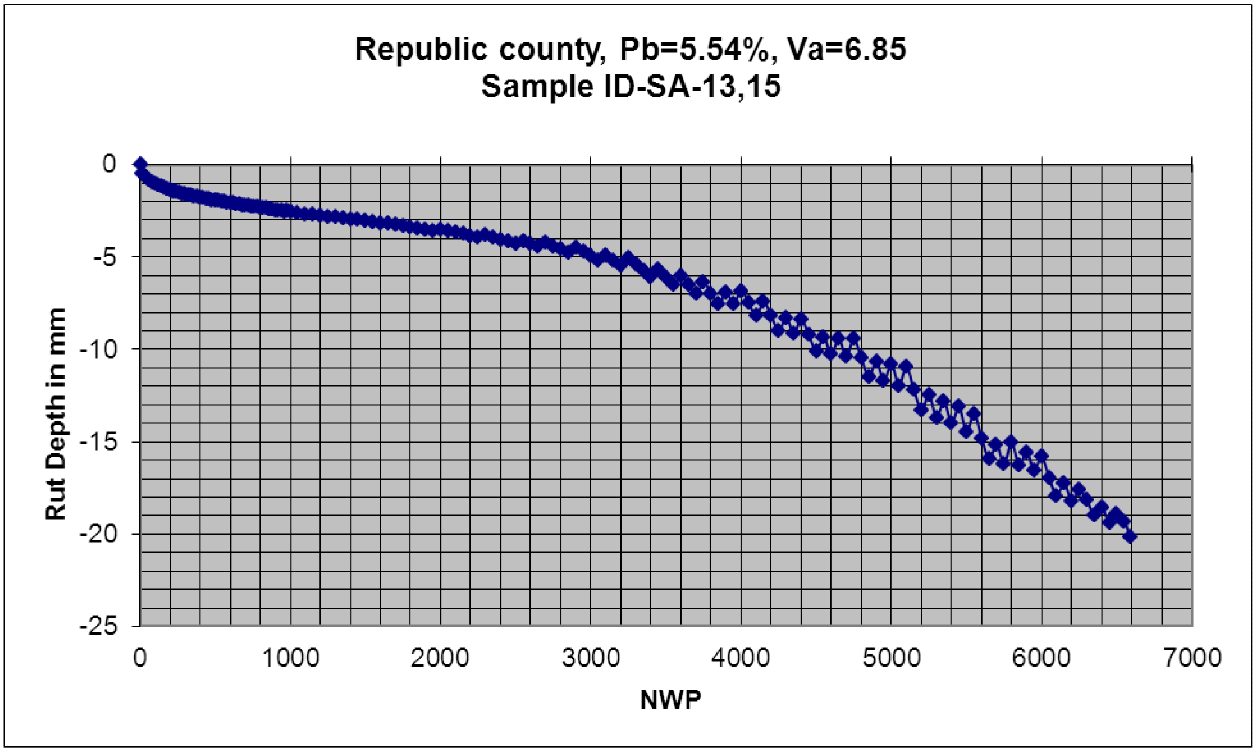
Republic county, Pb=5.54%, Va=6.85
Sample ID-SA-13,15
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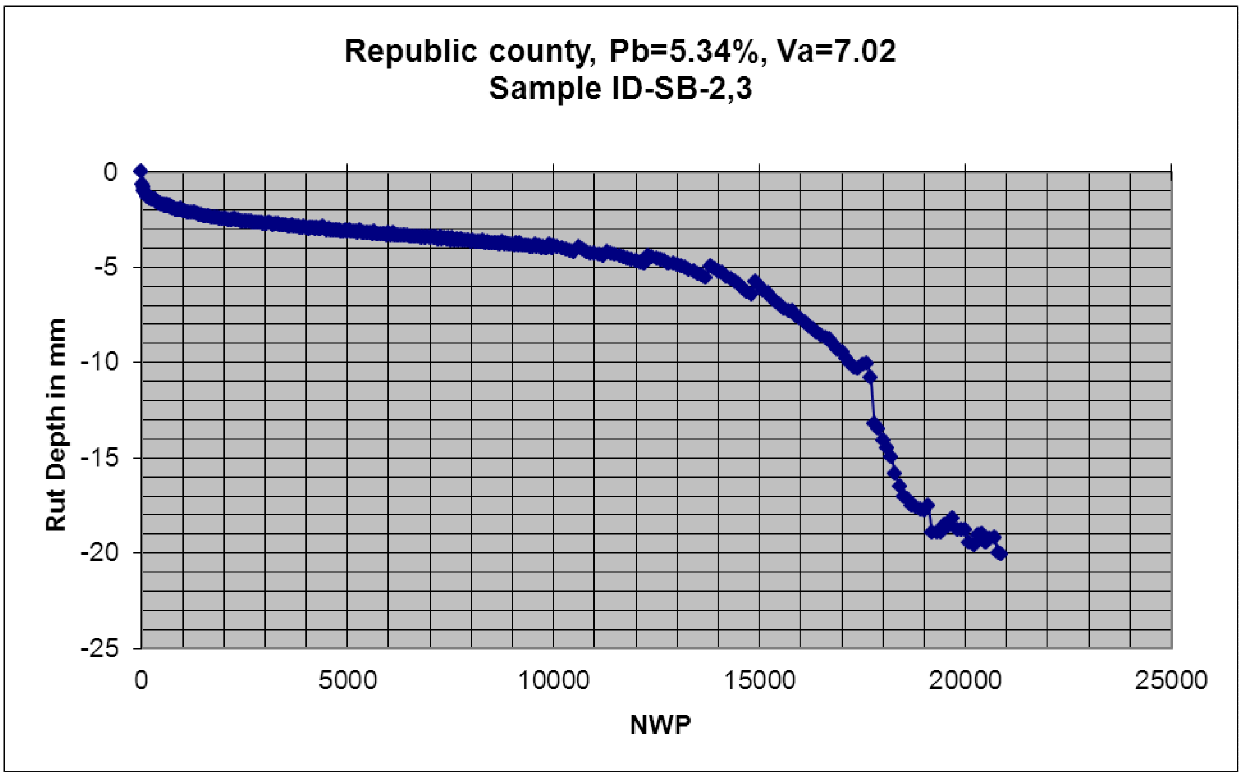
Republic county, Pb=5.34%, Va=7.02
Sample ID-SB-2,3
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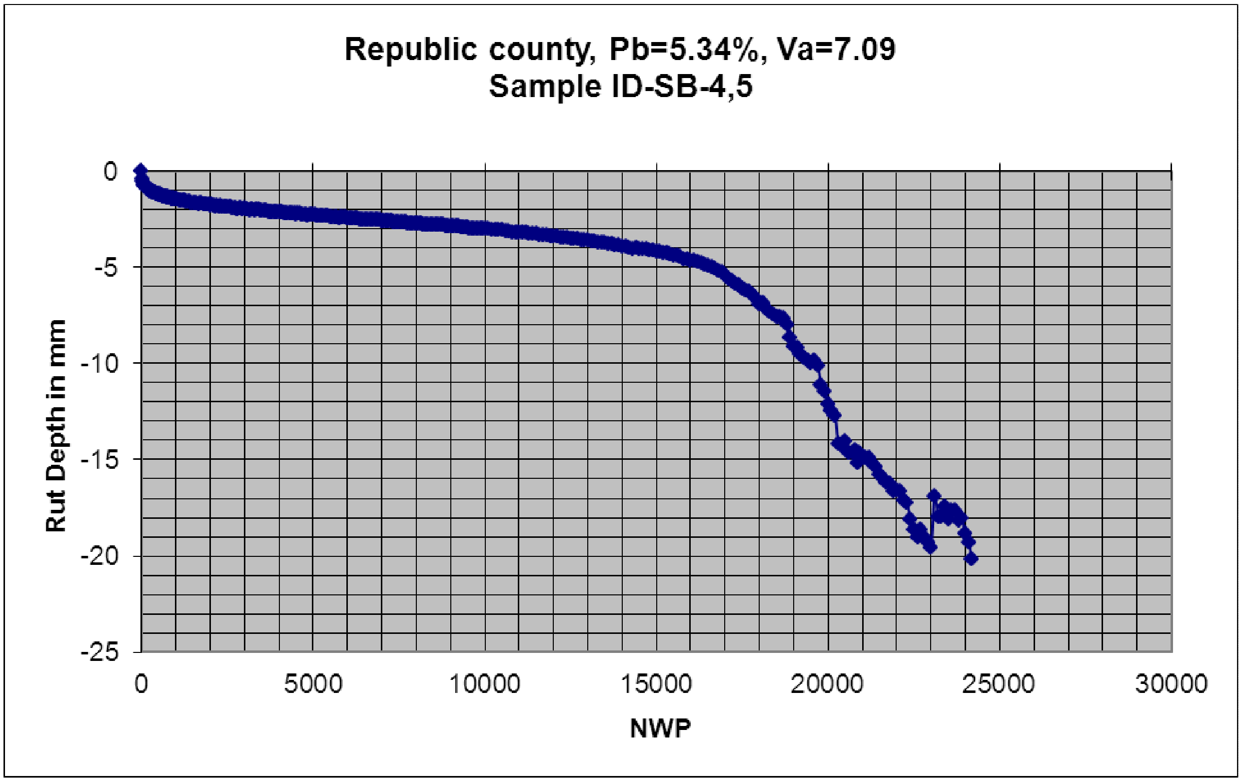
Republic county, Pb=5.34%, Va=7.09
Sample ID-SB-4,5
