ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

Effects of Aggregate and Mixture Properties on the Rutting Performance of HMA Wearing Course

(A Case Study on the Gohatsion - Dejen Trunk Road Segment)

A Thesis submitted to

The School of Civil and Environmental Engineering

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By
Abraham Efraem

Advisor
Ephrem G/Egziabher (Eng.)

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M.Sc. Thesis on

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Abraham Efraem Hagoss

Addis Ababa Institute of Technology
School of Civil and Environmental Engineering

Approved by board of examiners

Ephrem G/ Egziabher (Eng.)	……………………….	…………………….
Advisor	Signature	Date
……………………….	………………………	…………………….
External Examiner	Signature	Date
……………………….	……………………..	…………………….
Internal Examiner	Signature	Date
……………………….	……………………	…………………….
Chairman	Signature	Date

DECLARATION

I certify that this research work titled “Effects of Aggregate and Mixture Properties on the Rutting Performance of HMA Wearing Course (A Case Study on the Gohatsion - Dejen Trunk Road Segment)” is my own work. The work has not been presented elsewhere for assessment and award of any degree or diploma. Where a material has been used from other sources, it has been properly acknowledged/ referred.

Abraham Efraem	………………………..	……………………
Name	Signature	Date

ABSTRACT

It is known that the availability of a road network of an appropriate level of service and quality is vital in facilitating the overall economic development of a country. In this regard, Ethiopia, a country which is on the fast- track of rapid economic development, obviously requires a road network which is able to cater for the vast traffic/transport demand.

In recent years, the quality of many of the major trunk roads in the country has been compromised, as the roads have experienced deterioration, which has given rise to the need for periodic maintenance and rehabilitation. One of the major types of distresses observed on major sections of the trunk roads is permanent deformation in the form of rutting. Performance of roads with regards to ride-ability and roughness is known to have a considerable cost implication to the road users in terms of operational cost, in addition to affecting their safety and comfort . Permanent deformation, in this regard has a detrimental effect.

Rutting in paved roads can be attributed to various factors such as the pavement structure, quality of individual constituent pavement materials, magnitude and regime of loading, environmental factors, such as moisture temperature, and others. Despite the fact that HMA mix design has evolved from the conventional empirical mix design approaches (Marshall and Hveem), to the state-of-the-art and more advanced Superpave procedures, due to the lack of technological advancement, the customary Marshall mix design and testing method, bearing the effects of its empiricism, is what is currently being utilized to design ' better' performing HMA mixtures.

This research focuses on assessing the effects of aggregate and mixtures properties on the rutting performance of HMA wearing (surface) course mixtures, by attempting to establish correlations between the independent variables (HMA aggregate and mixture properties) and the dependent variable (rut depth), still adopting the fairly simple and readily available Marshall mix design and testing procedure . To meet the objectives of the study, various tests were conducted on HMA core samples, obtained from a pertinently rutted trunk road segment, selected for the case study -Gohatsion-Dejen Road Segment. Statistical correlation analysis coupled with subjective evaluation was then adopted to assess the effects of the various aggregate and volumetric parameters on the rutting performance of HMA mixtures.

The results of the study revealed that mixes with higher air void content (in-place and re-compacted), lower voids filled with asphalt(in-place and re-compacted), lower Marshall flow value and higher coarse aggregate angularity generally perform better with regards to rutting. It was also found that 0.45 Power Gradation plots give an important indication on the effect of aggregate gradations on the rutting performance of mixtures, and that mixes with aggregate gradations, which plot on the coarser side of the maximum density line, generally perform better with regards to rutting. On the other hand, the inadequacy of the mere 75 blow Marshall

Compaction approach to simulate traffic loading in excess of one million ESA's, for representing the current levels of high traffic loadings was also underlined. Moreover, it was noted that the legal load limit of the country, which is 10 tonnes axle loading, is being exceeded, which would prove to have a considerable effect on the rutting performance of mixtures.

Finally, suggestions were made on how to design HMA mixtures with better rutting resistance, still adopting the customary Marshall mix design procedure, coupled with the refusal density approach, and the 0.45 Power Gradation Chart.

Key words: *Rutting, HMA, Re-compacted, Stability, Flow, Angularity, Voids, Va, VMA, VFA, stripping Marshall Mix Design, 0.45 Power Gradation Plot, Marshall Compaction, Refusal Density, Axle loading*

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ABREVIATIONS

HMA	Hot Mix Asphalt
Va	Air Void content
VMA	Voids in Mineral Aggregate
VFA	Voids Filled With Asphalt
ESA	Equivalent Standard Axles
ESAL	Equivalent Standard Axle Load
LWT	Loaded Wheel Tester
GLWT	Georgia Loaded Wheel Tester
APA	Asphalt Pavement Analyzer
HWTD	Hamburg Wheel Tracking Device
MMLS3	One-third Scale Model Mobile Load Simulator
PI	Penetration Index
FHWA	Federal Highway Administration of the U.S.
AC	Asphalt Cement
BBR	Bending Beam Rheometer
DSR	Dynamic Shear Rheometer
PAV	Pressure Aging Vessel
RTFO	Rolling Thin-Film Oven
DSR	Dynamic Shear Rheometer
DTT	Direct Tension Tester
WC	Wearing Course
BC	Binder Course

ERA Ethiopian Roads Authority

MDL Maximum Density Line

1. INTRODUCTION

1.1. Background

It is known that the availability of a road network of an appropriate level of service and quality is vital in facilitating the overall economic development of a country. In this regard, Ethiopia, as a country which is on the fast- track of rapid economic development, obviously requires a road network which is able to cater for the vast traffic/transport demand. To this end, the Ethiopian Roads Authority(ERA) is expediting the construction of new roads and rehabilitation/upgrading of existing roads of varying functional classification.

However, in recent years, the quality of many of the major trunk roads in the country has been compromised, as many of the roads have experienced deterioration, which has given rise to the need for the periodic maintenance and rehabilitation. One of the major types of distresses observed on major sections of the trunk roads is permanent deformation in the form of rutting. An increase in the severity and extent of permanent deformation (rutting) over considerably visible stretches is observed on many of the existing trunk roads.

Rutting in paved roads is defined as the formation of twin longitudinal depressions along the wheel paths mainly caused by progressive movement of materials due to repeated loading. Depending on the magnitude of the traffic load and the relative strength of the pavement layers, rutting can occur in the upper HMA wearing course, base or subgrade layer or in a combination of these layers. Permanent deformation in paved roads can be attributed to various factors such as the pavement structure, quality of individual constituent pavement materials, magnitude and regime of loading, environmental factors, such as moisture temperature, and others.

Performance of roads with regards to ride-ability and roughness is known to have a significant cost implication to the road users in terms of operational cost, in addition to affecting the safety and comfort of the road users. Permanent deformation, in this regard has a detrimental effect. A wide variety of equipment and procedures have been developed and used to assess rutting characteristics of HMA mixes in the laboratory. These include: the traditional Marshall and Hveem tests, uniaxial and triaxial static and dynamic creep tests, and the Superpave direct shear test (Collins et al., 1995). Performance testing has been deemed necessary for a broad acceptance of the Superpave mix design system. Researchers have sought for a simple and yet reliable testing procedure to assess rutting potential of HMA. Currently, the most common type of laboratory equipment of this nature is a loaded wheel tester (LWT). Several LWTs currently are being used around the world. They include the Georgia Loaded Wheel Tester (GLWT), Asphalt Pavement Analyzer (APA), Hamburg Wheel Tracking Device (HWTD), LCPC (French) Wheel Tracker, Purdue University Laboratory Wheel Tracking Device (pURWheel), and one-third scale Model Mobile Load Simulator (MMLS3) (Cooley L. A. et al., 2001).

Despite the recent trends to resort to the Superpave mix design and performance testing method, the Marshall method is the most widely used method around the world and in our country to establish optimum asphalt contents of HMA mixes based on the concept of stability (resistance to deformation). However, as this method is not based on fundamental engineering properties nor has been validated in the field to predict rutting in HMA pavements, it is not possible to obtain a direct indicative correlation of which HMA property contributes most to rutting. Consequently, researchers elsewhere in the world have used various types of creep tests for laboratory evaluation of HMA permanent deformation (Collins et al., 1995). Moreover, as has been discussed above, mix design has evolved from conventional Marshall design to the superpave design and beyond, as it has become increasingly important to identify practical laboratory test methods to predict the performance of HMA pavements in this regard.

In our country Ethiopia, owing to the lack of technological advancements to implement such advanced laboratory tests for the evaluation/prediction of rutting, there are no direct laboratory procedures practiced so far, which give a direct indication on the effects of the different HMA material and mixture properties on rutting. However, it is evident that, to be able to design mixtures that have adequate resistance to rutting, the effect of mixtures' volumetric composition and properties of the component materials, on their permanent deformation response must be clearly understood.

1.2. Problem Statement

In recent years, many of the major trunk roads of the country have experienced an increase in the severity and extent of permanent deformation (rutting) over considerably visible stretches. The quality and rideability of trunk roads is so vital in that such roads provide linkage amongst the most economically and geographically critical regions of the country .

Performance of roads with regards to ride-ability and roughness is known to have a significant cost implication to the road users in terms of operational cost, in addition to affecting their safety and comfort. Permanent deformation, in this regard has a detrimental effect. Permanent deformation in paved roads can be attributed to various factors such as the pavement structure, quality of individual constituent pavement materials, magnitude and regime of loading, environmental factors, such as moisture temperature, and others. One of the most common design considerations used to address the quality of HMA mixtures with regards to future performance is the marshal mix design method. The individual material properties, HMA mix volumetrics, strength interms of marshal stability, and resistance to deformations in terms of flow are determined by employing the Marshal Mix Design method, to ensure acceptable future performance.

Several field studies have been undertaken so far throughout the world to try to identify (correlate) hot-mix-ashpalt (HMA) material and mixture properties and/or design parameters that relate to rutting. Several of these studies were large in scope and involved extensive field sampling and laboratory testing. However, as far as the scope of the literature review conducted in this study is concerned, no such researches have been conducted so far in our country. And taking into consideration the fact that rutting is a commonly observed form of distress on many of the major trunk roads of the country, establishment of correlations between the various aggregate and mixture properties of HMA mixtures and rutting, on the major trunk roads of the country is vital, in that it can create an understanding on the basic root causes of the problem, and can be instigative for further assessment and identification of the remedial measures.

1.3. Research Objective

The main objective of the study is to determine the effect of the various aggregate and mixture properties on the rutting performance of HMA mixtures, still adopting the fairly simple and readily available mix design and testing procedures. This is expected to be achieved through statistical correlation analysis coupled with subjective evaluation of the different contributing parameters (independent variables) and rutting expressed in terms of rut depth (dependent variable). The results of the study could then create a conducive ground for further assessment of counteractive measures.

1.4. Scope and Limitations

The research focuses on establishing correlations between rut depth and the various aggregate and mixture parameters of HMA, through a case-study. For this purpose, a pertinent severely and early-rutted trunk road segement, Gohatsion-Dejen road, has been selected for the case study. Field sampling (obtaining 4 inch diameter cores) and rut depth measurements have been conducted at reasonable and affordable extents, on sections (spots) on which the observed rutting has been deemed (and verified) to be in the HMA wearing course. Moreover, available laboratory and field investigation equipments have been employed to obtain representative results such that plausible correlation results could be obtained.

The work is limited to the, equipment, instruments and workmanship available. To this end, among the various procedures available elsewhere in the world to assess/predict the rutting potential of HMA mixtures, the Marshal mix design and testing method, along with the other various relevant test procedures, has been adopted as the primary method to assess the rutting characteristics of the in-place HMA cores and re-compacted mixes in the laboratory.

1.5. Organization of the Thesis

The report contains five chapters with thirteen appendices. After introducing the objective of this research in chapter 1, chapter 2 comprises of a comprehensive literature review regarding the details of the subject. Here, the definitions, mechanisms and causes of rutting in HMA pavements and mixtures have been thoroughly discussed. The different material and mixture properties that influence rutting in HMA pavements, with focus given to the Marshall method of asphalt mixture design and testing have been methodically covered. In chapter 3, the general methodology adopted as well as the sampling and test procedures followed during the process of this research have been presented. Moreover, findings regarding the construction history, traffic data and pavement section details have been discussed in chapter 3. Chapter 4 includes the discussion and analysis of the data and laboratory test results. In this chapter, the different laboratory test results have been thoroughly discussed and rut depth has been correlated with the various aggregate and mixture properties of HMA. Finally, the conclusions that were drawn from the findings in chapter 4 and relevant recommendations are discussed in chapter 5.

2. LITERATURE REVIEW

The literature review covers the most significant parts of the subject , and is intended to serve as an introductory input to the subsequent analysis stages of the project, by seeking to improve the knowledge about permanent deformation (rutting) and its contributing factors. Unfortunately, it was not possible to find researches conducted in our country on the subject area. However, a conscious effort has been made to encompass the efforts through the researches accomplished so far elsewhere in the world, and to interpret and utilize the information obtained in the context of the situation in Ethiopia.

2.1. Rutting Defined

Rutting is a longitudinal surface depression in the wheel path accompanied, in most cases, by pavement upheaval along the sides of the rut. Pavement rutting which results in distorted pavement surface is the accumulation of permanent deformation in all or a portion of the layers in a pavement structure. Longitudinal variability in the magnitude of rutting causes roughness. Water may become trapped in ruts resulting in reduced skid resistance. Increased potential for hydroplaning and spray that reduces visibility. Progression of rutting can lead to cracking and eventually complete disintegration.. Repetitive application of heavy trucks with increasingly high pressure tires drives rut formation in high quality layers. The stresses induced near surface layers by the high pressure tires may exceed the ability of the materials to resist densification below critical void levels and subsequent densification (Parker, F. and Brown, E.R, as edited by Richard C. M., 1992). Rutting can occur in all layers of the pavement structure and generally results from lateral distortion and densification. Moreover, rutting represents a continuous accumulation of incrementally small permanent deformations from each load application (Rabbira Garba, 2002).

Rutting in asphalt concrete pavement usually appears as a longitudinal depression under the wheel paths of vehicles and a small bulging on the sides. The extent of rutting gradually accumulates with increasing numbers of wheel load applications on the pavement (Haroon and Jim, 1997). Another researcher Asiamah (2002) defined Permanent deformation in the form of rutting, as an unrecoverable deformation visible as a depressed channel in the wheel path of the roadway. According to this researcher, it is a progressive movement of materials under static or cyclic loads either in the top (asphalt) layer or the underlying layers.

2.2. Mechanisms of Rutting

Eisemann and Hilmar (1997) studied asphalt pavement deformation phenomenon using wheel tracking device and measuring the average rut depth as well as the volume of displaced materials below the tires and in the upheaval zones adjacent to them. They concluded that:-

1. In the initial stages of trafficking, the increase of irreversible deformation below the tires is distinctly greater than the increase in the upheaval zones. Therefore, in the initial phase, traffic compaction or densification is the primary mechanism of rut development.

2. After the initial stage, the volume decrease below the tires is approximately equal to the volume increase in the adjacent upheaval zones. This indicates that most of the compaction under traffic is completed and further rutting is caused essentially by shear deformation, i.e., distortion without volume change. Thus, shear deformation is considered to be the primary mechanism of rutting for the greater part of the lifetime of the pavement.

If the asphalt pavement structure is weak and large stresses and strains are induced in all the pavement layers under heavy wheel loads, initial densification and subsequent shear flow can be developed in various pavement layers. Under such a situation, all pavement layers contribute to total surface rutting of the asphalt pavement. The rut depth under the wheel paths will be the sum of the permanent deformations of all the pavement layers (Haroon and Jim, 1997).

Asiamah (2002), classifies mechanisms of rutting by processes through which it occurs as, densification, shoving, and raveling.

- **Rutting by densification :-**Densification rutting occurs when there is additional compaction in the pavement surface or in any of the underlying layers (base, subbase or subgrade) after the road is open to traffic. The surface may undergo further compaction under traffic loading resulting in rutting when compaction is inadequate during construction of the pavement..

- **Rutting by raveling :-**This type of rutting is caused by the loss of material in the wheel path. Dislodgement of individual aggregate particles under the action of tires occurs when there is inadequate compaction, low asphalt content or excessive aging of the asphalt binder, which usually result in loss of adhesion between the aggregates and the asphalt binder. Ruts caused by raveling tend to be dry, ragged looking and non-uniform. Rut resulting from the loss of surface material may also be due to abrasion. In this situation aggregate particles wear out if traffic conditions are too abrasive or the aggregates are soft. These ruts are continuous with more resistant aggregate particles exposed and sticking out in the wheel paths .