Rut Depth in mm
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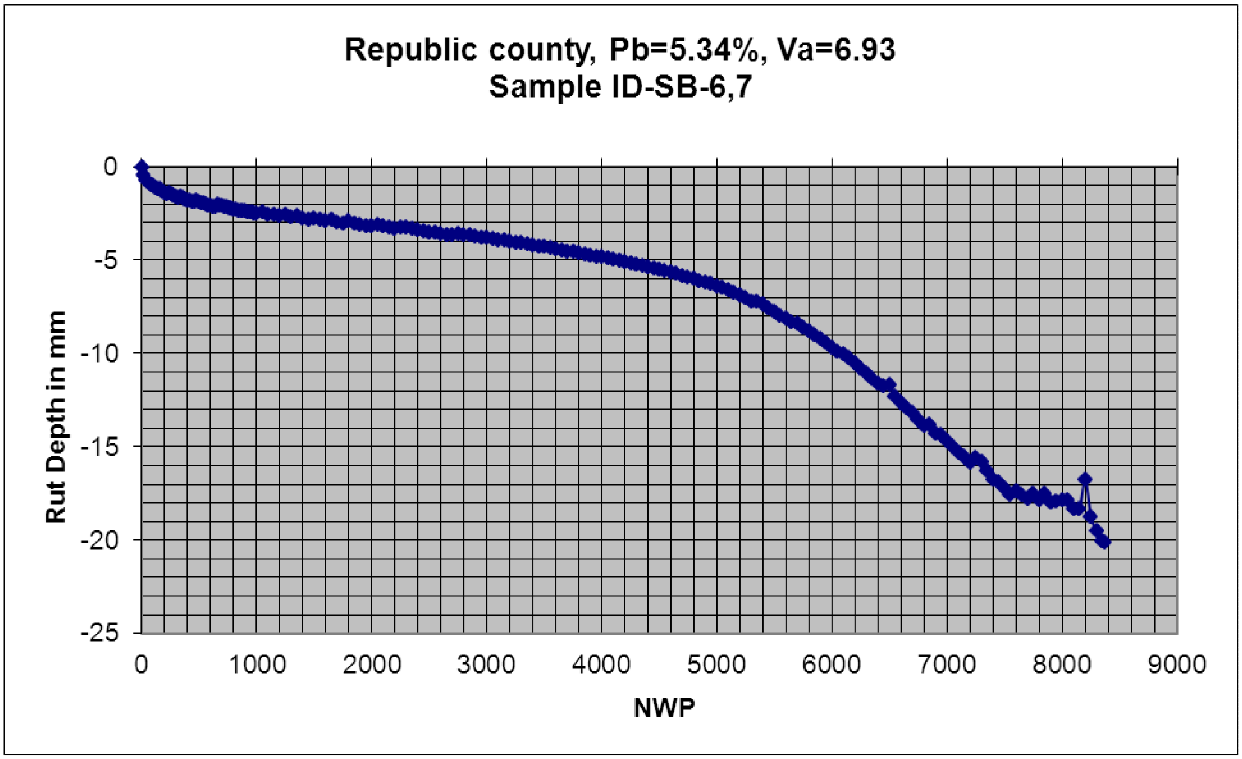
Republic county, Pb=5.34%, Va=6.93
Sample ID-SB-6,7
Rut Depth in mm
NWP
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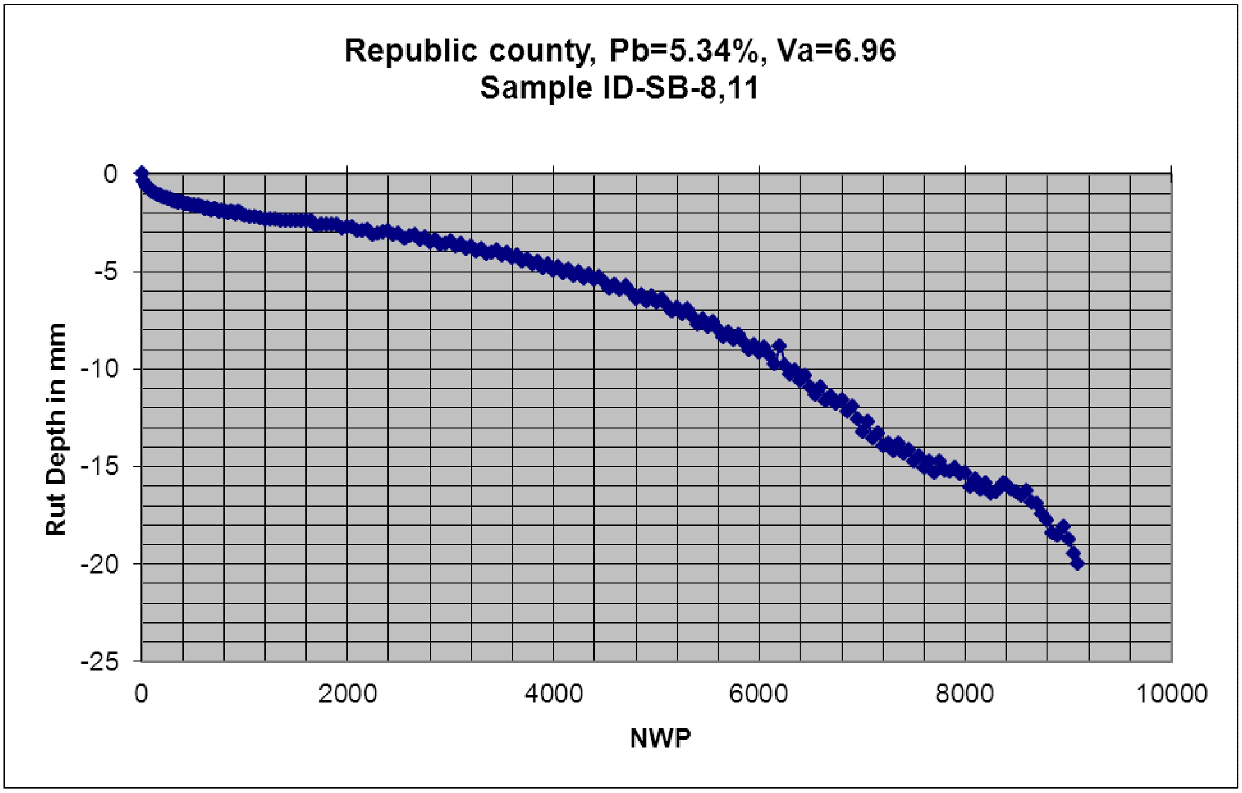
Republic county, Pb=5.34%, Va=6.96
Sample ID-SB-8,11
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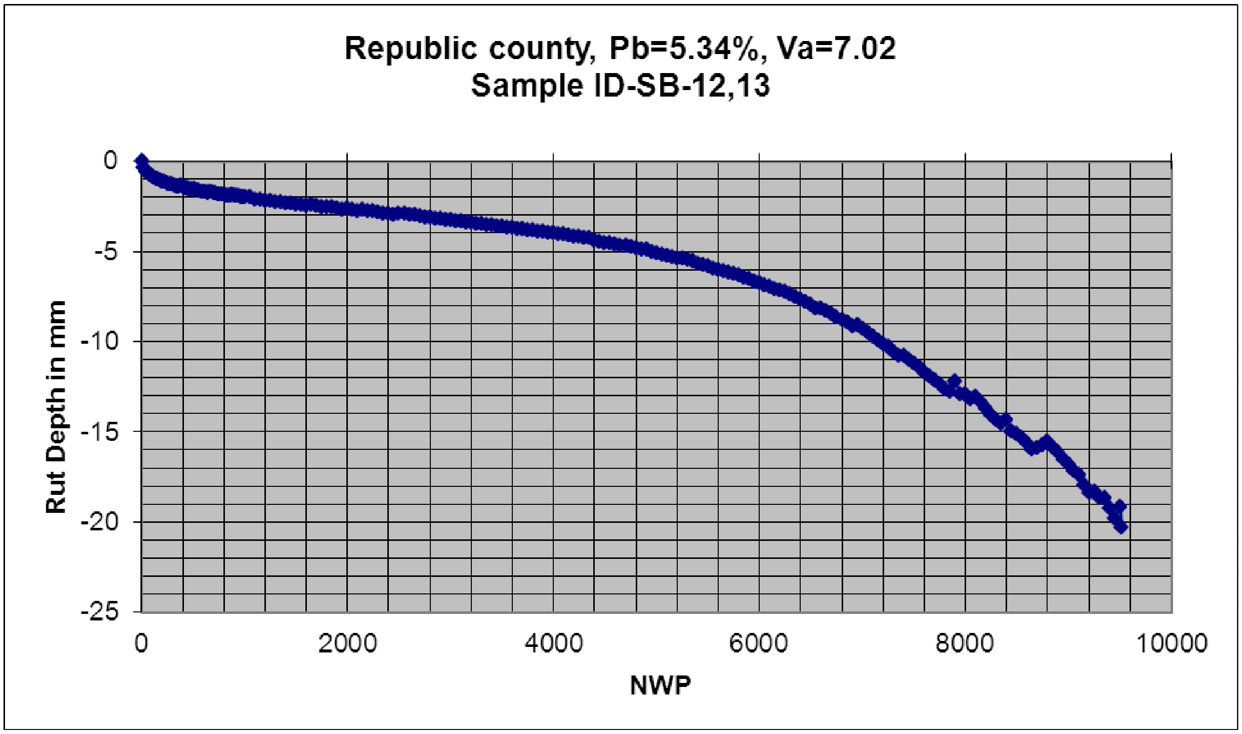