- **Rutting by shoving :-**At low air void contents (less than 4%) shear deformation may occur within the asphalt mixture under traffic loading. In this situation pavement material is laterally displaced along shear planes within the mixture, which shows signs of mixture instability. Shoving may be transverse or longitudinal. The rut is usually seen as depression in the loaded area in the wheel path and ridges appear along both edges of the wheel paths. Shear deformation is usually caused by lack of resistance to shear stresses generated in the pavement surface from tire pressures.

Lekarp et al. (1996) in their study on influences on permanent deformation behavior of unbound granular materials, described that flexible pavement carries load in shear deformation. According to this study, an element of HMA layer subjected to traffic loading transfers the load from the surface to the underlying layers through inter-granular contact and resistance to flow of the binder matrix.. According to Lekarp et al. (1996), permanent deformation is generally considered to be the result of three mechanisms: consolidation, distortion, and attrition.

- **Distortion :-**Bending of flat particles, sliding and rolling of rounded grains are considered to be distortion. HMA materials flow laterally due to loss of interlocking of contracting particles, rather than densification. This type of rutting is mainly caused by an asphalt mixture with very low shear strength to resist the repeated heavy loads to which it is subjected .

- **Consolidation:-** The change in shape and compressibility of particle assemblies is considered as consolidation. Volume changes due to changes in grain arrangements, particle orientation, and generalized contraction of the assembly without modification of the soil structure. Rutting caused by densification of high air void mixtures are usually not considered during initial mix design It is assumed that good engineering and construction practices will be followed, proper compaction will be achieved on the roadway. However, at high air void levels, one-dimensional densification can be a problem. Consolidation type rutting normally occurs in subgrade, subbase, or base below the asphalt layer. Although stiffer paving materials will partially reduce this type of rutting, it is normally considered more of a structural problem rather than a materials problem. It is often the result of a too thin pavement section because there is simply not enough depth of cover on the sub grade to reduce the stress from applied loads to a tolerable level. It may also be the result of a sub grade that has been unexpectedly weakened by the intrusion of moisture.

- **Attrition:-**The change in a material's fabric and packing is considered an attrition. It is due to crushing and breakage of particles, particularly at inter-particle contact points. Permanent deformation can continue as long as attrition occurs in a granular assembly.

Another researcher Regis L. E. de Carvalho (2012), describes that two mechanisms are the main causes of rutting development. According to this study, compaction is the primary mechanism at initial stages of loading. Compaction (i.e., densification) occurs as the material volume decreases underneath the wheel path with no significant upheaval along the sides of the wheel path. After this initial stage is complete, further volume decrease of material beneath the wheel path at each load application approximately equals the volume increase in the upheaval along the sides. This deformation mode is essentially caused by shear (i.e., distortion without volume alteration). When enough distortion has occurred, the asphalt concrete undergoes shear flow and deformations increase rapidly at an increasing rate termed tertiary flow. Figure 2.1 depicts rutting development versus load application in which region 1 is mainly caused by material densification, region 2 is predominantly shear deformations, and region 3 is tertiary flow to shear failure. The primary stage, represented by region 1 in Figure 2.1, happens early on in the pavement's service life, usually within the first year. The pavement will probably be rehabilitated prior to reaching the tertiary stage (region 3) due to rutting already reaching the agency's threshold or another distress triggering the need for maintenance.

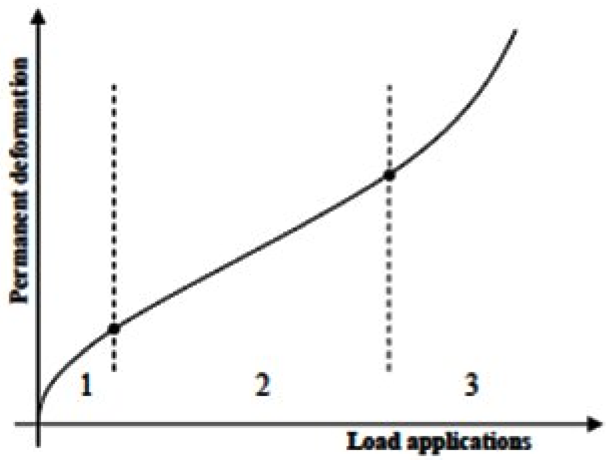

Figure 2-1 Rutting development versus load applications (Regis L. E. de Carvalho (2012)

Another research by Parker, F. and Brown, E.R, as edited by Richard C. M., (1992), came up with a model that describes the rutting of asphalt concrete as a two phase process as illustrated in Figure 2.2.

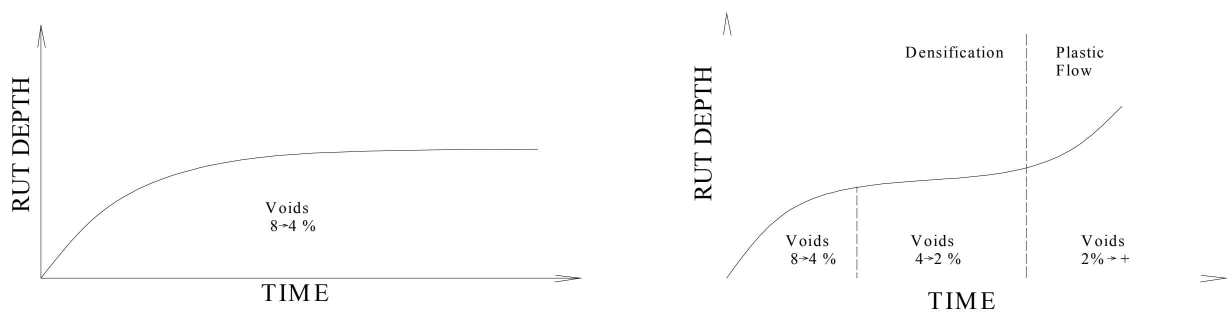

Figure 2-2 Model for rut development in asphalt concrete (Parker, F. and Brown, E.R, as edited by Richard C. M., 1992)

In the first phase repeated load applications cause densification from as constructed void content(8% or less) . In properly designed mixes densification stabilizes at about 4% and rut depth development ceases or decreases to very low rates as illustrated in figure 2.2. Densification may stabilize at higher voids, but if the voids are much higher than 4%, problems with durability may develop. At about 4% voids, the ability to resist permanent deformation in properly designed mixes is optimum. At this stage, it is critical that the aggregate skeletal structure have the ability to resist further densification, and this is best accomplished with well graded aggregate with angular rough textured particles. Asphalt content is also critical as the mix reaches about 4% voids. Excess asphalt will decrease inter-granular contacts, weakening the aggregate skeletal structure and leading to further densification. Excess asphalt can weaken otherwise very stable aggregate structures. This emphasizes that aggregate properties and optimum asphalt content are equally important aspects of the mix design and construction. For pavements that experience severe rutting, densification constitutes and second phase conditions develop. When voids reach about 2%, the mix becomes very unstable and plastic flow develops., as illustrated in figure 2.2. Rut depth increases rapidly and upheaval outside wheel paths begins. Carried to extremes, pushing and shoving may develop causing a dramatic increase in roughness. Dilation may occur as the material shears and flows plastically from wheel paths and may cause an apparent increase in voids (Parker, F. and Brown, E.R, as edited by Richard C. M., 1992).

2.3. Causes of Rutting

Generally there are three causes of rutting in asphalt pavements: accumulation of permanent deformation in the asphalt surfacing layer, permanent deformation of subgrade or underlying layers, and wear of pavements caused by studded tires. In the past subgrade deformation was considered to be the primary cause of rutting and many pavement design methods applied a limiting criteria on vertical strain at the subgrade level. However recent researches indicate that most of the rutting occurs in the upper part of the asphalt surfacing layer. These three causes of rutting can act in combination, i.e., the rutting could be the sum of permanent deformation in all layers and wear from studded tires (Rabbira Garba, 2002).

2.3.1. Rutting Caused by Weak Asphalt Mixture

Rutting resulting from accumulation of permanent deformation in the asphalt layer is considered to be the principal component of flexible pavement rutting. This is because of the increase in truck tire pressures and axle loads, which puts asphalt mixtures nearest the pavement surface under increasingly high stresses. Brown and Cross (1992) conducted an extensive national study of rutting in hot mix asphalt pavements, in United States . The conclusion from this study regarding the location of rutting was that the majority of rutting was occurring in the top 3 to 4 inches (75 to 100 mm) of the asphalt concrete layers. They found that the rutting in the subgrade was generally very small. According to this study, to reduce this form of deterioration, it was recommended that it is necessary to pay more attention to the selection of materials and mix design; and to be able to design mixtures that have adequate resistance to rutting, the effect of mixtures' volumetric composition and properties of the component materials on their permanent deformation response must be clearly understood.

Rutting in asphalt layers is caused by an asphalt mixture that is too low in shear strength to resist the repeated heavy loads to which it is subjected. Asphalt pavement rutting from weak asphalt mixtures is a high temperature phenomenon, i.e., it most often occurs during the summer when high pavement temperatures are evident (Rabbira Garba, 2002). Figure 2.3 illustrates rutting caused by weak asphalt mixture.

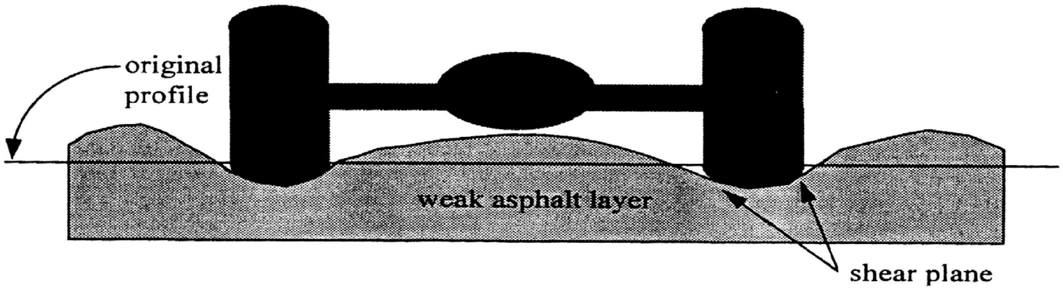

Figure 2-3 Rutting caused by weak asphalt layer, (Rabbira Garba,2002)

As mentioned above, permanent deformation in asphalt concrete consists of densification and shear deformation. Shear deformation occurs with no change in volume, ,i.e., it is distortional. Asphalt concrete may also dilate or increase in volume under load. Deformation involving dilatancy is also referred to as shear flow or plastic flow in some literatures. Such deformation can lead to debonding at the binder aggregate interface and deterioration of the pavement. Figure 2.4 illustrates the mechanisms of rutting in asphalt layers.

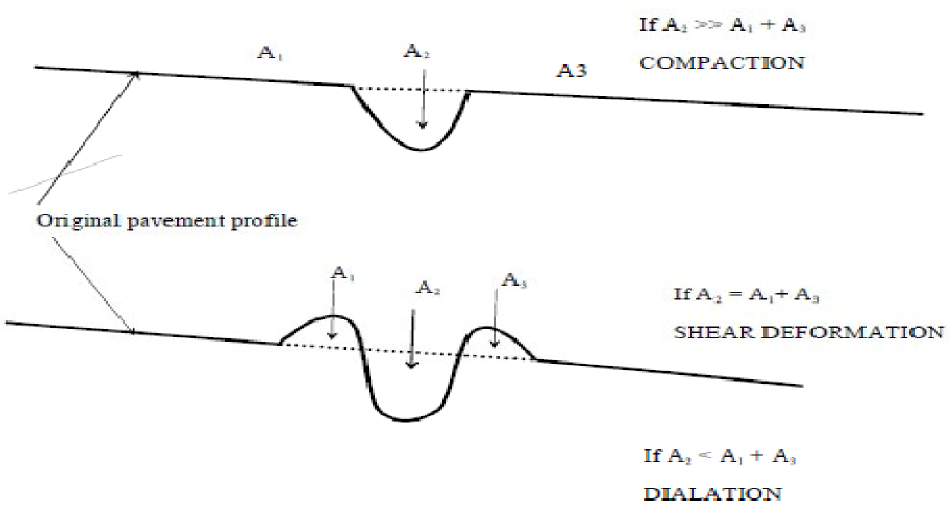

Figure 2-4 Illustration of the rutting mechanism (Rabbira Garba, 2002)

Asphalt concrete pavements are usually constructed at initial air void content of 7-8%. It is anticipated that further compaction of the pavement will occur under traffic to around 4% air voids, after which conditions may stabilize. Densification, in general, is not a problem if the asphalt surface is uniformly compacted by traffic. However, with channelized traffic flow, most of the densification occurs in the wheel path, creating longitudinal ruts (Asiamah, 2002).

2.3.2. Rutting Caused by Weak Subgrade or Underlying layer(s)

Rutting can be caused by too much repeated load applied to subgrade, subbase or base, below the asphalt layer. In many cases this is due to insufficient depth of cover on the subgrade resulting from too thin an asphalt section to reduce the stress from applied loads to tolerable level. Thus this type of rutting is considered to be more of a structural problem than a materials problem and is often referred to as structural rutting. Intrusion of moisture can also be the cause for weakening of the subgrade. In this type of rutting, the accumulated permanent deformation occurs in the subgrade. Figure 2.5 illustrates rutting from weak subgrade (Rabbira Garba, 2002).

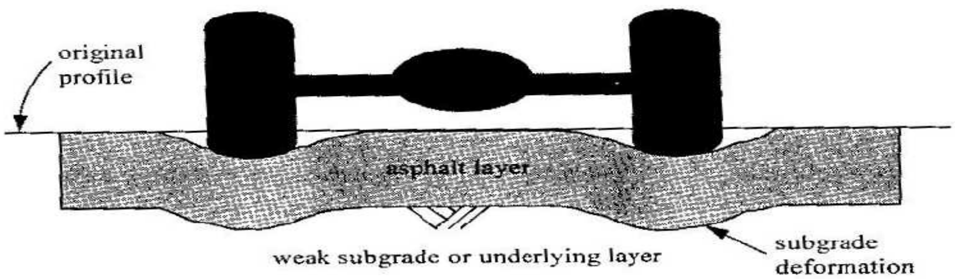

Figure 2-5 Rutting from weak subgrade (Rabbira Garba, 2002)

The base or subbase may undergo further compaction resulting in rutting of the pavement surface when there is inadequate compaction of these layers during construction or when the pavement surface is under designed or when there is poor subsurface drainage. The subgrade may also undergo compaction resulting in rutting when there is inadequate pavement structure above it to reduce the subgrade vertical stress/strain to allowable limits. Weak and yielding layers below the pavement structure may subside from traffic loading resulting in subsidence ruts. These tend to be fairly wide (750-1000 mm) with a shallow sloping saucer shape cross section (Asiamah, 2002).

2.3.3. Rutting Caused by Pavement Wear

The studded tires, used in Nordic countries, cause significant wear of the pavements, which results in longitudinal depression in the wheel path. Because of this, wear resistant mixtures, which are usually of high binder content and low void content are usually specified for high volume roads in such areas. But this kind of mixtures are also susceptible to shear , and hence, in such situations, observed rutting in the field would most probably be the combined effect of wear and permanent deformation (Rabbira Garba, 2002). However since this type of rutting is not an issue in our country, it will not be discussed in detail.

Figure 2-6 Rutting caused mainly by studded tire wear (Rabbira Garba, 2002)

2.3.4. Identification of Cause of Rutting

In General the total rutting in pavements is the combination of accumulated permanent deformation in all layers in the pavement structure. Forensic trenches are the preferred approach for determining the permanent deformation in each layer. However, it is very costly and destructive to do so. Some researchers suggest that the shape of the surface profile can indicate which layer is responsible for the failure of the pavement structure due to rutting. Figure 2.7 shows three transverse profiles typical of three different scenarios in which majority of rutting comes from (a) asphalt concrete surface layer, (b) granular base and (c) subbase/subgrade. If the majority of rutting originates in the underlying unbound base and subbase layers, little or no heave is observed. When the asphalt concrete layer is responsible for total rutting, heave is observed. In extreme situations of very stiff underlying layers – e.g., composite pavement with a Portland cement (PCC) slab acting as a base layer – the heave may dominate the settlement portion. The majority of the failures from rutting are due to excessive deformation in the asphalt concrete layer (White et al., 2002).

a)

b)

c)

Figure 2-7 Effects of rutting concentration in different layers on permanent deformation surface profile of flexible pavements: (a) asphalt concrete, (b) granular base, and (c) subbase/subgrade (White et al., 2002).

As discussed in the previous section, (Asiamah (2002) indicated that weak and yielding layers below the pavement structure may subside from traffic loading resulting in subsidence ruts, and that these ruts tend to be fairly wide (750-1000 mm) with a shallow sloping saucer shape cross section. (Rabbira Garba, 2002) as discussed above has also tried to explain the difference between the rutting caused due to weak asphalt mixtures and underlying layers based on the shape of the surface profile and heave phenomenon. Many other approaches have been taken to address the prediction of rutting and its causes.. One such analytical opportunity explored suggests that the area under the transverse profile can be used to hypothesize the origin of the rutting from within the pavement structure. (Simpson et al., 1995**)**.

2.4. HMA Material and Mixture Properties That Influence Rutting

2.4.1. General

Hot-mix asphalt is a material composed of aggregates and asphalt binder. Ideally the aggregate skeleton should be capable of supporting and carrying the traffic loads applied to the mixture if it is sufficiently contained and kept bonded together at all times. Since the aggregate particles are not very cohesive the asphalt binder acts as a glue or a bonding material to keep the aggregate skeleton together. However, the asphalt binder must be sufficiently strong to resist excessive shear loads generated between the aggregate particles. If the binder is not strong enough, especially in hot weather, rolling tires can dislodge aggregate particles and shear deformation may easily occur (Huber G.A., 1999). The selection of the right aggregate structure and the choice of the most appropriate binder having the required properties are therefore important for HMA to resist rutting. Some of the aggregate properties that influence shear properties and therefore, rut resistance are particle shape, texture and crushed faces and gradation. The viscosity or the stiffness is also a property of the asphalt binder that affects rut resistance; a higher viscosity or stiffer binder, especially at higher temperatures, results in higher rut resistance. Asphalt content, dust to asphalt ratio or percent of mineral filler and film thickness are also some of the properties of the mixture that have been found to affect mixture rutting potential and performance. (Nukunya B. et al.,2001).
Other volumetric properties of the asphalt mixture may also affect rutting, such as voids in the mineral aggregate (VMA) , voids filled with asphalt (VFA), and air voids (Va). By providing

appropriate VMA, it is believed that rutting may be minimized, and mixture durability will be enhanced. Compaction characteristics, characterized by the rate of densification during laboratory compaction, can also provide information relative to a pavement's resistance to permanent deformation. In theory, the greater the compactive effort required to compact a sample, the greater its shear resistance. (Stacy G., 2002).

In short, Permanent deformation of asphalt-aggregate mixes is a complex phenomenon where aggregate, asphalt and aggregate-asphalt interface properties control the overall performance Furthermore, overtime these properties change until the mix reaches the end of its useful life.

2.4.2. Aggregate Properties

The largest portion of the resistance to permanent deformation of the mixture is provided by the aggregate structure. Aggregate is expected to provide a strong, stone skeleton to resist repeated load applications. Gradation, shape, and surface texture have a great influence on HMA properties. Angular, rough-textured aggregates provide more shear strength than rounded, smooth-textured aggregates. When a load is applied to the aggregate in an asphalt mixture, the angular, cubical, rough-textured aggregate particles lock tightly together and function as a large, single elastic mass, thus increasing the shear strength of the asphalt mixture. Conversely, instead of locking together, smooth, rounded aggregate particles tend to slide past each other. If the aggregate provides a high degree of internal friction, the shear strength of the asphalt mixture will be increased and, therefore, the resistance to rutting. This is accomplished by selecting an aggregate that is angular, cubical, has a rough surface texture, and is graded in a manner to develop particle to particle contact (Arif C. et al., 2001).

T. W. Kennedy et al., (1996) stated that, in order to prevent permanent deformation of HMA pavements :-

- one should avoid gradations near the maximum density because, although they theoretically produce the strongest HMA mixtures, due to their relatively low voids in the mineral aggregate, these types of mixtures are very sensitive to asphalt content and present the risk of flushing due to inevitable variations during construction.

- It is better to use aggregates with angular particles because they exhibit greater interlock and internal friction and, hence, result in greater mechanical stability than rounded particles.

- It is better to use aggregates with rough surface texture because they tend to form stronger mechanical bonds when compared to smooth-textured aggregates and provide higher VMA in a compacted mass.

2.4.2.1. Gradation

Gradation is perhaps the most important property of an aggregate. It affects almost all the important properties of HMA, including stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance, and resistance to moisture damage. Therefore, gradation is a primary consideration in asphalt mix design, and the specifications used by most agencies limit the gradations that can be used in HMA (Rafiqul A. T. and Musharraf Z., 2005). Generally, the shear strength and hence the resistance to permanent deformation of asphalt mixtures depends on the mechanical interlock of the aggregate skeleton especially the stone structure of coarse aggregates. Loss of stability, which can lead to rutting, can in general occur when gradations containing excesses of certain size fractions are used. It is believed that, strength or resistance to shear failure, in pavements and other aggregate layers that carry loads is increased greatly if the mixture is dense graded (Rabbira Garba, 2002).

It might be reasonable to believe that the best gradation is one that produces the maximum density. This would involve a particle arrangement where smaller particles are packed between the larger particles, which reduces the void space between particles. However, some minimum amount of void space is necessary to provide adequate volume for the asphalt binder to occupy. Therefore, although it may not be the "best" aggregate gradation, a maximum density gradation does provide a common reference. A widely used equation to describe a maximum density gradation was developed by Fuller and Thompson (1907).

Their basic equation is:

$$= - \qquad \text{..(1)}$$

where: P = % finer than the sieve

d = aggregate size being considered

D = maximum aggregate size to be used

n = parameter which adjusts curve for fineness or coarseness (for maximum particle density n ≈ 0.5 according to Fuller and Thompson)

In the early 1960s, the Federal Highway Administration of the U.S. (FHWA) introduced the standard gradation graph used in the HMA industry today. This graph uses n = 0.45 and is convenient for determining the maximum density line and adjusting gradation (Roberts et al., 1996). This graph is slightly different than other gradation graphs because it uses the sieve size raised to the n^{th} power (usually 0.45) as the x-axis units.

Based on the 0.45 power gradation chart, the resistance of a mix to deformation under load can be assessed. Generally, mixes with gradations curves which plot above the maximum density line. are expected to have high resistance to deformation under load (Rafiqul A. T. and Musharraf Z., 2005). According to the study by Bouchard G.P, as edited by Richard C. M. (1992), if the grading of manufactured fines reduces V.M.A. substantially when the product tends to the coarser side of the deviation envelope, or closer to the maximum density line, a mechanism of mix difficulty emerges. When voids in any portion of a pavement are low, the pore asphalt will tend to lubricate the mix and pushing will occur. Air voids must be plentiful to allow microscopic depressions from each load to rebound to their original position.

Another dimension to the 0.45 power gradation chart was the introduction of the 'restricted zone', which resides along the maximum density gradation between the intermediate size (either 4.75 or 2.36 mm) and the 0.3 mm size. It forms a band through which gradations should not pass. as shown in figure 2.8. Gradations that pass through the restricted zone have often been called “humped gradations” because of the characteristic hump in the grading curve that passes through the restricted zone. In most cases, a humped gradation was thought to indicate a mixture that possesses too much fine sand in relation to total sand. This gradation was thought to practically always results in tender mix behavior, which is manifested by a mixture that is difficult to compact during construction and offers reduced resistance to permanent deformation during its performance life. Gradations that violate the restricted zone were thought to possess weak aggregate skeletons that depend too much on asphalt binder stiffness to achieve mixture shear strength. Moreover, These mixtures were also thought to be very sensitive to asphalt content and easily become plastic. However, other researchers have made various studies to investigate the actual effect of this restricted zone to the permanent deformation behavior of mixes, and found out that gradations that violated the restricted zone performed similarly to or better than the mixes having gradations passing outside the restricted zone; and therefore, concluded that the restricted zone requirement is redundant for mixes meeting all Superpave volumetric parameters. Arif C. et al. (2001), through their study on the effects of Superpave restricted zone on permanent deformation, concluded that:-

1. no relationship between the restricted zone and permanent deformation was found using HMA mixtures of high to relatively low quality.
2. Superpave HMA mixtures above the restricted zone were generally most resistant to permanent deformation and mixtures below the restricted zone were generally most susceptible to permanent deformation.

Based on their study Arif C. et al. (2001) recommended that the restricted zone should be eliminated from the Superpave specifications.

One thing that should be taken care of while plotting the 0.45 Power Gradation Charts, is the distinction between the nominal maximum aggregate size and the maximum aggregate size. Findings from the research by Huber,G.A., and Shuler, T.S, as edited by Richard C. M. (1992)

indicated that the maximum density line on a 0.45 Power Chart should be drawn through the origin to the 100% passing sieve size. The research emphasizes that the maximum aggregate size should be defined as th smallest sieve opening through which the entire amount of aggregate is required to pass, i.e, one sieve larger than the nominal maximum aggregate size. The nominal aggregate size is defined in the research to be the sieve size, one size larger than the first sieve to retain more than 10% of the total aggregates, in a standard set of sieves. MS-2 also stipulates that for processed aggregates, the maximum aggregate size in a standard set of sieves listed in applicable specifications is two sizes larger than the first sieve to retain more than 10% of the the material.

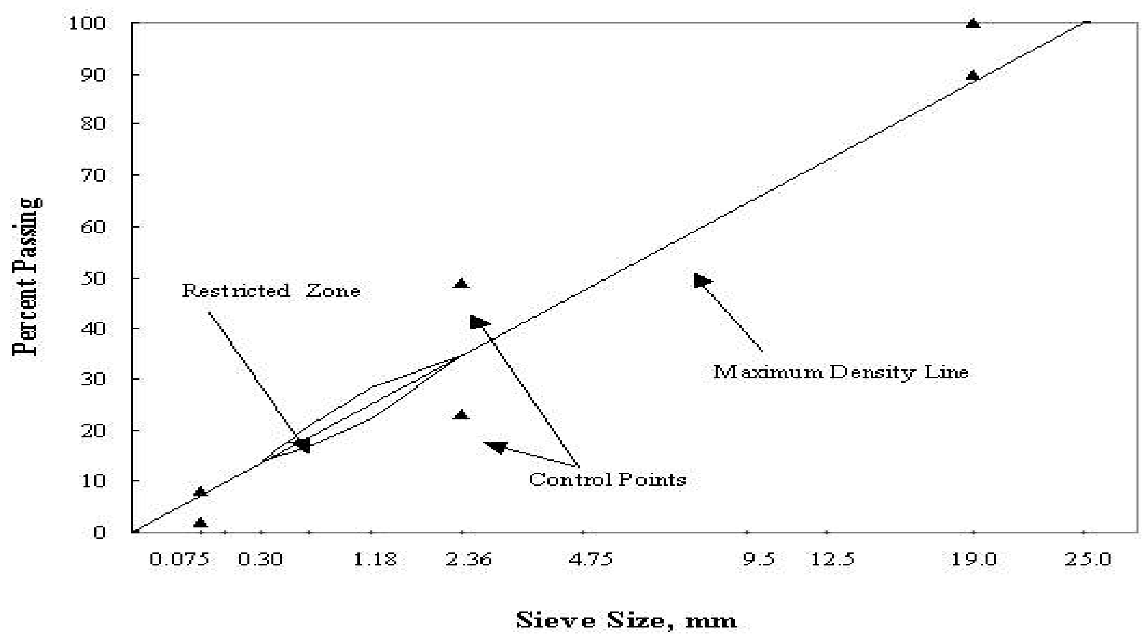

Figure 2-8 Typical Superpave Gradation Control for 19.0-mm Mixtures (Arif C. et al., 2001)

2.4.2.2. Fine Aggregates

C. Crawford (1989*)*, concluded from a study related to tender mixtures that particle shape and the amount of material passing the No. 4 sieve (4.75mm) were major factors contributing to the tenderness of an asphalt concrete mixture. He also stated that rounded, uncrushed aggregates are more likely to contribute to tender mixtures and, therefore, more rutting susceptible, especially as the amount of uncrushed material passing No. 4 sieve increases**.** B. F. Kallas and J. M. Griffith (1957) studied the influence of fine aggregates on asphalt paving mixtures and demonstrated that an increase in angularity of crushed fines increased the Marshall and Hveem stability values at the optimum asphalt content. An increase in angularity in the fine aggregate also increased the void content at a given compaction level and the optimum asphalt content. Y. H. Huang (2004)described that the contribution of fine aggregates to internal friction of HMA is quantified as percent of air voids in loosely compacted fine aggregates(smaller than 2.36mm). Higher void content in this case reflects a more textured fine aggregate.

2.4.2.3. Coarse Aggregates

The aggregate interlock and internal friction is responsible for the HMA rutting resistance. M. Yeggoni et al. (1994) conducted a laboratory study to evaluate the influence of coarse aggregate shape and texture on permanent deformation characteristics of HMA mixtures. The authors concluded that an increase in the percentage of crushed coarse aggregate resulted in increased Hveem stability, Marshall stability, and resistance to permanent deformation. They also found a strong correlation between rutting potential and the shape of the coarse aggregate particles. E. R. Brown et al., 1989) concluded that the maximum aggregate size greatly affected the pavement performance and that larger maximum aggregate sizes produce higher stability, better skid resistance, and lower optimum asphalt contents.

2.4.2.4. Aggregate Angularity

coarse Aggregate angularity is defined as the percent by weight of the aggregate particles larger than 4.75 mm with one/two or more fractured faces. A fractured face is defined as angular, rough or broken surface of an aggregate particle created by crushing, by other artificial means or by nature. Fine aggregate angularity is defined as the percent of air voids present in loosely compacted aggregate that passes the 2.36 mm sieve. Aggregate particle shape and surface texture affect the strength of aggregate particles, the bond with asphalt binder, and the resistance to sliding of one particle over another. Particles with rough, fractured faces allow a better bond with the asphalt binder than do rounded smooth gravel particles. Rough faces on the aggregate particles also allow a higher friction strength to be developed if some load would tend to force one particle to slide over an adjacent particle (Rabbira Garba, 2002). Arasan S. et al. (2010) indicated that there is a strong correlation between fractal dimension of coarse aggregates and mechanical properties of asphalt concrete. According to this researcher, a linear relationship was found between the fractal dimension and both Marshall Stability and Marshall Quotient. The effect of variables such as asphalt type, air voids, and temperature on permanent deformation are more amplified with the aggregates with smooth and polished surface, which is probably because the load bearing capacity of the mixtures with poorer interlock among round aggregates depends on the viscosity of a binder much more than the mixtures with better interlock among angular aggregates (Kim, Y.R et al, as edited by Richard C. M. (1992).

Another research by Huber,G.A., and Shuler, T.S, as edited by Richard C. M. (1992)described that mixtures with uncrushed aggregates compact into a denser arrangement than do uncrushed aggregates which have more macro texture(sharp edges) and micro texture(freshly fractured faces). The study concluded that rounded gravels produced mixtures with approximately one percent lower VMA than crushed limestone.

2.4.2.5. Aggregate toughness

Aggregate toughness is characterized using Los Angeles Abrasion Test. L.A. abrasion test (test method ASTM C 131 or AASHTO T 96) is a measure of degradation of mineral aggregates. It gives a combination of actions including abrasion or attrition, impact, and grinding for a prescribed number of revolutions in a rotating steel drum containing with a specific number of steel spheres. This test has been widely used as an indicator of the relative quality or competence of various sources of aggregate having similar mineral compositions (Rafiqul A. T. and Musharraf Z., 2005). Attrition which is the change in a material's fabric and packing is due to crushing and breakage of particles, particularly at inter-particle contact points. Permanent deformation can continue as long as attrition occurs in a granular assembly (Lekarp et al.,1996).

2.4.2.6. Filler Properties

The quality and quantity of filler used in hot asphaltic mixtures greatly affect the performance. The function of mineral filler has been recognized to be more than filling voids. Fillers usually stiffen asphalt cements variably depending on the filler type and quantity. Reducing the filler content in the mix results in an increase of VMA and is often resorted to by designers. However, this is not a wise step. Aggregate inter-particle contact provides nearly all the internal shear resistance of an asphalt mix and the material passing 75micron sieve together with the asphalt makes a major contribution to the mix cohesion. High cohesion provides the internal tensile strength and mix toughness to resist the shearing forces which cause rutting. It is therefore necessary to avoid low percentage of fillers. However excessive amounts of filler reduce the VMA, increase the aggregate surface area and reduce the asphalt film thickness thereby affecting the durability of the mix (S.K. Rao et al.,2007).