Republic county, Pb=5.34%, Va=7.02
Sample ID-SB-12,13
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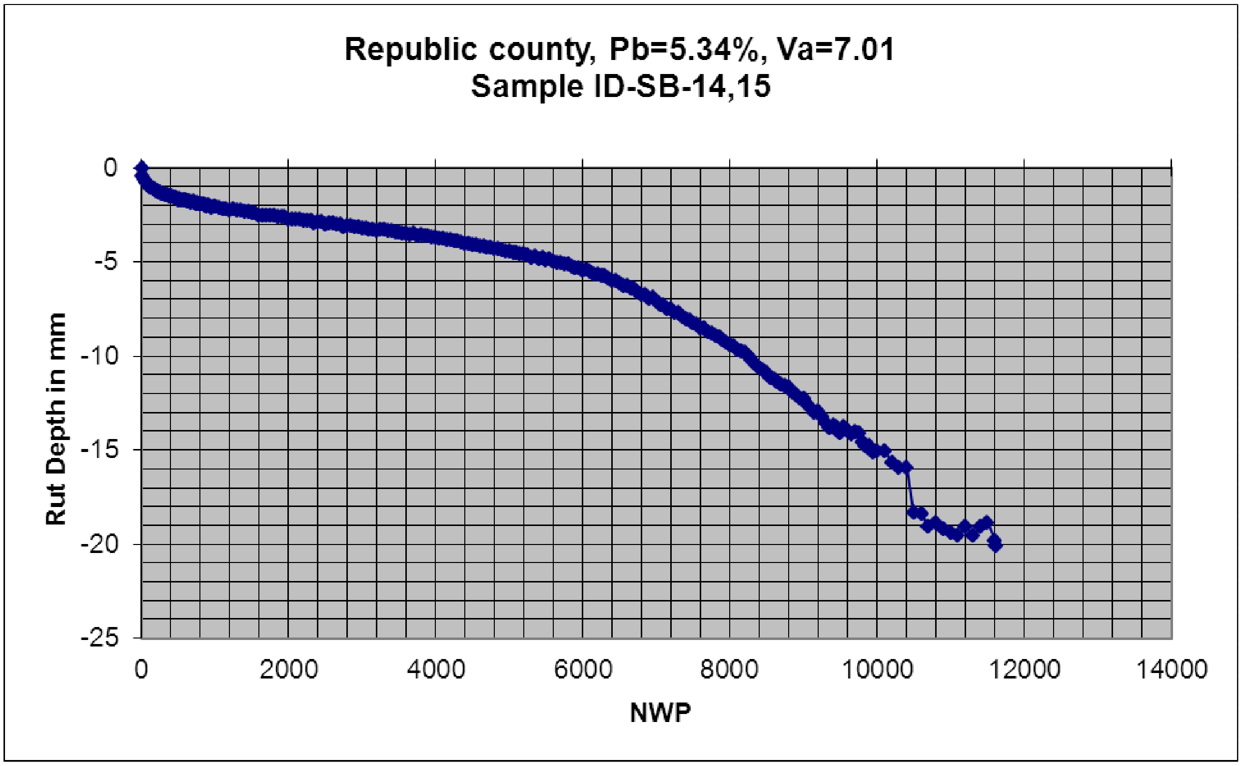
NWP



Republic county, Pb=5.34%, Va=7.01
Sample ID-SB-14,15
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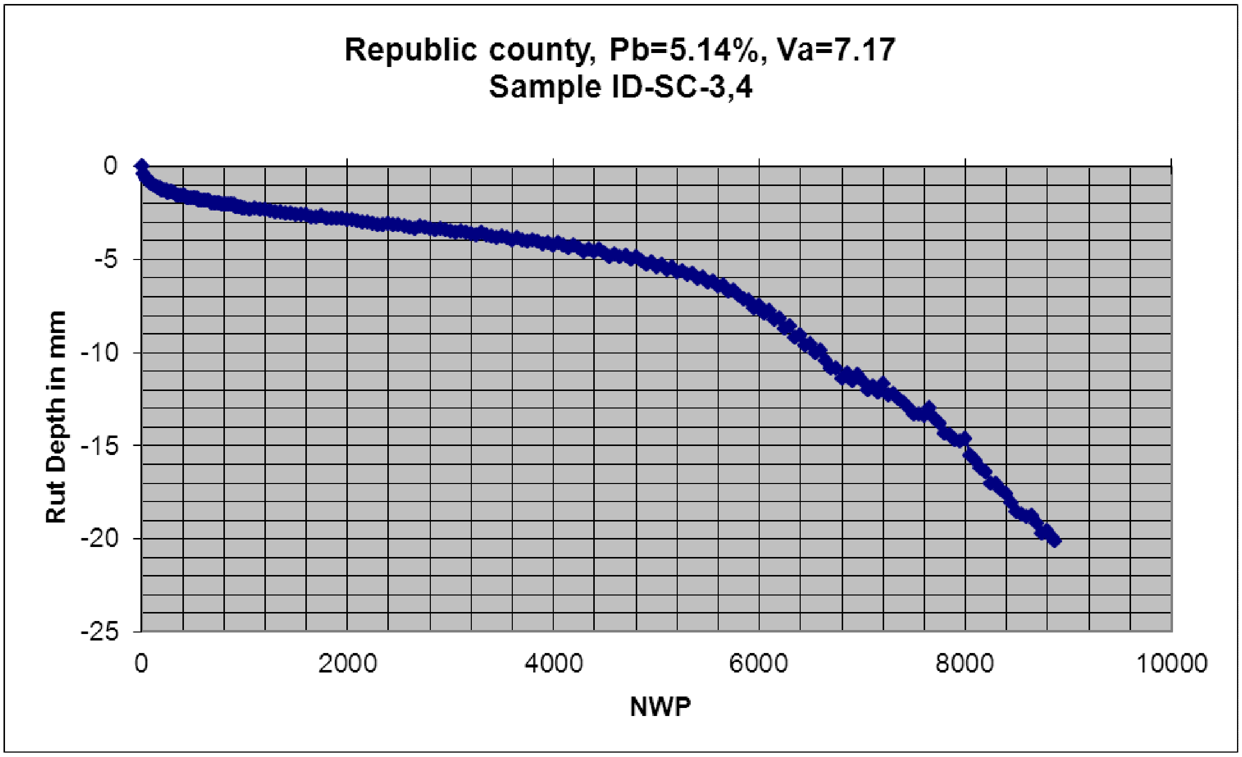

Republic county, Pb=5.14%, Va=7.17
Sample ID-SC-3,4
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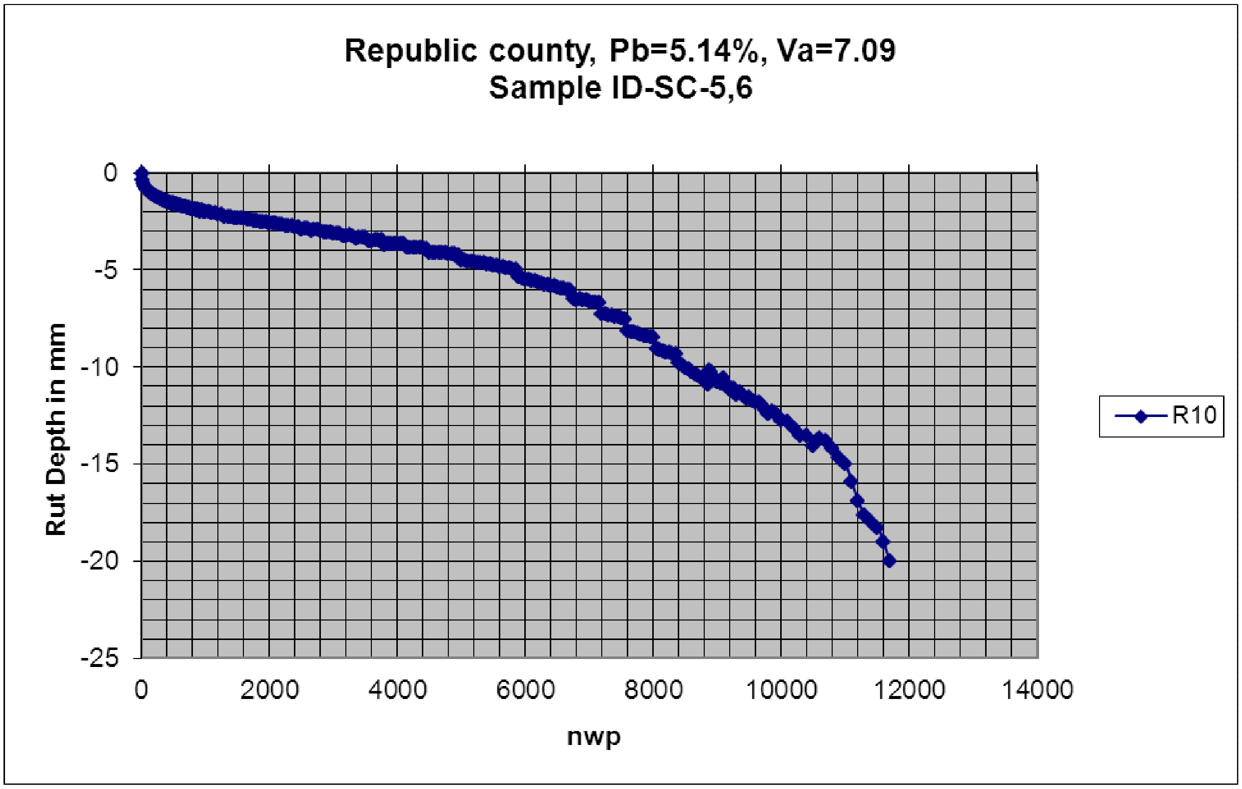
Republic county, Pb=5.14%, Va=7.09
Sample ID-SC-5,6
Rut Depth in mm
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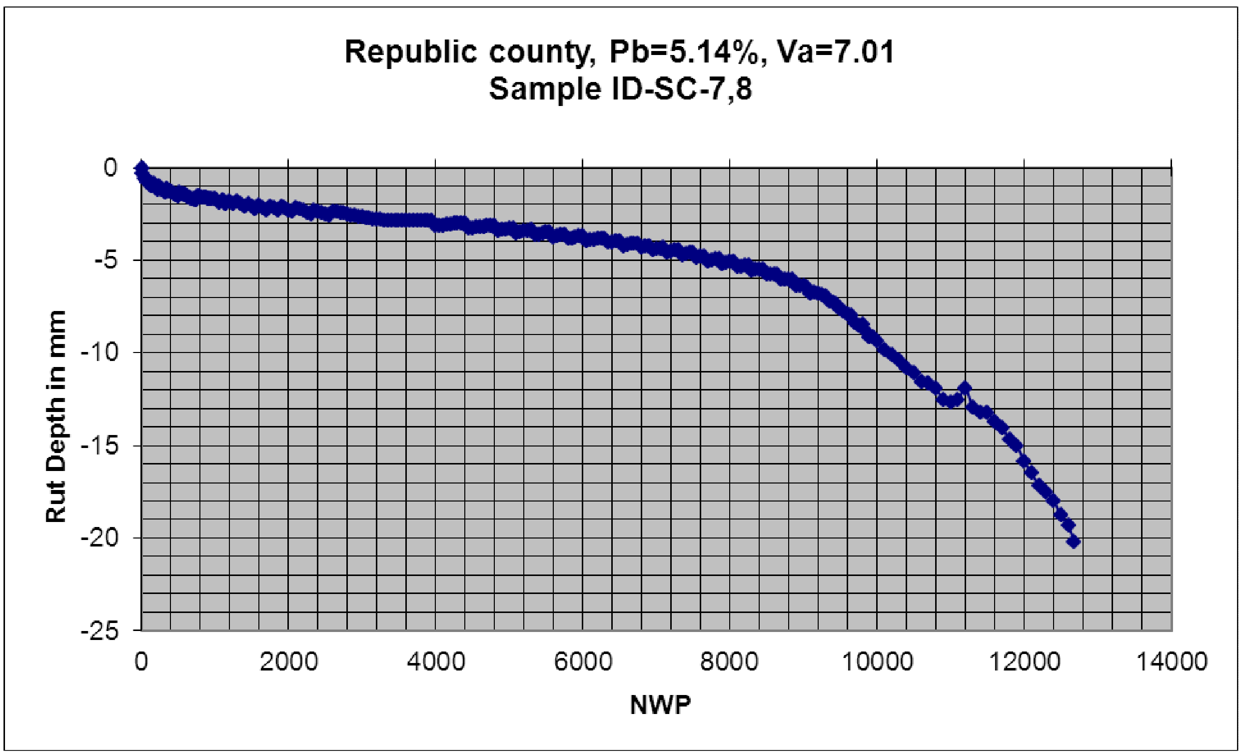
R10


Republic county, Pb=5.14%, Va=7.01
Sample ID-SC-7,8
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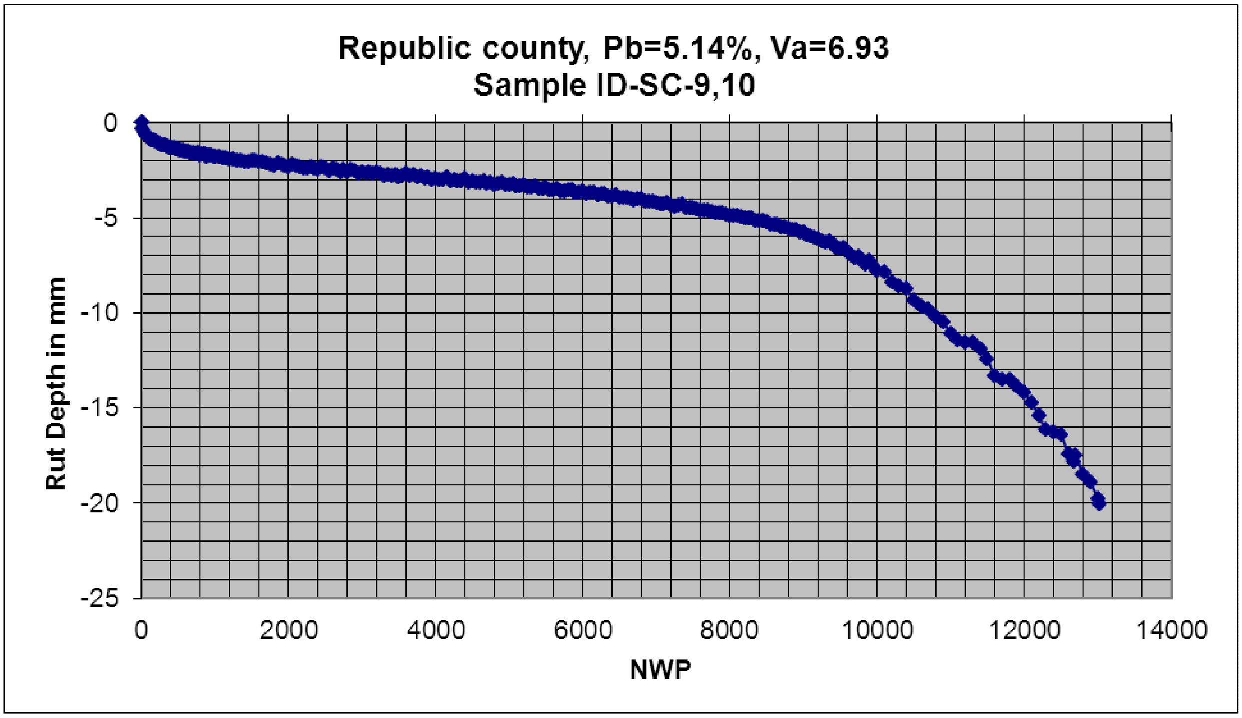