Shahrour, A.M., and Saloukeh, G.B, as edited by Richard C. M., (1992) described that filler in asphalt mixtures may act as an asphalt extender or it may stiffen the binder depending on the type of filler used. The fillers which extend the asphalt-cement effectively are increasing the asphalt-cement volume in the mix. Stiffening of mixes caused by certain types of fillers is not easy to demonstrate. However becomes quite apparent during laboratory mixing. Some fillers do stiffen laboratory mixes to the point that a great effort is required to mix them physically. stiffening in filler-bitumen mixtures can be measured by a decrease in penetration , an increase in softening point or an increase in viscosity. Fillers from different sources will produce quite different levels of stiffening. The difference in stiffening cannot be readily explained by filler size, gradation or particle shape; nor is the stiffening always reflected in the Marshal Properties of HMA. The study concluded that hydrated lime filler has shown superior stiffening properties when mixed with binder compared to all other filler types. For an addition of 0.8 ratio of hydrated lime to bitumen, the softening point increases from 51.7(for AC 60/70) to 79.4, the penetration(0.1mm) decreases from 57 to 23 at 25^0C, The PI increases from - 0.78 to +2.0. And the study finally recommended the use of hydrated lime filler in a ratio of 0.5 to 0.8 of the bitumen content as a partial substitute of the mineral filler.

E. R. Brown et al. (1989)from various laboratory and field studies, concluded that additional minus No. 200 (filler) material produced a lower optimum asphalt content (filler material fills the voids in certain asphalt mixtures and lowers the optimum asphalt content), a higher stability, and a more asphalt sensitive mixture. Some filler is required for stability, but an excessive amount (greater than 6 percent in conventional mixtures) produced unsatisfactory mixtures. Zemichael B. M. (2007), in his study on the effect of different fillers on HMA mechanical properties concluded that the dynamic modulus values of HMA mixes increased with increase in filler content. However, the stability values of mixes prepared with crushed stone and lime stone were found to increase up to a maximum and then decrease with increasing in filler content above 4%, whereas the stability values of mixes containing volcanic cinder keeps increasing with filler content. It is recommended that the broadly accepted minus 0.075mm(No.200) to asphalt ratio limits of 0.6 - 1.2 should be used . Without enough "dust" in the mixture for proper initial asphalt stiffening and dispersion, many pavements of an early age are at undue risk (Sanders, C.A. and Dukatz, E.L., as edited by Richard C. M. (1992).

2.4.3. Asphalt / Binder Properties

Bituminous binders form another important component of hot-mix asphalt mixtures. Asphalt binders are visco-elastic materials whose resistance to deformation under load is very sensitive to loading time and temperature. The empirical measures of binder properties include penetration, ductility, and softening point. This properties as empirical as they are cannot be expressed in engineering units and, therefore, can't be directly related to any of the required rheological properties of asphalt binders. They do not give an indication of whether a particular binder at the test temperature is more elastic or more viscous. To take care of this, Susceptibility parameters were proposed in an attempt to describe the visco-elastic properties of binders within the time-temperature domain. The susceptibility parameters may be grouped in to two types: Temperature susceptibility parameters and shear susceptibility parameters.. Temperature susceptibility parameters include parameters such as temperature required to change the penetration by a certain amount, slope of the logarithmic plot of penetration versus temperature, viscosity changes as a function of temperature, penetration index, penetration-viscosity-number, etc. However all the parameters still carry the problem of empiricism. most of them do not take the time dependency of asphalt binder properties in to account, and as the time and temperature dependency of asphalt is not linear, the parameters are not constant material properties. Oxidative aging and physical hardening were also considered as durability factors that cause changes in properties of binders and thus affect performance (Rabbira Garba, 2002).

The foregoing discussions indicate that conventional measures of asphalt binder properties are not capable of characterizing its fundamental rheological behavior. According to Sousa et al, (1994) the properties of the binders that influence rutting are

- Temperature susceptibility and rate of loading,
- Aging effects, and
- Moisture effects.

The emergence of the Superpave system for the design of hot mix asphalt, which is based on mehanistic-concepts to design superior performing asphalt pavements, it has been possible to account for materials characteristics in light of climatic and traffic conditions. Perhaps the most significant component of Superpave is its new asphalt binder grading system, which is designed to link with pavement performance. The asphalt binder grading system in Superpave is called performance grading (PG) system. All PG binders are characterized based upon fundamental engineering parameters. Additionally, Superpave accounts for the impact of climatic factors on binder characteristics at both hot and cold temperature regimes. The superpave binder grading tests are based upon engineering properties that control the three major modes of distress in asphalt pavements; rutting, fatigue cracking and thermal cracking. The direct tension(DT) test is intended to determine the resistance of asphalt to thermal cracking. the Bending Beam Rheometer(BBR) is designed to measure the critical stiffness(S) at which asphalt becomes brittle and susceptible to thermal cracking. The Dynamic Shear Rheometer(DSR) is the device that is used for fatigue and rutting characterization. The rheometer protocols are designed to measure elastic and damping properties of asphalt binder via the complex shear modulus, (G*). The rutting parameter is G*/sinδ where (δ) is the phase angle and is related to damping. The fatigue characterization is conducted on asphalt binder which has aged via the Pressure Aging Vessel (PAV) process, while rutting characterization is conducted on the asphalt binder that is aged using a Rolling Thin-Film Oven(RTFO) test (Y. H. Huang, 2004). In general, various pieces of equipment are used to measure stress strain relationships in the binder at the specified testing temperatures. The equipment includes the Rotational Viscometer, RV, Dynamic Shear Rheometer, DSR, the Bending Beam Rheometer, BBR, and the Direct Tension Tester, DTT (Angelo et al., 2000).

Comparison of the value of G*/sinδ for asphalt binders to the specification limit will indicate how well they will perform with respect to permanent deformation. The contribution of the asphalt binder to permanent deformation is controlled by specifying a minimum value of 2.2 kPa for the stiffness parameter, G*/sinδ parameter is correlated to that portion of the accumulated, non-recoverable deformation occurring in a pavement that is attributable to the asphalt binder (K. Wayne et al., 2010). However, as this new measures of the fundamental rheological properties of asphalt cement require complicated testing procedures and apparatus, such will not be covered under this research. Moreover, the Performance Grade, PG, binder specification (AASHTO M 320-02) is intended to select the binder to optimize its effect on the performance of the pavement. The PG binder specification is based on the rheological properties of the asphalt binder measured over wide range of temperatures and aging conditions. The PG of a binder is rated based on the maximum and minimum pavement temperatures. For example a PG 64-22 is rated to perform on pavements where the maximum pavement temperature is 64^oC or less and the minimum pavement temperature is -22^oC or higher. In this regard, Angelo et al. (2000) recommends that harder asphalt, e.g., one PG higher temperature grading be used where rutting is expected in the field.

Aging of asphalt is also an important aspect controlling the mix behavior over the life of the pavement structure. Moisture effects cannot be directly associated with the binder or with the aggregate as they usually affect the interface between the asphalt and the aggregate. However, because it influences the inter-aggregate bond it could be thought of as degradation of the asphalt (J. P. Zaniewski, and S. H. Nallamothu, 2003).

Another important development, which has resulted due to the recognition of the importance of the binder in mitigating rutting, has led many engineers to specify harder binders or polymer-modified binders for pavements in hot climates and heavy-duty pavements (Rabbira Garba, 2002). Asphalt binders have a limited capacity to perform when under wide range of loads and weather conditions which occur over the life of a pavement (Chen et al., 2002). Therefore, binders are modified to improve their performance. The use of bitumen modified with thermoplastic copolymers, elastomers or plastomers in special hot mixes for industrial road surfacing dates back to the 1970's (Brule et al., 1995). Improvement in resistance to rutting, thermal cracking, fatigue damage, stripping, and temperature susceptibility have led polymer-modified binders, PMA, to be substituted for asphalt in many paving and maintenance applications. According to the study by Rafiqul A. T. and Musharraf Z. (2005) on the evaluation of rutting potential of HMA using the Asphalt Pavement Analyzer, mixes containing modified binders which have higher viscosities (and hence higher resistance to flow) show low rut potential. The unmodified binders which have low viscosity exhibit high rut potential. Therefore, the viscosity of binders at 135°C can be a good performance-based binder evaluation parameter. On the other hand, According to a national study of rutting in HMA pavements by Brown and Cross (1992), which tried to correlate HMA properties with the rate of rutting, it was concluded that the properties of the asphalt cements extracted from representative core mixtures are not closely related to rutting. The research indicated that the amount of asphalt cement is of primary importance and that the properties of the asphalt cement are of secondary importance with regards to rutting.

2.4.4. HMA Mixture Properties

2.4.4.1. General

Asphalt concrete consists of asphalt binder, aggregates and air voids. The properties of asphalt concrete depend on the quality of its components, the construction process, and the mix design proportions. It is generally recognized that the volumetric composition of mixtures greatly influence their performance, i.e., their resistance to distresses. A mixture with good performance is one, which is resistant to various load-related and thermally induced distresses such as rutting, fatigue cracking and low temperature cracking (Rabbira Garba, 2002).

2.4.4.2. HMA Volumetrics Basics

Simplistically, a hot-mix asphalt (HMA) material comprises three material components- airvoids, mineral aggregates and bituminous binder. It has long been acknowledged that the performance of HMA mixtures is more significantly influenced by the relative volumetric proportions of the three components. The study and use of the volumetric proportioning of HMA mixtures is called "volumetrics" (Brian J. C. et al.,). The study by Rabbira Garba (2002) summarizes HMA Volumetric as follows:

- Void content (Va)- is the percent by volume of air between the coated aggregate particles in a compacted asphalt mixture.
- Binder content (Pb) - is the percent by weight of asphalt binder in the total mixture, including asphalt binder and aggregates.
- Voids in mineral aggregates (VMA) - is the volume of compacted paving mix not occupied by the aggregates when the volume of the aggregates is calculated based on their bulk specific gravity.
- Absorbed asphalt volume (Vba)- is the volume of asphalt binder absorbed in to the aggregates.
- Effective asphalt volume (Vbeff)- is the volume of asphalt binder not absorbed into the aggregates
- Voids filled with asphalt (VFA)- is the percentage of voids in mineral aggregate filled with asphalt binder.

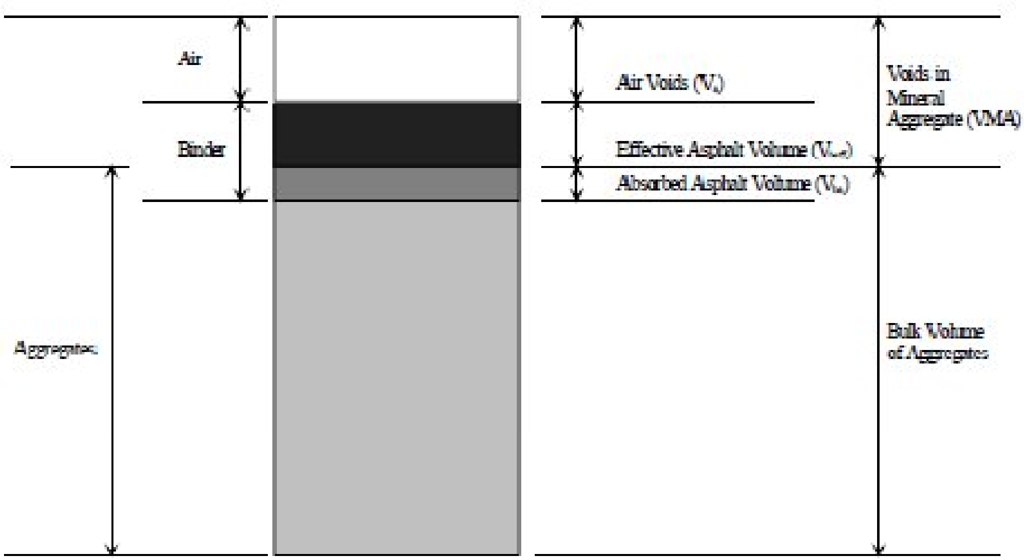

Figure 2-9 Volumetric properties of compacted asphalt mixture (Rabbira Garba, 2002)

The following relationships are used to compute some of the volumetric parameters:-

- Air voids, (Va), expressed as a percent of total volume is given by:

$$\mathrm{Va} = 100(\frac{}{}) \quad \text{..........(2)}$$

Where:
Gmm = maximum specific gravity of the mixture, and
Gmb = bulk specific gravity of compacted mixture.

- Voids in mineral aggregate (VMA) as a percent of bulk volume can be calculated using equation

$$\mathrm{VMA} = 100 - (\frac{*}{}) \quad \text{..........(3)}$$

Where:
Ps = aggregate as percent of total weight of mixture, and
Gsb = bulk specific gravity of aggregates.

- Voids filled with asphalt can be expressed as:

$$\mathrm{VFA} = 100(\frac{}{}) \quad \text{..........(4)}$$

2.4.4.3. Effects of HMA Volumetric Parameters on Rutting

It is generally recognized that the volumetric composition of mixtures greatly influences their resistance to distresses. A mixture with good performance is one, which is resistant to various load-related and thermally induced distresses such as rutting, fatigue-cracking and low temperature cracking (Rabbira Garba, 2002).

Effect of Voids in Mineral Aggregate (VMA)

The main goal of providing 'just sufficient' amount the VMA of bituminous mixtures is to furnish enough space for the asphalt cement so that it can provide adequate adhesion to bind aggregate particles but without bleeding when temperature rises and asphalt expands. During asphalt mixture design of a given aggregate gradation and skeleton properties, mixes with asphalt contents on the 'wet' increasing side of the VMA curve have a tendency to bleed and/or exhibit plastic flow. Any amount of additional compaction from traffic of such mixes leads to inadequate room for asphalt expansion, loss of aggregate contact, rutting and shoving. Generally, during mixture design, aggregate grading should be modified to provide additional VMA. (MS-2

, 1996). According to the study conducted by Dwight et al. (1999), to perform well, an intersection pavement, in addition to having adequate thickness to provide the structural capacity to meet traffic needs, must also have other additional HMA mixture properties. The voids in mineral aggregate, VMA, property of the mix is an important factor. Mixes with marginally low VMA can be sensitive to relatively small changes in the binder content. Small increase in binder can cause these mixes to be susceptible to rutting and shoving. On the other hand mixes with high VMA have thick asphalt coatings on the aggregate particles. This can act like lubricant, allowing the particles to reorient themselves under traffic, which leads to rutting, shoving or bleeding**.**

A field investigation of rutting near signalized intersections in Pittsburgh, Pennsylvania (in the united states) by Kandhal et al.(1998) indicated the following mixture property-related causes relating to poor performance of the pavement:

- Low voids in the mineral aggregate,
- Low air voids,

In the above field investigation, although the mixes were designed in the laboratory with high VMA and air void content, the asphalt pavements densified significantly in the field to yield very low VMA and air voids. It was recommended that the HMA at the intersections should have the following attributes:

- Should maintain adequate VMA to ensure durability,
- Should not densify below 4 percent air voids under slow and standing traffic during hot summer days.

Generally, in the Marshal mix design procedure of MS-2, the laboratory compaction is intended to simulate the in-place density after the mix has endured several years of traffic. However, as the level of traffic on high traffic density corridors currently has increased tremendously, compared to the originally considered traffic loading of 1 million ESAL (one million equivalent standard axles), the in-place densities increase beyond the laboratory densities with further movement of traffic after months of opening. This leads to a decrease in air voids below a certain limit (around 3%), leading to shoving and rutting, especially when the temperature rises. To make up for the this deficiency of the marshal mix design method, researchers have come up with the concept of Refusal density which is is a state of compaction where the density does not change with further compactive effort. To achieve 3% air voids level at refusal density, the best way is to control/modify the aggregate gradation in such a way that it results in higher VMA of mix, which can be attained by making use of the 0.45 power gradation chart (S.K. Rao et al.2007).

Effect of Air Voids (Va)

It is known that during mix design, the most usual design range of air voids of (3-5)% is the design level designed after several years of traffic. This design range will normally be achieved if the mix is designed at the correct compactive effort and percent air voids after construction is approximate to 8%. It is shown that mixtures that ultimately consolidate to less than 3% can be expected to rut and shove if placed in heavily trafficked locations. Factors which contribute to such occurence can be arbitrary or accidental increase in asphalt content at mixing facility, and increased amount of ultrafine particles passing 75µm beyond that is used in the laboratory which will act as asphalt extender (MS-2 , 1996). According to the study Rafiqul A. T. and Musharraf Z. (2005) on the evaluation of rutting potential of HMA using the Asphalt Pavement Analyzer, clear relationship between the percent air voids and rut depth for cylindrical samples was evident. In another study of in-place rutting of asphalt pavements by Brown and Cross (1989), it was concluded that :

- most of rutting observed is attributed low air voids
- satisfactory laboratory compactive effort must be utilized to obtain better performance with regards to rutting
- Marshal flow is a good indicator of rutting potential
- asphalt content must be correctly selected to yield the appropriated void content, and not arbitrarily increased

Even though the above study revealed that low air void was the major attribute to rutting, it pointed out that Va may increase with additional traffic, once rutting has commenced, which may be misleading. According to the findings in this strudy, plastic flow of the asphalt mixture is likely to begin once the Va is reduced to approximately 3 percent.