Republic county, Pb=5.14%, Va=6.93
Sample ID-SC-9,10
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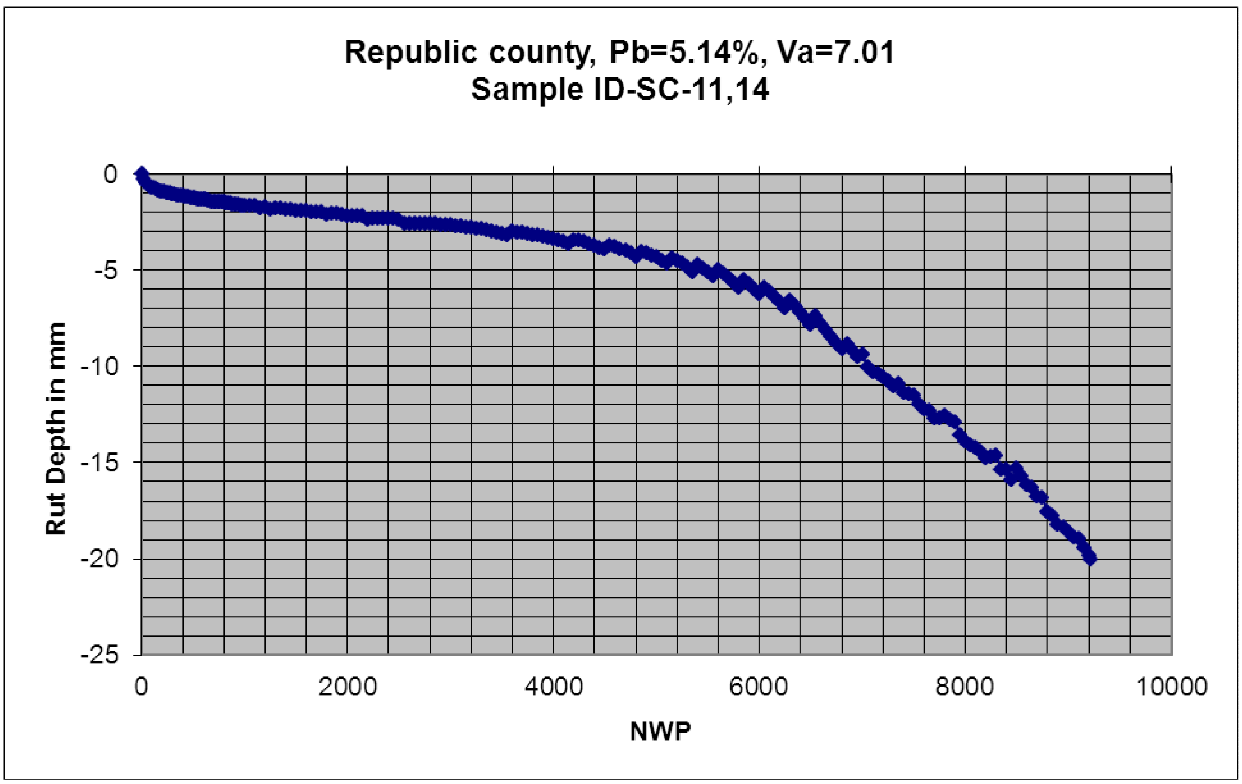
Republic county, Pb=5.14%, Va=7.01
Sample ID-SC-11,14
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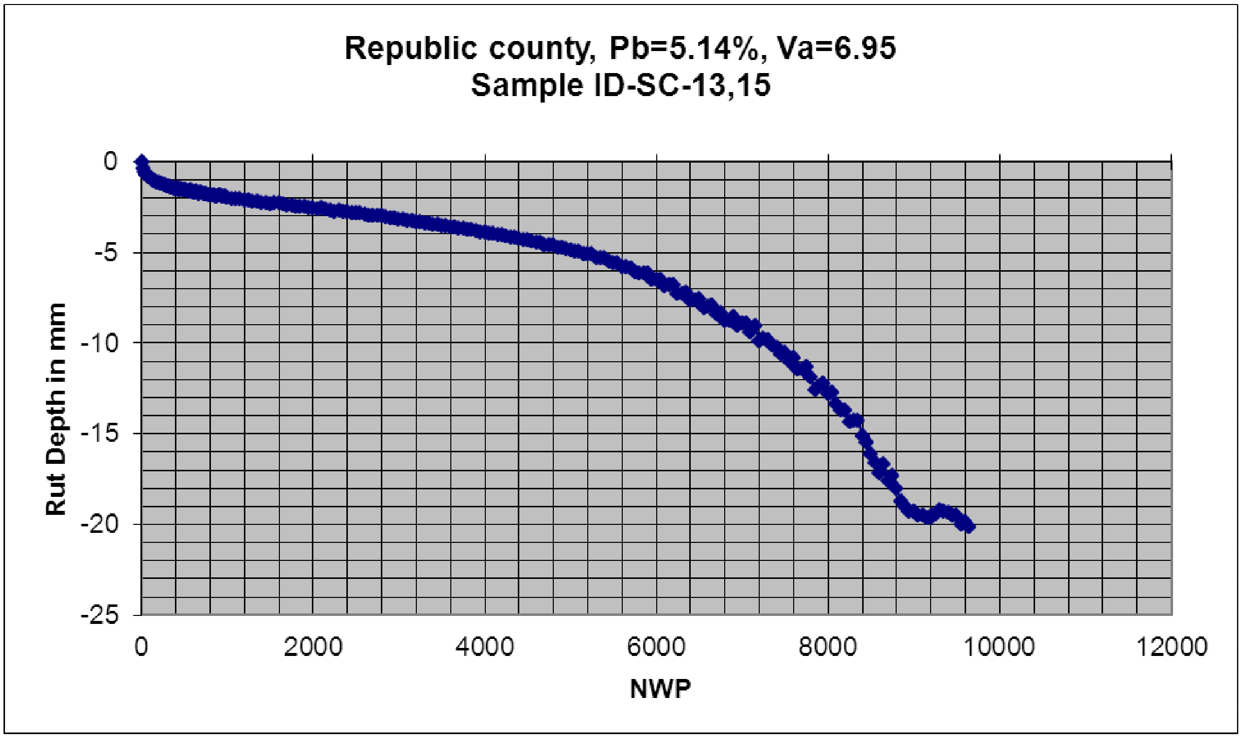
Republic county, Pb=5.14%, Va=6.95
Sample ID-SC-13,15
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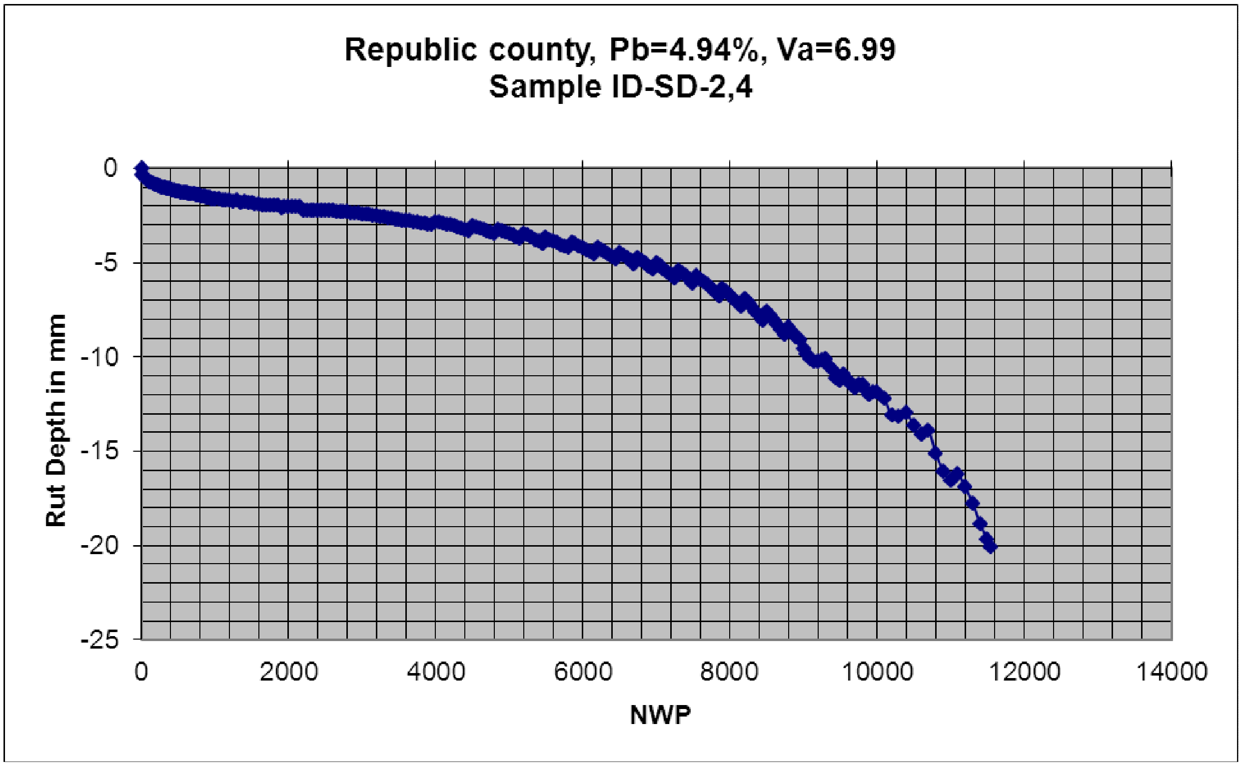
Republic county, Pb=4.94%, Va=6.99
Sample ID-SD-2,4
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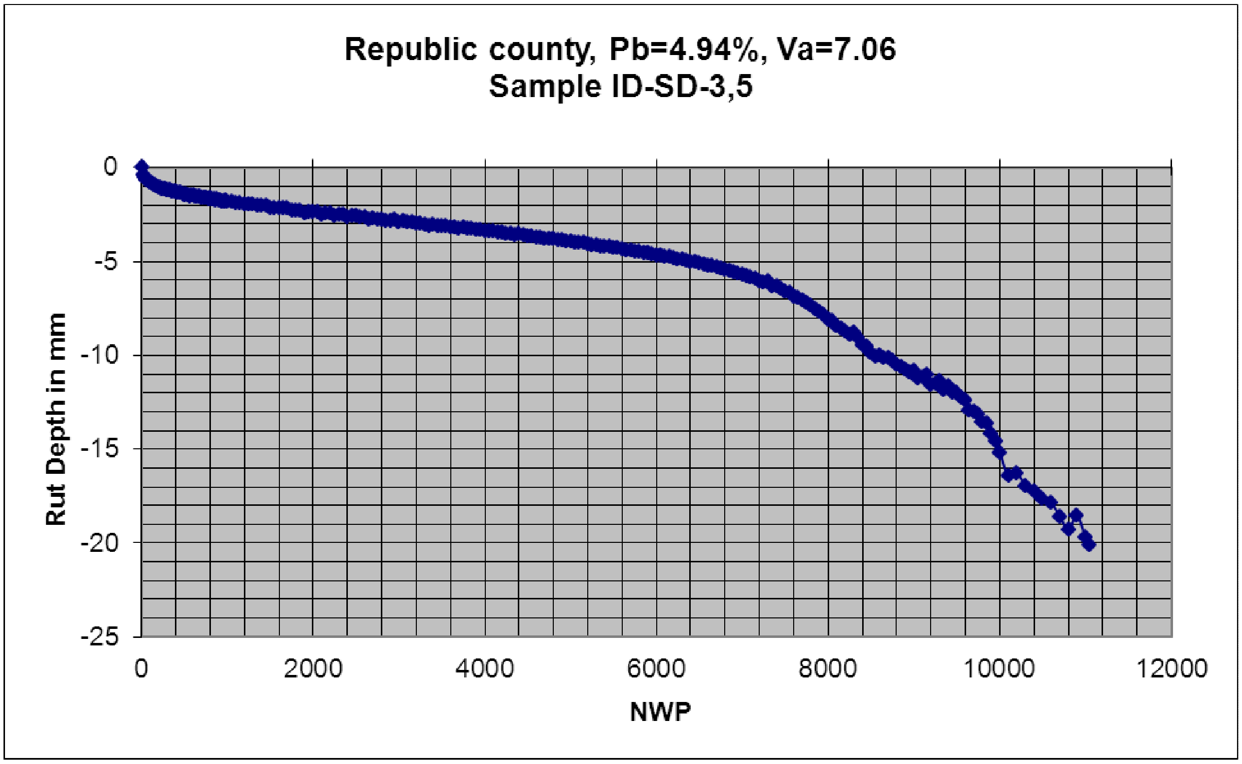
Republic county, Pb=4.94%, Va=7.06
Sample ID-SD-3,5
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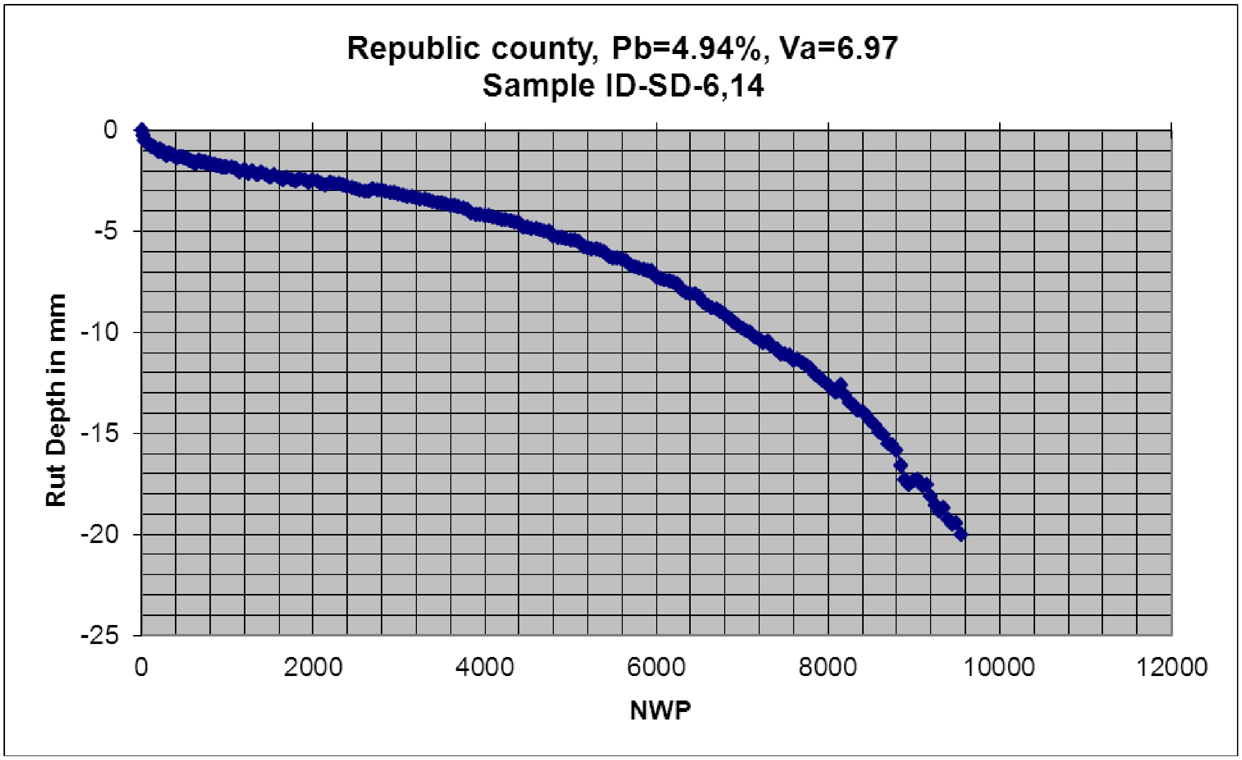