Generally, it is well established in literatures that low air voids (usually lower than 3% as determined from bulk specific gravity and maximum theoretical specific gravity of the mixture) cause rutting. The results show that if the in-place voids drop below 3.0 to 4.0%, the probability of experiencing unacceptable rates of rutting increases. According to Killingsworth, B. M. (2004), over-compacted asphalt mixture (Va below 3%) can cause rutting, shoving and bleeding, while AC surfaces whose densities are too low (Va above 8%) allow water and air to penetrate into a pavement, increasing the danger for water damage, oxidation, raveling, and cracking. According to Brown and Cross (1992), the amount of rutting is also a function of voids in the re-compacted mix, for cores obtained from pavements. Voids in the re-compacted mixes are an estimate of the mix design void content.

Effect of Compaction level

At the same asphalt content both Va and VMA decrease with increase in compactive effort. For instance, if a mix is designed slightly to the left of the minimum VMA curve at a compaction level of 50 blows and the pavement actually endures heavier traffic than expected (close to 75 blows design level), such mix is expected to be susceptible to rutting. Hence, the appropriate level of compactive effort, which simulates the design traffic must be selected (MS-2 , 1996). According to the study of in-place rutting of asphalt pavements by Brown and Cross (1989), it was concluded that satisfactory laboratory compactive effort must be utilized to obtain better performance with regards to rutting.

Effect of Voids Filled with Asphalt (VFA)

The main effect of controlling the VFA criteria is to limit the maximum levels of VMA and maximum levels of asphalt content. VFA also restricts the allowable Va content for mixes that are near the VMA criteria. Mixes designed for heavy traffic will not pass the VFA criteria with relatively low Va (less that 3.5%), even though that amount of Va is within the acceptable range. This is because low Va can be very critical in terms of permanent deformation (MS-2 , 1996).

Combined effect of HMA Volumetric Parameters

It is known that the none of the HMA volumetric parameters is independent of the other. All of those properties are results and measures of the aggregate gradation, texture and nature, amount of fillers, degree of compaction of mix, and binder content (VFA, VMA and Va are all interrelated, and only two are necessary to solve for the other). A national study of rutting in HMA pavements by (Brown and Cross, 1992), tried to correlate HMA properties with the rate of rutting (as defined in this study is the ratio of the rut depth to the experienced traffic loading in terms of equivalent standard axles). The study concluded that:

- In terms of mix design properties, good performing pavements had higher design air voids (Va), higher VMA, lower VFA, and higher stability. However, These correlations developed for mix design properties have little practical meaning since mix properties produced during plant production likely deviate from mix design properties.

- Selection of the proper compaction level during the mix design phase is critical for proper pavement performance. If the mix design compactive effort is too low, excessive asphalt contents will be designed and rutting could develop as a result of low in-place air voids due to a higher density in-place after traffic than achieved in the mix design.

- By evaluating the aggregate, asphalt content and gradation from cores and mix properties from recompacted material, the material and volumetric properties of the mix "as-placed" can be estimated

- The amount of rutting is also a function of voids in the re-compacted mix. Voids in the recompacted mixes are an estimate of the mix design void content.

- results on the re-compacted samples showed that mixes had unacceptable rates of rutting when voids were below 3%, and if voids stayed above 4% none of the mixes had unacceptable rates of rutting.

- when the voids drop below 3.0% to 3.5% in-place the probability of rutting increases significantly.

- Marshal recompacted mixes with higher recompacted voids, higher VMA and higher stability perform better. Mixes with lower voids filled also perform better. The measured high stability for good performing mixes was caused by the high voids and high rate of oxidation and cannot be used to predict rutting because the Marshall stability values from laboratory compacted samples during construction will be lower than that for aged mixes.

- In-place air void contents above approximately 3.0% are needed to decrease the probability of premature rutting throughout the life of the pavement. In-place air void contents below approximately 3.0% greatly increase the probability of premature rutting. The asphalt mixture must be placed with a void content significantly above 3.0% (usually 5-7%) using a reasonably high compactive effort to insure that the voids in the mix stay above 3.0% during traffic.

2.4.5. Applicability of Marshall Mix Design with Regards to Rutting

A wide variety of equipment and procedures have been developed and used to assess rutting characteristics of HMA mixes in the laboratory. These include: the traditional Marshall and Hveem tests, uniaxial and triaxial static and dynamic creep tests, and the Superpave direct shear test. Among these, the Marshall and Hveem methods are most widely used methods around the world, to establish optimum asphalt contents of HMA mixes based on the concept of stability (resistance to deformation). This stability, however, is neither based on fundamental engineering properties nor has been validated in the field to predict rutting in HMA pavements. The Marshall and Hveem test methods also do not indicate the potential for fatigue cracking in HMA pavements . Researchers have used various types of creep tests for laboratory evaluation of HMA permanent deformation (Collins et al., 1995).

As mix design evolved from conventional Marshall design to the superpave design and beyond, it became increasingly important to identify practical laboratory test methods to predict the performance of HMA pavements. Performance testing has been deemed necessary for a broad acceptance of the Superpave mix design system. Researchers have sought for a simple and yet reliable testing procedure to assess rutting potential of HMA for more than for long. Currently, the most common type of laboratory equipment of this nature is the loaded wheel tester (LWT). Several LWTs currently are being used in the developed world. They include the Georgia Loaded Wheel Tester (GLWT), Asphalt Pavement Analyzer (APA), Hamburg Wheel Tracking

Device (HWTD), LCPC (French) Wheel Tracker, Purdue University Laboratory Wheel Tracking Device (pURWheel), and one-third scale Model Mobile Load Simulator (MMLS3) (Cooley L. A. et al., 2001).

Generally, the purpose of mix design is to determine the proportions of aggregate and binder that would produce a mix, which is economical and has the following desirable properties:

- sufficient binder to ensure durability
- sufficient voids in mineral aggregate, so as to minimize post construction compaction without loss of stability and without causing bleeding, and to minimize harmful effects of air and water.
- sufficient workability to permit laying of the mix without risk of segregation, and
- sufficient performance characteristics over the service life of the pavement. deformation (Rabbira Garba, 2002)

As described by many researchers, Marshall Test in its entirety cannot be used as a direct measure of HMA's resistance to rutting. However, Marshal Quotient(MQ) , which is the ratio of stability to flow, can be used as a measure of materials in-service resistance to shear stresses, permanent deformation and hence rutting ((Martinsh Zaumanis, 2010). The Marshall Quotient(MQ) clearly represents the performance properties of asphalt mixtures such as stiffness, resistance to shear stress and permanent deformation. From the empirical methods of asphalt concrete mixture design, Marshall stability is considered to be a surrogate measure of mixture's shear strength, while Marshall flow is specified to limit permanent deformation (Rabbira Garba, 2002). MS-2 (1996) describes the Marshall Method of Mix Design as the mix design method employed to indirectly control the resistance of bituminous mixtures to plastic flow. B. V. Kök and N. Kuloglu (2007)stated that Marshall Stability can be simply described as the ability of the bituminous mixture to resist excessive permanent deformation. Y. H. Huang (2004) indicated that due to the very fast rate of loading during the standard test procedure, the stability is a measure of the cohesion, while the flow index is a measure of the internal friction.

A study of in-place rutting of asphalt pavements by Brown and Cross (1989), concluded that :

- satisfactory laboratory compactive effort must be utilized to obtain better performance with regards to rutting
- Marshal flow is a good indicator of rutting potential

According to another consequent research by Brown and Cross (1992), it was found that Marshal re-compacted mixes with higher re-compacted voids, higher VMA and higher stability perform better. Mixes with lower voids filled with asphalt also perform better.

On the other hand, due to the complexity of the approach, and requirement for sophisticated apparatus and skill to apply the Superpave method of asphalt mixture design, some researchers and agencies have resorted towards modifying the original Marshal Mix Design Method (as outlined in the MS-2 specifications), and making up for its major deficiencies. According to such researchers, in the Marshal procedure of MS-2, the laboratory compaction is intended to simulate the in-place density after the mix has endured several years of traffic. At the time of time of developing the method, heavy traffic was regarded as something close to 1 million ESA's (one million equivalent standard axles), and the laboratory compaction of 75 blows on the two faces of a Marshal specimen was discerned to achieve the corresponding density level. However, currently on high density traffic corridors (including those in our country), loadings of that magnitude occur within a few months (or the first year) of opening to traffic and the in-place densities increase beyond the laboratory densities with further movement of traffic. As the density increases, air voids in the mix decrease. As air voids decrease below a certain limit (around 3%) the asphalt film enveloping the aggregate particles tends to push them apart, reducing the aggregate particle contact. This action becomes more and more predominant as the pavement temperatures rise in the summer and decrease the viscosity of asphalt. Because of the falling viscosity of the asphalt and diminished aggregate particle contact, the mix loses its shear resistance and deforms as a rut under vehicle wheel with an accompanying heave on the side. The probability of plastic deformation (rutting) of an asphalt mix is very high when the in place air voids fall below 3% . This emphasizes the fact that the air void level in the mix should remain above 3%, if plastic deformation is to be avoided (S.K. Rao et al., 2007). In fact MS-2 warns that mixtures that ultimately consolidate to less than 3% air voids can be expected to rut and shove if placed in heavy traffic locations. Hence, to achieve better performance mixes, pavement investigators (and researchers) have come up with a modification (extention) of the Marshall method, which consists of ensuring that a mix designed by Marshal with 75-blow compaction (say for 4.5 % air voids) also does not compact to lower than 3% air voids when compacted further to 'refusal density' . Refusal density is a state of compaction where the density does not change with further compactive effort. Shuler, T.S., and Huber, G.A, as edited by Richard C. M. (1992) also concluded that by designing mixtures to a level of refusal density where air voids could never be reduced to some minimum level, plastic flow or loss in shear strength due to a high percentage of voids filled, should not occur. It is important to note that Densification of asphalt concrete after construction and during traffic varies between mixtures and is related to the method of laboratory compaction, design criteria , and construction technique.

To achieve 3% air voids level at refusal density while designing the mix with 75 blow, Marshal compaction, there are two possible ways. One way is by reducing the asphalt content and designing the mix at a higher initial air voids content (say, close to the upper limit); and the other way is by choosing an aggregate grading with higher voids ratio, that is one which results in a higher VMA of mix. But it is known that mixes prepared in the first option would be less durable and prone to premature cracking and raveling. For mixes prepared in the latter way, it would be necessary to study the aggregate grading characteristics and to select grading that yields higher VMA values. To this end, the 0.45 Power Gradation Chart comes in to picture along with the project specification band. As discussed in the previous sections, gradings that closely follow the 0.45 power grading curve give rise to low VMA values. Gradings that plot above or below are capable of generating greater VMA values. Grading which plots above is a finer grading, and in such a grading the contribution of aggregate internal friction to the shear resistance of the mix is low. Gradings that plot below the 0.45 power grading line are coarser gradings, and such gradings yield higher VMA values and should be preferred. But grading plotting that plot too much above or too much below the 0.45 power grading line give rise to tender or harsh mixes respectively, causing compaction or segregation problems in the field (S.K. Rao et al.,2007). Therefore, VMA increase should be achieved by an overall adjustment of the gradation. MS-2,(1996) stipulates that by adjusting the proportional percentages of the aggregates that substantially contribute to the intermediate sizes, the gradation curve can be revised to plot further away from the maximum density line. According to the investigations by S.K. Rao et al.,(2007) of several mixes for bituminous concrete with 26.5mm nominal maximum size of aggregate, gradings coarser in the finer fractions upto 2.36mm sieve size and finer in the coarser fractions from 9.5mm sieve size upwards relative to the 0.45 power grading, as shown in figure 2.10 below, offer best VMA values and also better overall Marshall characteristics.

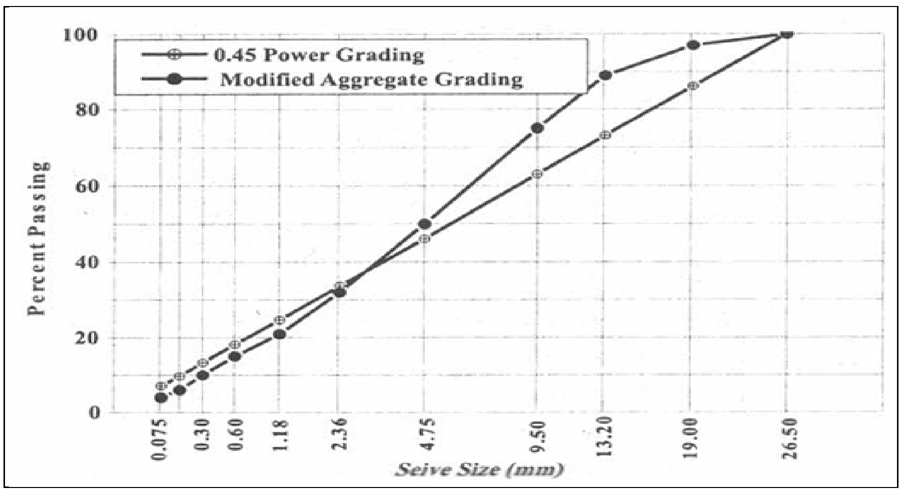

Figure 2-10 A Typical modified aggregate grading plotted on 0.45 power grading , which offers best VMA values (S.K. Rao et al.,2007)

ERA Pavement Design Manual, Volume-I, Flexible Pavements and Gravel Roads - 2002, recommends that under severe loading conditions asphalt mixes must be expected to experience significant secondary compaction in the wheel paths, and hence refusal density design must be adopted to ensure that there is still at least 3% Va. Severe conditions have not been precisely defined but were described to consist of a combination of two or more the following:-

- High maximum temperatures
- Very heavy axle loads
- Very channeled traffic
- Stopping or slow moving heavy vehicles

According to ERA Pavement Design Manual, Volume-I, Flexible Pavements and Gravel Roads - 2002, failure by plastic deformation in continuously graded mixes occurs very rapidly once the VIM are below 3 per cent. Therefore the aim of refusal density design is to ensure that at refusal there is still at least 3 per cent voids in the mix. Two methods namely the extended Marshall Compaction and Compaction by vibrating hammer have been recommended to determine the refusal density. Moreover, for severe sites, the base course specifications, BC1 and BC2 have been recommended to be the most appropriate. However, there are certain reservations that the multi-blow Marshall compaction may cause breakdown of aggregate particles, and that excessive cooling may occur due to the long time entailed.

2.4.6. Other Factors Which Influence Rutting

Rutting is a very complex phenomenon, which is affected by various parameters. It is very difficult to fully and exactly spot each and every aspect of the causes rutting in a pavement section. Some asphalt mixes that have a good history of resisting rutting in posted speed applications may not perform in intersections, climbing lanes, truck weigh stations, and other slow speed areas. The slow moving or standing loads occurring at these sites subject the pavement to higher stress conditions which can be enough to induce rutting and shoving. Braking, accelerating and turning movements generate shear stresses at the pavement surface. Engine fluid droppings and heat exhaust increases with slower traffic and has a softening effect on asphalt. In addition, load repetitions at intersections are sometimes double than that of mainline pavement due to the cross flow of traffic (J. P. Zaniewski, and S. H. Nallamothu, 2003).

To achieve desired performance for asphalt intersections these pavements have to be treated differently than regular open-road pavements by designing them for more severe conditions. The intersection can be built for more severe conditions by insuring structural adequacy, Selecting and controlling materials, following good construction practices, and implementing the plan.(Dwight et al., 1999). According to the study conducted by Dwight et al. (1999), to perform well, an intersection pavement, in addition to having adequate thickness to provide the structural capacity to meet traffic needs, must also have other additional HMA mixture properties. The voids in mineral aggregate, VMA, property of the mix is an important factor. Mixes with marginally low VMA can be sensitive to relatively small changes in the binder content. Small increase in binder can cause these mixes to be susceptible to rutting and shoving. On the other hand mixes with high VMA have thick asphalt coatings on the aggregate particles. This can act like lubricant, allowing the particles to reorient themselves under traffic, which leads to rutting, shoving or bleeding. Careful selection of the asphalt binder and the aggregate is required to provide optimum pavement performance. More rut resistance binders are needed at intersections. As described by by J. P. Zaniewski, and S. H. Nallamothu, (2003), AASHTO's MP-2, standard specification for Superpave volumetric mix design, requires that the high temperature grade be increased by two grades for standing traffic (less than 20km/hr) and by one grade for slow traffic

(20 to 70 km/hr). Other related factors which affect permanent deformation area as under-listed below.