Republic county, Pb=4.94%, Va=6.97
Sample ID-SD-6,14
Rut Depth in mm
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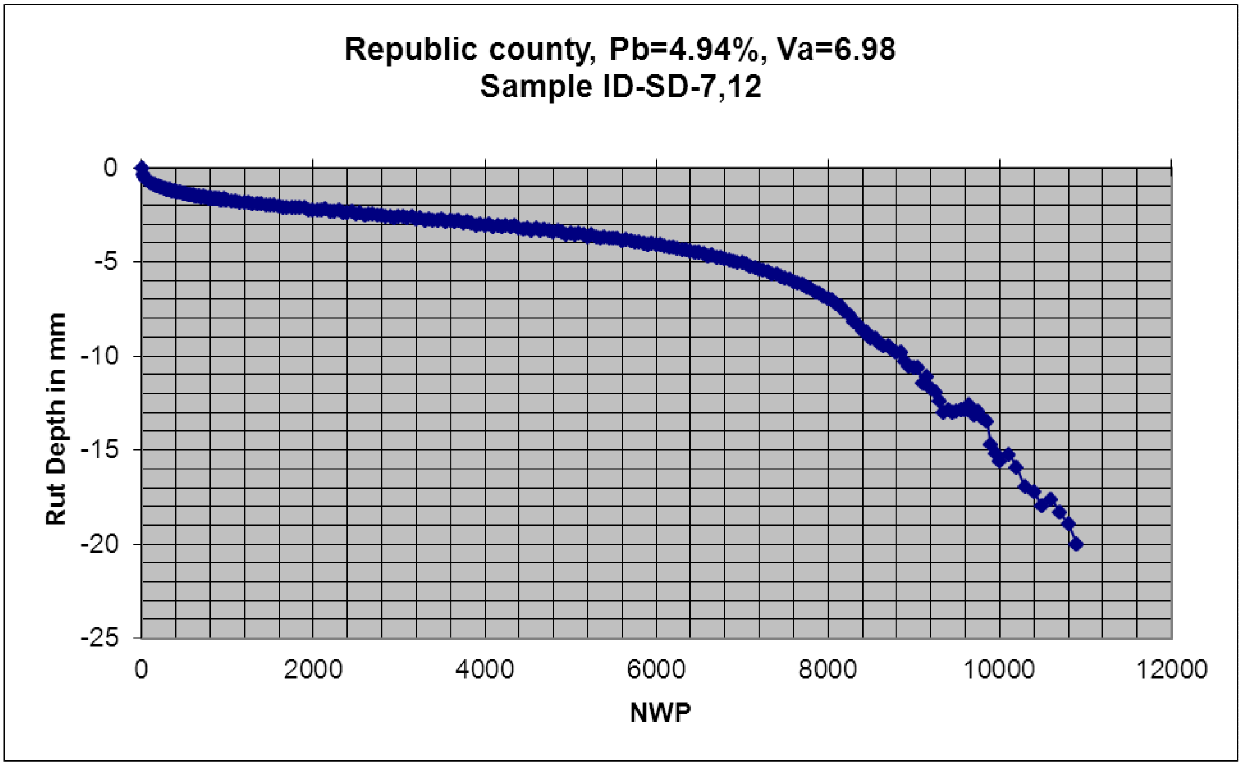
Republic county, Pb=4.94%, Va=6.98
Sample ID-SD-7,12
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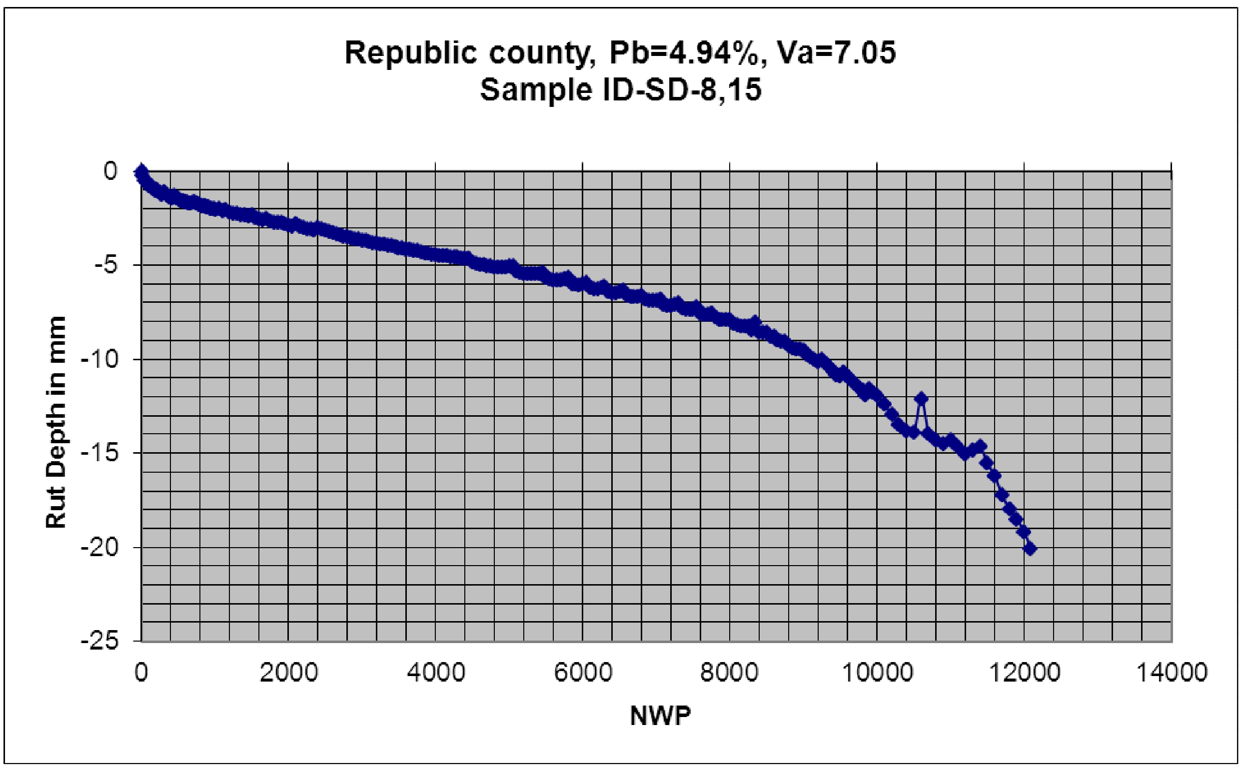
Republic county, Pb=4.94%, Va=7.05
Sample ID-SD-8,15
Rut Depth in mm
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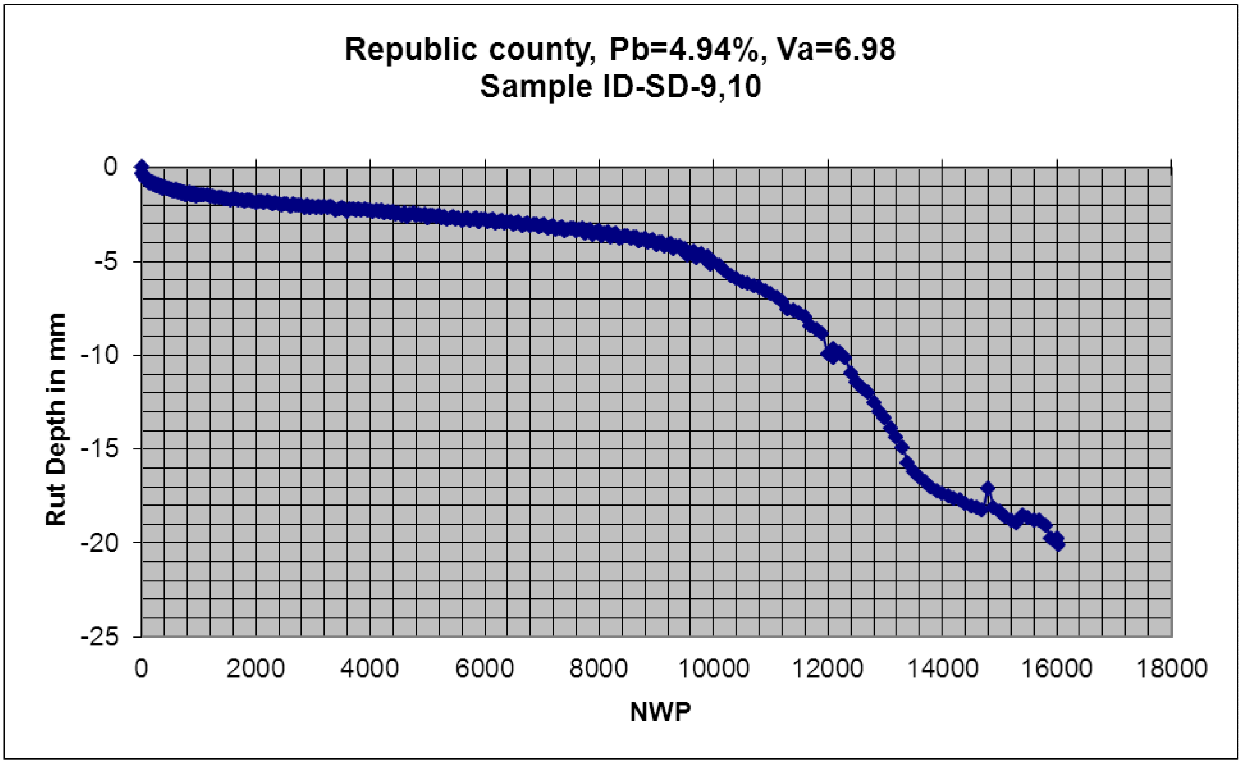
Republic county, Pb=4.94%, Va=6.98
Sample ID-SD-9,10
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-15
-20
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Rut Depth in mm
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6000
8000
10000
12000
14000
16000
18000
NWP