2.4.6.1. Effect of Number of Load Cycles

Previous studies have manifested that the permanent deformation increases with increased load repetition until a point where the deformation starts to decline. This stabilization of the material is only achieved if the applied stress level is low. However, if the load application is relatively large a sudden increase of the deformation can appear in the material. The particles in the material reach a point where they no longer can withstand the stresses from the load and either break or a rearrangement of the particles takes place, which in its turn leads to a sudden collapse. The development of permanent deformation is a gradual process and every load repetition contributes to an accumulation of strain and by that an increase of the total rutting. Therefore, studies of the effect of number of load repetition are substantial for the analysis of the long-term behavior of the material (Lekarp, F., 1997).

2.4.6.2. Tire Pressure

The study by Markshek, K.M. et al., (1986) questions a commonly made assumption that contact pressure approximately equals tire inflation pressure. The study showed that increased tire pressure produced proportionally smaller gross contact areas. This suggests that the commonly made assumption of equal pressure becomes increasingly less valid. This suggests that some contact areas will have pressures greater than calculated for a uniform pressure. Extraordinary high tire pressures mean that asphalt concrete layers which are nearest to the surface in a pavement structure may not be immune to rutting.

2.4.6.3. Moisture

One desirable property of bituminous mixtures is their resistance to moisture induced damages. The moisture induced damages (typically called stripping) can be defined as the weakening of or eventual loss of the adhesive bond between the aggregate surface and the asphalt binder in HMA pavements or mixtures, usually under the presence of moisture (Zemichael B. M. , 2007). The effect of moisture on rutting is also another factor which should not be underestimated. Observations while cutting the trench and during the laboratory testing operation by Brown and Cross (1989) showed a significant amount of uncoated aggregate on rutted sections near shoulder locations(source of moisture intrusion). Consequently the researchers concluded that the observed rutting on those sections was partly attributed to stripping.

2.4.6.4. Pavement Temperature and Loading Speed

According to the study by Fwa, T. F. and Tan, S.A, as edited by Richard C. M., (1992), effect of temperature became increasingly pronounced when the test speed using the wheel tracking apparatus was reduced. This has a significant implication on the rutting behavior of asphalt pavements at traffic-light junctions where slow moving and static traffic loads are of major concern. Taking 10,800 passes of load applied at 45^0C and 60^0C , the corresponding rut depths measured were 3.89mm and 12.95mm. From this, it was concluded that in the tropical climates where pavement surface temperature stays above 60^0C for 3-4hrs on a typically hot sunny day, the rutting performance of asphalt mixes is a critical design consideration for pavements at traffic -light junctions and slow lanes that carry heavy traffic.

2.5. Concluding Summary

Based on the literatures reviewed, it is evident that the various HMA material and volumetric properties have clear correlations with rutting, and that the simplified empirical HMA mix design and testing procedures can be employed for this purpose. This is of course, resorted to, in the inevitable lack of the more advanced laboratory apparatus and testing procedures, which can more accurately predict, assess and evaluate HMA rutting,

As far as the scope of the conducted literature review is concerned, specific studies regarding the effects of HMA material and mixtures properties have not been conducted in our country, particularly on the major trunk roads. Hence, permanent deformation(rutting) being a major problem on most of the major trunk roads in the country, it is evidently of major significance to conduct a research pertaining the correlation of the various aggregate and mixture properties of HMA and permanent deformation in the form of rutting.

3. RESEARCH METHODOLOGY

3.1. Introduction

The study involves investigating the various aggregate, filler and volumetric parameters of HMA cores (in-place and re-compacted cores), obtained from sections of road (trunk-road section selected for the case study) the with varying measured rut depths. As depicted in the flow chart in figure 3.1 the overall study procedure involved the selection of road sections to be evaluated, collection of traffic and construction data of the subject trunk-road selected for the case-study, obtaining cores across traffic lanes of the selected sections, conducting rut depth and related field measurements, conducting laboratory tests on the core samples, and conducting statistical correlation analysis between the various aggregate and mixture properties of HMA and rut depth.

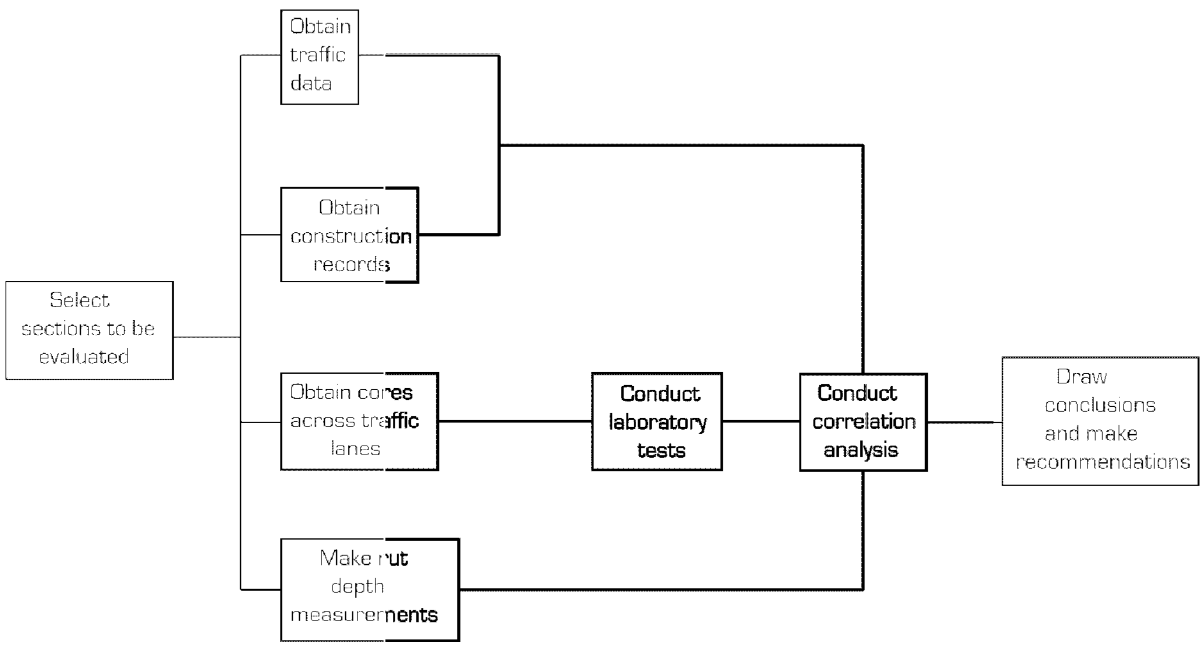

Figure 3-1 The overall Research Plan

3.2. Data Collection

3.2.1. Selection of the Trunk Road Segment for the Case-Study and Sections within the Road to be Evaluated

To meet the objective of this research, which is to establish correlations between the various HMA aggragate and mixture properties and rutting, it was essential to indentify and select a road segment on which not only was rutting a predominant mode of distress, but also there was evidence that there were sections on which the cause of rutting was the HMA wearing course and not the underlying granular base course material the subbase or the subgrade. To this end, a pertinently early rutted trunk road segement- The Gohatsion-Dejen Road Segment, which had a well-recorded history of "early" permanent deformation in the form of rutting (in addition to shoving, corrugation and bleeding, which, even though not solely, but to some extent give an idea about the source of permanent deformation) has been selected for the case study.

Four major criteria were considered during the selection of the sections with in the trunk-road segment to be evaluated:-

1. evidence that the source of rutting was in the HMA layer not in the underlying granular base course material the subbase or the subgrade
2. effect of road section geometry on the speed of traffic loading and mode of vehicular maneuver
3. ensuring uniformity with regards to the pavement section pertinent to the subgrade CBR values
4. requirement of varying of rut depths pertinent to the statistical correlation analysis

The first criteria was that such sections were selected in such a way that there was evidence that the source of rutting was in the HMA layer not in the underlying granular base course material the subbase or the subgrade. As explained in section 2.3.4 of this paper, it is well established in literatures that the shape of the surface profile can indicate which layer is responsible for the failure of the pavement structure due to rutting. In general when the asphalt concrete layer is responsible for total rutting, significant heave is observed on the sides of the rut (White et al., 2002), (Rabbira Garba, 2002). Moreover, other coincidental distresses such as transversal shoving and bleeding, which coupled with the shape of the surface profile, give an indication that the cause of the observed rutting is the HMA surface course, have also been identified. The surface profiles of each of the sampling sections is as shown in figure 3.2.

In fact, the subject road selected for the case-study has a history of premature failure of the asphalt surface course, after two months of construction on considerable length of the project road, as indicated in the [Project Completion Report for Rehabilitation of Trunk Road, Phase III (Gohatsion - Dejen Section) , 2009]. The deformations characterized by corrugation, and rutting (in addition to bleeding) following the wheel path were observed on considerable length of the project road. According to this report, an investigation on the pavement including temperature measurement at different times of a day (9:00 am, noon and 3:00pm) consecutively for a week at different four locations representative to the problem were conducted and according to the results of the investigation, a temperature more than 60°C (expected maximum pavement temperature as per the Marshal mix design method) was measured at 3:00pm for four days out of the seven measurement days, and it was finally concluded that, the high temperature in conjunction with the steep slope, sharp curves, increased number of truck trailers (including fuel trucks to and from Sudan) may have attributed to the instability of the mix of the asphalt pavement. This information however, has simply been used from an objective point of view, and the actual findings of this research have not been prejudiced, in any way what so ever, in this regard.

Table 3:1 Details of sampling and field measurement locations

Sampling location ID	Station	Approximate location coordinate	Measured average rut depth(mm)	Description of sampling location with respect to road geometry		Remark
				Horizontal Alignment (radius)	Vertical Alignment (grade)	
1	3+200	X=0416940 Y=1108748	14.3	On a tangent	7.286% Uphill (LHS)	
2	6+680	X=0416698 Y=1110199	28.8	LHS curve, R=110m	8.378% Uphill (LHS)	
3	13+280	X=0416177 Y=1112250	48.2	LHS curve, R=300m	7.994% Uphill (LHS)	
4	16+800	X=0414412 Y=1111834	38.6	LHS curve, R=750m	8.426% Uphill (LHS)	
5	21+940	X=0410841 Y=1114830	22.4	RHS curve, R=45m	11.393% Uphill (RHS)	
6	28+700	X=0410849 Y=1114845	7.4	at the end of a RHS sharp curve, R=82.5m, L = 110m	7.938% Uphill RHS)	
7	26+800	X=0410829 Y=1114845	87.6	On a tangent	8.333% Downhill RHS)	near the beginning of a RHS curve, R=100m

On the other hand, it is known that one of the factors which affect rutting is the effect of the road section geometry on the speed of traffic loading and mode of vehicular maneuver. Low speed sections such as climbing lanes or steep grades are known to entail high stress conditions, which can be enough to induce rutting and shoving, compared to flatter grades. Turning movements on sharp curve areas are also known to generate shear stresses at the pavement surface, which can load to rutting. Hence, the selection of the specific sections of road to be evaluated needed to ensure consistency in this regard. To this effect, as shown in table 3.1 most of the sections selected for evaluation are on curves (radius varying from 45m to 750m), except for sections 1 and 7 which are on tangents. Section-7, however, even though it is actually located on a tangent, it is practically situated near the beginning of a RHS curve of 100m radius. Regarding vertical alignment, all the sections are located on steep uphill grades , with grades varying from 7.3% - 11.4%, except for section-7 which is located on steep downhill grade of 8.33% gradient, and which happens to be the most rutted section.

The third major criteria considered during the selection of the sections with in the trunk-road segment, was ensuring uniformity with regards to the pavement section pertinent to the subgrade CBR values. To this end, as indicated in the as-built drawings in the [Project Completion Report for Rehabilitation of Trunk Road, Phase III (Gohatsion - Dejen Section) , 2009], pavement sections of approximately close subgrade CBR strengths, with in the range 3-10 %, have been selected. Consequently all of the sections selected for analysis have pavement sections, in which a 50mm HMA suface course overlies a bituminous stabilized binder course of 50mm thickness, base course of 200-250mm thickness, subbase course of 100-200mm thickness and an existing subbase course of 100mm thickness, as shown in Appdendix B.

The fourth major criteria considered during the selection of the sections with in the trunk-road segment to be evaluated was the requirement of varying of rut depths pertinent to the statistical correlation analysis. For this purpose, rut depth measurements were made on a number of locations, in which the source of rutting was deemed and verified by the aforementioned procedures to be the HMA surfacing material. For each location, the rut depth measurements were conducted on five spots of 1m interval along the rut line, and the average value was taken. Out of the various locations, seven sections of varying depths of rut, in the range of (7.4mm - 87.6mm) were selected, as shown in table 3.1.

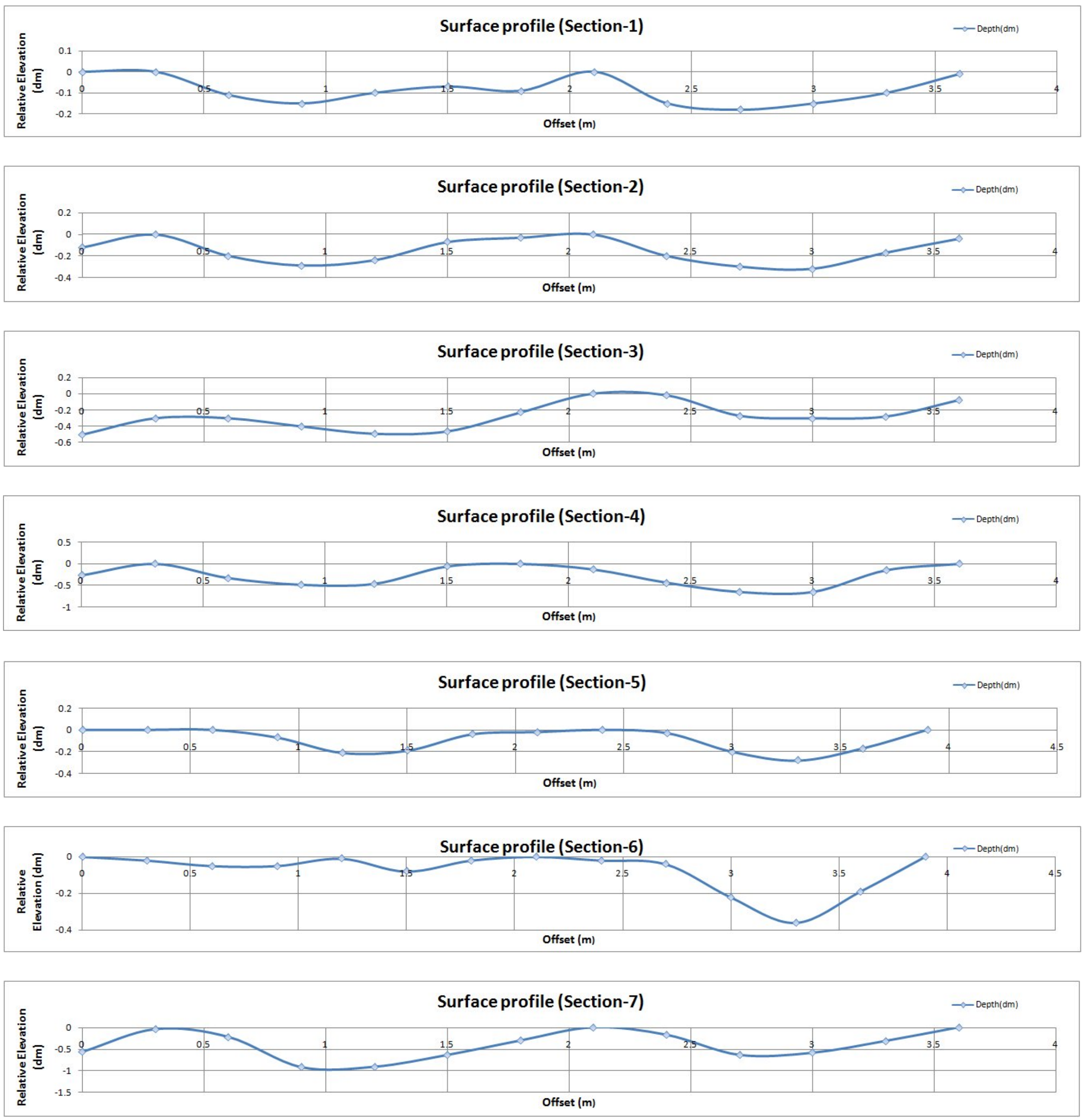

Figure 3-2 Surface profiles of evaluated sections of road

3.2.2. Construction Records and Traffic Data of the Subject Road

It was not possible to obtain the construction quality control data, design reports and traffic reports directly pertinent to the subject road. However, from the [Project Completion Report for Rehabilitation of Trunk Road, Phase III (Gohatsion - Dejen Section) , 2009], general ideas regarding the details of construction methods and materials used and pavement design details were obtained. Based on the aforementioned document:-

- there were three types of bituminous pavements used in the project- asphalt surface course, asphalt stabilized base course and open graded base asphalt of the bituminous semi-flexible pavement in Dejen town.
- the source of aggregate was at three locations at Abay river bed.
- the percent fracture of coarse aggregate was far more than the requirements.
- the asphalt concrete mix used for pavement construction was proportioned using the tested aggregates, fillers and bitumen by the standard Marshall mix design method.
- tests conducted on core samples after placement of the asphalt concrete mix were thickness and degree of compaction
- MC-250 cut back asphalt was used for prime coat, RC-250 as tack coat, and 60-70 penetration grade asphalt for all bituminous mixes

Regarding the design traffic class of the subject road, based on the typical cross-section drawings, for subgrade classes **a)** S2, **b)** S3 and **c)** S4, pavement sections comprising of **a)** [50mm asphalt surface course, 50mm asphalt stabilized bindercourse, 250mm basecourse, 200mm subbase course, 100mm existing surface subbase course], **b)** [50mm asphalt surface course, 50mm asphalt stabilized bindercourse, 250mm basecourse, 100mm subbase course, 100mm existing surface subbase course], **c)** [50mm Asphalt surface course, 50mm asphalt stabilized bindercourse, 200mm basecourse, 100mm subbase course, 100mm existing surface subbase course], were provided respectively. Based on the relevant stipulations in ERA Pavement Design Manual, Volume-I, Flexible Pavements and Gravel Roads - 2002, and the Flexible Pavement Design Catalog, taking into consideration the materials and pavement thickness, it is evident that the subject road belongs to traffic class T6 (6 - 10 million ESA's).

On the other hand, from the report 'Traffic Survey and Safety Measures Report for Addis Ababa - Gohatsion Road Project, Detailed Engineering Design, Tender Document Preparation and Construction Supervision (2012) By Hitcon Engineering', relevant data pertaining the traffic (AADT), traffic growth trends and axle load survey, and cumulative number of standard axles for the road stretch from Chancho to Gohatsion was obtained and objectively analyzed and utilized for the purpose of this research. The major assumption here was that AADT for the road stretch from Chancho to Gohatsion, which was determined on the basis of only the normal traffic operating on the road on the year 2012, would be a fair estimate of the traffic incoming to and outgoing from the Gohatsion - Dejen Road Segment; as major diversion of the incoming and outgoing traffic is not expected at Gohatsion town. Consequently the AADT for the Chancho-Gohatsion road segment for the year 2012 was determined to be 1464, and the AADT projected for the year 2013 to be 1584, as shown in Appendix K. The same assumption holds for the axle load survey and the related cumulative equivalent standard axles determined based on the AADT for the most loaded direction. According to the above report, the consultant carried out axle load surveys for two days on a sample of commercial vehicles along Addis-Chancho section, from both directions, by a mobile weigh bridge for a total of 552 vehicles (8 small buses, 4 medium buses, 32 large buses, 146 small trucks, 64 Medium trucks, 94 heavy trucks, 23 Fuel trucks and 182 truck trailers). Consequently, equivalent axle loads for each vehicle category were determined as shown in table 3.2. The cumulative number of standard axles, experienced by the subject road during the years of 2012 and 2013 was then determined to be 2.6 million ESA's, for the year 2013, as shown in table 3.3.

ERA Pavement Design Manual, Volume-I, Flexible Pavements and Gravel Roads - 2002, recommends that under severe loading conditions asphalt mixes must be expected to experience significant secondary compaction in the wheel paths, and hence refusal density design must be adopted to ensure that there is still at least 3% Va . Severe conditions are described to consist of a combination of two or more the following:-

- High maximum temperatures
- Very heavy axle loads
- Very channeled traffic
- Stopping or slow moving heavy vehicles

As the subject road is known to be located on high summer temperature area (at times with greater than 60^0C pavement temperature), where heavy axle loads exceeding the legal load limit (10 ton axle load , according to the ERA Manual 2002, Pavement Rehabilitation & Asphalt Overlay Manual) are experienced and where traffic maneuver is slow moving, turning due to the road geometry, the road obviously required the refusal density design approach. However, the construction documents clearly indicated that the standard Marshall Mix Design approach was utilized for the asphalt concrete mix design used for pavement construction. On the other hand, from the cumulative number of standard axles approximated for the subject road (table 3.2), it is evident that the legal design load limit of 10 ton axle load, beyond which pavement overloading is entailed, is being exceeded , and this has a detrimental effect on the rutting performance of pavements. Of course, the excessive axle loads can be taken into account during pavement account to determine a structurally sufficient layer thickness. However, taking into consideration the geometric nature of the subject road, the subsequently severe loading conditions, and high pavement temperatures (exceeding 60^0C at times) the excessive axle loads may also have contributed to rutting.

Table 3:2 Cumulative number of standard axles for Chancho-Gohatsion Road Segment ('Traffic Survey and Safety Measures Report for Addis Ababa - Gohatsion Road Project, Detailed Engineering Design, Tender Document Preparation and Construction Supervision, Hitcon Engineering (2012)",

Vehicle Category	ESAL(ton)
Small Bus	0.38
Medium bus	0.38
Large Bus	2.31
Small truck	0.3
Medium truck	3.5
Heavy truck	10.97
Truck trailers	22.72

Table 3:3 Cumulative number of standard axles for Chancho-Gohatsion Road Segment ('Traffic Survey and Safety Measures Report for Addis Ababa - Gohatsion Road Project, Detailed Engineering Design, Tender Document Preparation and Construction Supervision, Hitcon Engineering (2012)".

year	Small Buses 12- 25 seats	Medium Bus 25 - 45 seat	Large Bus >45 seat	Small Truck/ Isuzu	Medium Truck	Heavy Truck	Trucks & Trailer	Total for the year	Cumulative	year
2012	5	7	200	22	221	788	2164	1243449	1.24E+06	1
2013	5	7	214	24	241	863	2369	1359495	2.60E+06	2

3.2.3. Rut Depth Measurement

The rut depth measurement was accomplished using a 3.6 m straight edge. As mentioned above, rut depth measurements were made on a number of locations, in which the source of rutting was deemed and analytically verified to be the HMA surfacing material. For each location, rut depth measurements were conducted on five spots of 1m interval, along the rut line, and the average value was taken. The details of the rut depth measurements and related site conditions are included in Appendix A.

Figure 3-3 Rut depth measurement at section-7 (km 26+800)

3.2.4. Obtaining Core Samples

Five core samples, within a radius of 3m from the center of the rut depth measurement locations, were obtained from each of the selected seven sections using the standard 4-inch diameter core cutting machine. At each sampling section, on the rut line in question, two core-samples to the left of the rut line (L1 and L2), two core-samples to the right of the rut line (R1 and R2) and one core-sample along the rut line (C) were obtained, at 75cms interval (longitudinally) as shown in figure 3.4. However, due care was taken not to extract core samples from significantly visible rut-side heaves.

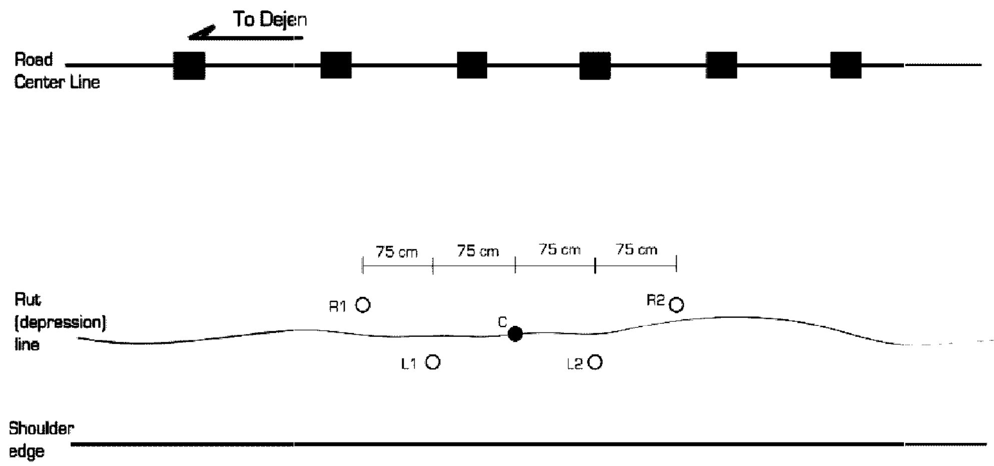

Figure 3-4 Core sampling plan

The height of the obtained core samples was variable deepening on the performance of the core cutting machine, the specific road-section condition and specific pavement structure. The research focuses on the top HMA surface course of the pavement, which is of 50mm thickness, as identified from the as-built drawings of the subject road. In fact, all of the sections of the road, from which core samples were obtained, have a 100mm thick bituminous surfacing (in which a 50mm asphalt concrete wearing course [WC] overlies a 50mm thick a binder course[BC]), base course of 200-250mm thickness, subbase course of 100-200mm thickness and an existing subbase course of 100mm thickness, as shown in Appdendix B. However, for the purpose of laboratory testing, only the top 50mm of the obtained core samples was used. Based on the stipulations in ERA Pavement Design Manual, Volume-I, Flexible Pavements and Gravel Roads - 2002, and the Flexible Pavement Design Catalog, the binder course [BC], can either be of quality the same as asphalt concrete [BC1], dense and continuously graded, but with a slightly higher nominal maximum aggregate size, or a bituminous macadam [BC2], which is close and continuously graded, with a slightly higher nominal maximum aggregate size. Hence, the fact that the investigation focuses only on the top HMA surface course, could be justified in that 1) the top HMA surface course is the layer which is expected to experience most, the effect of traffic induced stresses, and hence rutting, 2) the quality of the immediately underlying binder course is of the same to less denser, and coarse graded particle size distribution, which ensures higher voids in mix, and higher resistance to plastic deformation.

Figure 3-5 Typical core sample extraction process and extracted core samples at km 26+800

3.3. Laboratory Testing

3.3.1. General

Out of the wide variety of equipment and procedures used to assess rutting characteristics of HMA mixes in the laboratory, the Marshall testing method along with some other pertinent test procedures has been adopted for this research. The tests conducted are primarily intended to determine two basic properties, in-place and re-compacted. It is known that there is a difference between the properties of mix in mix design and plant production. Thus, it was envisaged that by evaluating the aggregate, asphalt content and gradation from in-place cores and mix properties from re-compacted cores, the volumetric properties of the mix 'as-placed' can be estimated (even though ideally this information should be obtained during mix production). Hence, the objective of conducting tests on the in-place and re-compacted cores was to evaluate the in-place mix properties and the mix properties "as-placed."

As mentioned above, five cores have been obtained from each of the seven sampling sections. Three cores out of the five have been used to determine the in-place properties. The tests and volumetric analysis have been conducted on in-place cores and re-compacted cores of HMA obtained from the pavement section in question. the in-place tests are intended to characterize the properties of HMA mixtures, as they are during the time of sampling. On the other hand, the re-compacted tests are intended to characterize or estimate the properties of HMA mixtures "as-placed", i.e., to characterize what was expected during the original mix design. For this purpose, 3 out of the 5 cores per section have been utilized to characterize the in-place properties including the Bulk specific gravity of mix AASTHO T166,, Marshall stability, flow (ASTM D1559/AASTHO T245), extraction of asphalt and coarse aggregates, aggregate-bitumen coating and stripping test (AASTHO T182), bulk specific gravity of coarse aggregates (AASTHO T85), fine aggregates (AASTHO T84) and fillers (AASTHO T100), and determination of percent fractured particles in coarse aggregates ASTM D 5821. The remaining 2 cores have been utilized maximum theoretical density determination (AASTHO T209) and assessing the re-compacted properties including heating, breaking-up, and re-compacting, determination of the bulk specific gravity of re-compacted mix AASTHO T166, Marshall stability, and flow (ASTM D1559/AASTHO T245). Moreover, the void analysis for both the in-place and re-compacted mixtures has been conducted. The overall test plan is as depicted in figure 3.6 below.

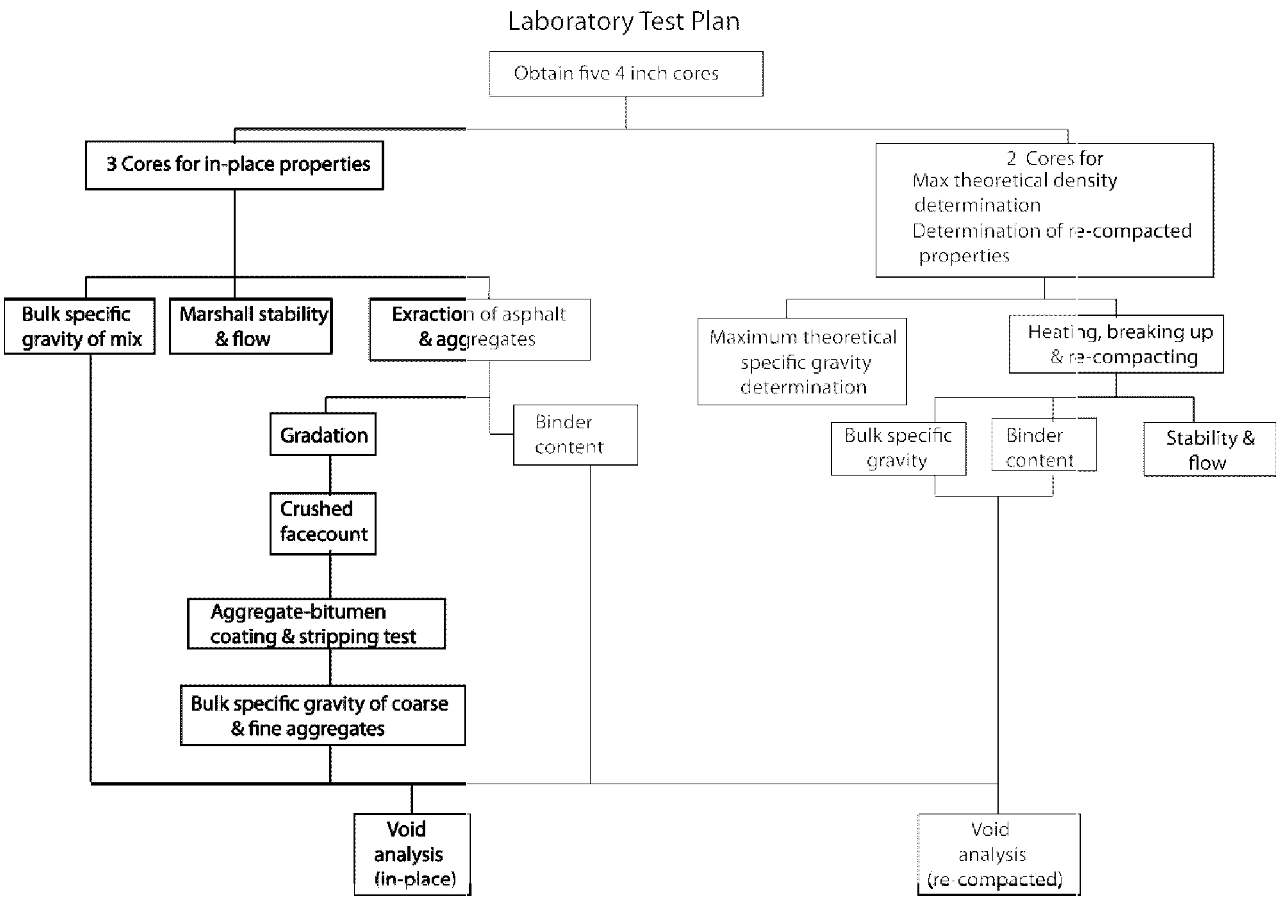

Figure 3-6 The overall laboratory test plan

3.3.2. Marshall Stability and Flow

Marshall stability and flow tests as outlined in AASHTO designation T 245, were conducted on in-place and re-compacted core samples obtained from each sampling section. Regarding the in-place core samples, Marshall stability and flow tests for three samples from each sampling section were conducted. The samples were sawed in to the required 50mm thickness, representative of the HMA surface course. The bulk specific gravity of the specimens were determined. The samples were then immersed in a thermostatically controlled water bath with a temperature of 60±1 °C. The specimens were removed from the water bath, their surface dried, and placed in the Marshall testing machine. Load was applied to the specimen by means of a constant rate of movement of the load jack of 50.8mm per minute, until the maximum load was reached and the load decreases. The maximum load in KN was recorded. In addition, the flow, expressed in units of 0.25mm was measured. The measured stability values were adjusted based on the conversion factors (correlation ratios) for the actual volume of specimens, which depart from the standard sample 63.5mm thickness.

Regarding the re-compacted core samples, for each sampling section, one of the two remaining cores (from the total of five samples) was heated in the oven, at a temperature of 105-110 °C for 16-18 hours, and fully broken-up. The sample was then heated to the mixing temperature of 150°C, which is the approximate temperature in the range of 135 - 163°C, expected to bring about the kinematic viscosity of the 60-70 penetration grade asphalt to the range of 170±20 centistokes. The sample is then mixed, and allowed to slightly cool down its compaction temperature of 130-140°C, which is the approximate temperature expected to bring about the kinematic viscosity of the 60-70 penetration grade asphalt to the range of 280±30 centistokes. The sample was then placed in a mold in which a paper disk has been placed, spaded with spatula 15 times around its perimeter and 10 times over the interior, a paper placed on the top the mix and compacted using 75 blows {which is justified from the design ESAL considered for the pavement design of the subject road, i.e., traffic class T6 (6-10 million ESA's) and equivalent to the traffic ESAs of 2.6 million ESA's experienced by the subject road, from opening day for traffic to the day of sampling}, on both sides. The specimens were then left to cool overnight, and after determining their bulk specific gravities, the same procedures as outlined for the in-place cores were followed to determine the stability and flow of the samples.

3.3.3. Extraction of Asphalt and Aggregates and Binder Content Determination

The quantitative extraction of bitumen from bituminous paving mixtures as outlined in AASHTO designation T 164-97, was adopted, with the exception of the use of benzene instead of the specified extracting reagents such as Tricholoroethylene, Methylene chloride or Terpene extractant. The procedure has been applied on three out of the five core samples per section, i.e., the same samples which have been used to determine in-place Marshall stability and flow tests. The samples were first broken-up with their weights initially recorded. The bowl containing the test portion and solvent was then placed in the centrifuge extractor, with dry filter ring fit around the edge of the bowl. The centrifuge was set to revolve, and sufficient extractant (benzene), of measured volume, was added. The procedure was repeated until pure benzene begins to drain. All of the aggregate in the centrifuge bowl was then carefully transferred in to a tared metal pan, and oven dried to a temperature of around 110°C.

To determine the filler content, the benzene-bitumen-filler mix was stirred thoroughly, and 100ml of sample was taken. The 100ml sample was then heated in a cylinder flame. Upon heating, first the benzene evaporates, and then the asphalt burns and adheres to the sides of the container, leaving the mineral filler concentrated around the middle of the container. The asphalt was then carefully cleaned from the container until the 'ash-like' mineral filler remains. The weight of filler obtained from the 100ml of the benzene-bitumen-filler was then extrapolated to obtain the total weight of mineral filler in the entire benzene-bitumen-filler mix. This mineral filler weight along with the additional miner filler content from the filter paper (obtained by oven-drying and weighing), was used to determine the total weight of mineral filler in the total mix.

The total mass of the extracted aggregates was determined by adding the total weight of fillers to the mass of oven-dried aggregate after extraction. The mass of bitumen will finally be determined from the difference between the mass of the sample before extraction and the mass of total aggregates, and the corresponding bitumen content will be expressed as the mass of bitumen to the total mass of sample.

Figure 3-7 Laboratory extraction of asphalt and aggregates

3.3.4. Aggregate Angularity

The procedure as outlined in ASTM designation D 5821 has been adopted to determine the percent fractured particles in coarse aggregate. A fractured face is defined as being caused by mechanical means or by nature and should have sharp or slightly blunted edges. A broken surface constituting an area equal to at least 25% of the projected area of the particle, as viewed perpendicular to (looking directly at) the fractured face, has been considered an acceptable fractured face. Fractured Rock Particle - A rock particle having two fractured faces, as required for that class/type of aggregate in the specifications. A crushed particle has been defined as one with two or more fractured faces.

Coarse aggregate material sample (material retained on 4.75mm sieve size), obtained from extraction was initially separated on the 9.5mm sieve in to (19.0-9.5mm) and (9.5-4.75mm). The test was then conducted on each of these portions of the sample. Each sample was washed and dried to a constant weight. The test sample was then spread on a clean flat surface, and separated manually using spatula into either crushed particles (i.e., particles with two or more crushed faces) or uncrushed particles (i.e., particles with less than two crushed faces). After determining the weight of the crushed particles, the percentage of crushed particles for each of the coarse aggregate portions (19.0-9.5mm) and (9.5-4.75mm) is determined as:-

Percent crushed particles (P) = —— x100 ....(5)

Where:

F = Weight of crushed particles with at least the specified number of fractured faces, in grams

N = Weight of uncrushed particles, in grams

Total Percent Crushed Particles =

(% Crushed Particles [19.0 - 9.5 mm]) x (% of [19.0 - 9.5 mm] Material)
+
(% Crushed particles [9.5 - 4.75 mm]) x (% of [9.5 - 4.75 mm] Material)

a) b)

Figure 3-8 Determination of percent crushed particles: a) [19.0 - 9.5 mm] and b) [9.5 - 4.75 mm] for samples obtained from section-7

3.3.5. Aggregate - Bitumen Coating and Stripping Test

The procedure as outlined in AASHTO Designation T-182-84, has been adopted to determine the coating and stripping of the HMA mixtures. The method describes coating and static immersion procedure for determining the retention of a bituminous film on an aggregate surface in the presence. Aggregate of size range (9.5-6.3)mm was first washed in distilled water to remove all fines, and dried at 135-149°C, to a constant mass. 60-70 penetration grade bitumen, of the type and grade, same as that used in the project in question was prepared. 100g of oven dry aggregate was weighed into a container and heated to 135-149°C temperature, in an oven for 1 hour. The asphalt was heated separately to the same temperature. 5.5g of heated bitumen was then added to the hot aggregate, and the mixture was mixed vigorously with a heated spatula for 2-3minutes, until the aggregate was completely coated. The mixture was then allowed to cool to room temperature. The coated aggregate was then transferred to a 600ml glass container, and 400ml distilled water, at room temperature was added to the glass container. The coated aggregate was then allowed to remain immersed in the water for 16 to 18 hours. Finally by observation through the water, from above, with the aid of as shaded lamp, the percentage of the total visible area of aggregate which remains coated above or below 95% was determined. Brownish translucent areas were considered to be fully coated.

It is known that, as this method requires subjective evaluation of the test results of only two possible conditions, the procedure does not lend itself readily to a conventional statistical exercise. However, as has been discussed in the literature review section, the fact that rutting can be partly attributed to stripping , coupled with the smooth and rounded nature of a fairly significant proportion of the aggregates has instigated the need to conduct the test and rate the sections based on the subjective evaluation.

3.3.6. Volumetric Analysis

The pertinent procedures outlined in AASHTO, have been adopted to determine the volumetric parameters, Va, VMA, and VFA, of in-place and re-compacted samples. For this purpose, the bulk specific gravity of compacted mixture samples (AASTHO T166), the maximum theoretical specific gravity of the HMA samples (AASTHO T209), the bulk specific gravity of coarse aggregates (AASTHO T85), fine aggregates (AASTHO T84) and filler (AASTHO T100), the bulk specific gravity of combined aggregates, the binder content (AASTHO T182), were all determined, for all of the samples.

Equations 2,3 and 4 as indicated in section -2 have been used to compute the volumetric parameters.

3.3.7. Aggregate gradation

The particle size distribution of the extracted aggregates was determined on the basis of the 0.45 Power Gradation Chart. The 0.45 Power Gradation Chart was plotted with sieve sizes raised to 0.45 power on the X-axis, and percent material passing on the Y-axis. The gradations were then evaluated in comparison with the Maximum Density Curve, for the respective nominal maximum aggregate sizes, which was obtained using the following formula:-

$P = 100 \; -$ ....(6)

Where:

P = % finer than the sieve

d = aggregate size being considered

D = maximum aggregate size to be used

Effect of aggregate gradation (as plotted on the 0.45 Power gradation plots), on the rutting performance of the mixtures, was evaluated on the basis of :-

1. the distance between the aggregate gradations and the maximum density line, for each respective maximum aggregate size, which was determined on the basis of the following formula for each respective sieve size :-

   $$[\text{Difference} = P_x - P_x^m]$$

   Where - P_x is the percent passing sieve size 'x' in the evaluated gradation and,

   P_x^m is the percent passing sieve size'x' on the maximum density line

2. descriptive evaluation of the location of the aggregate gradations in relation to the maximum density line

Table 3:4 Typical 0.45 Power Gradation Plot Data for Maximum aggregate Size of 19mm

Sieve Size (mm)	sieve sizes (mm)raised to 0.45	Max. Density line	% passing of aggregate
A	B =$A^{0.45}$	C = $100*(A/M*)^{0.45}$	D
26.5			
19.0	3.76	100.0	
13.2	3.19	84.9	
9.5	2.75	73.2	
4.75	2.02	53.6	
2.36	1.47	39.1	
1.18	1.08	28.6	
0.60	0.79	21.1	
0.30	0.58	15.5	
0.15	0.43	11.3	
0.075	0.31	8.3	

M* - the maximum aggregate size =19 mm

3.4. Analysis of Data

The general procedure followed for the analysis of the data, primarily consists of performing statistical correlation analysis coupled with subjective evaluation. Linear correlation analysis was adopted to determine whether the dependent variable rut depth was significantly correlated to the independent variables including the in-place and re-compacted Va. VMA, VFA, Marshall Stability, Flow, Marshall quotient and Aggregate angularity. With the limited amount of data reported herein trends only were identified. The correlation analysis was performed by means of Microsoft Excel software, by obtaining linear correlation lines (trend lines) on scatter plots, and determining the sample coefficient of correlation 'r', which is an estimate of the population correlation coefficient 'ρ'. The significance of correlation was then tested at 5% significance level (95% confidence level), to check for the evidence of correlation in the population. However, aberrant observations in the scatter plots were identified as 'outliers', by analyzing the standardized residuals of the regression output. For the purpose of this research, as described in many statistics literatures, outliers have been defined as observations, the absolute value of the standardized residual of which, exceeds two. Moreover, subjective (descriptive) analysis of the correlations observed has been made whenever applicable.

One crucial assumption made during the entire discussion, interpretation and analysis phases of this research is that the effect of binder properties on the rutting performance of all the core-mixtures has been plausibly assumed to be constant, as the same 60/70 penetration grade asphalt was used for the HMA mixtures , for the entire project. Moreover, the effect of the contribution of the binder content , was indirectly taken into consideration, through the volumetric parameters.

The other fundamental assumption made was regarding the research approach of holding all other parameters constant to determine the effect of a single parameter. Regarding this, as the research deals with assessing the rutting performance of an already in-service road, it has been plausibly assumed that the effects of each of the aggregate and mixture properties on rutting will be determined, by systematically selecting sections with progressive rut depths. This way, the ' intrinsic' effects of each of the parameters is expected to be reasonably estimated, as a single road segment, constructed with essentially consistent material sources and workmanship, has been utilized for the case study.

4. RESULTS AND DISCUSSION

4.1. General

Based on the methodology reported in chapter 3, the test result data was used for analysis. An attempt was made to discuss the 'raw' test results initially and then analyze if there exists any relationship between rut depth and the various material and mixture parameters.

4.2. Discussion of Test Results

4.2.1. In-place Marshall and Volumetric Properties

The in-place Marshall and volumetric properties shown in table 4.1 are discussed below.

In-place Va

The in-place Va values are all outside the design range of 3-5%, as stipulated in MS-2, except for section 6, which has an average Va of 3.4%, which conforms to its lowest rut depth of 7.4mm. The section with the minimum in-place Va - section 2 (Average 1.0 % Va) is the fourth most rutted section(rut depth of 28.8mm). While, section 7 which is the most rutted section (rut depth of 87.6mm) has with an average Va of 1.5%, which is the second lowest ranked value.

In-place VMA

The VMA values for most of the sections are outside the design ranges,(for the respective nominal maximum aggregate sizes), except for sections 3 and 4, (with average VMA values of 12.5 and 11.7% respectively), which are ranked as 2nd and 3rd most rutted sections (with rut depths of 48.2mm and 38.6mm respectively). It is difficult to directly assess the VMA values as they are dependent on the actual design nominal maximum aggregate size and original design Va values before rutting or any other change in density or change in the aggregate orientation before the action of traffic.

In-place VFA

The in-place values for most of the sections are above the expected design range (65-75%) , for the respective traffic class , except for sections 1 and 6, with average in-place VFA values of 69% and 64.1%, which are ranked as the 6th and 7th most rutted sections, consistent with the theoretical explanations.

In-place Stability

Sites 2,3,5 and 6 have stability values above the desired design values of 8KN. Site 6 has the maximum value of stability of 14.3KN and is the least rutted section, consistent with theoretical expectations. However, this trend does not hold for the site with the minimum value of stability. Note however that it is misleading to compare the stability values with the design requirements in that, it is difficult to exactly understand the aging effect of the in-service pavement as compared to the laboratory controlled testing procedures during mix design and testing at 60°C temperature. It is also important to note that due to the empiricism of the Marshall mix design and testing procedure, in case of a variation in any of the standard test procedures, the results obtained may not be expected to be reliably explanatory.

In-place Flow

The flow values of all the sections are way outside (lower than) the design range of 8-14 — inch. However this may be understandable, as the in-place cores have been obtained from already aged pavements. However, the sections with the maximum and minimum flow values of 5.78 and 2.45 — inch respectively , are ranked as the most and least rutted sections.

Table 4:1 Marshall and Volumetric Properties of In-place Cores

sampling location	sample ID	% AC by weigh of total mix (**Pb**)	%Va	%VMA	%VFA	Stability (KN)	Flow (0.25mm)	Marshal quotient, MQ (KN/mm)
						Adjusted		
1	1L1	5	2.6	8.04	67.07	4.13	3.52	4.70
	1C	5.3	2.5	8.15	69.87	4.56	3.02	6.03
	1R1	5.2	2.4	7.98	70.19	5.23	2.59	8.07
	AVG.	**5.2**	**2.5**	**8.1**	**69.0**	**4.6**	**3.04**	**6.27**
2	2L1	5	1.1	10.39	89.75	11.05	4.28	10.33
	2L2	6.3	0.8	11.41	92.67	8.97	3	11.96
	2R1	4.3	1.0	9.69	89.41	7.11	3.48	8.18
	AVG.	**5.2**	**1.0**	**10.5**	**90.6**	**9.0**	**3.59**	**10.15**
3	3L1	6.1	2.3	13.07	82.74	9.65	4.23	9.12
	3L2	5.7	1.6	12.15	86.47	6.39	4.16	6.15
	3L3	5.3	2.1	12.19	82.76	11.87	3.24	14.66
	AVG.	**5.7**	**2.0**	**12.5**	**84.0**	**9.3**	**3.88**	**9.98**
4	4L1	5.2	1.8	11.38	84.35	6.06	5.05	4.80
	4R1	5.3	2.2	11.88	81.18	6.00	2.1	11.42
	4R2	5.4	2.2	11.94	81.59	5.00	5.19	3.85
	AVG.	**5.3**	**2.1**	**11.7**	**82.4**	**5.7**	**4.11**	**6.69**
5	5L1	5.7	1.3	9.97	86.73	9.76	4.9	7.97
	5C	5.9	1.4	10.19	86.65	8.30	4.12	8.06
	5R1	6	1.9	10.74	82.75	14.19	2.79	20.34
	AVG.	**5.9**	**1.5**	**10.3**	**85.4**	**10.7**	**3.94**	**12.12**
6	6L1	5.600	3.5	9.54	63.35	11.75	2.88	16.32
	6R1	5.600	3.5	9.57	63.09	13.45	2.37	22.70
	6R2	5.600	3.2	9.22	65.76	17.64	2.11	33.43
	AVG.	**5.6**	**3.4**	**9.4**	**64.1**	**14.3**	**2.45**	**24.15**
7	7L1	5.8	1.4	8.87	83.80	6.14	6.76	3.64
	7L2	5.9	1.6	9.14	82.22	5.73	5.51	4.16
	7R2	6	1.3	8.96	85.23	7.61	5.07	6.01
	AVG.	**5.9**	**1.5**	**9.0**	**83.8**	**6.5**	**5.78**	**4.60**

4.2.2. Re-compacted Marshall and Volumetric Properties

The Re-compacted Marshall and volumetric properties shown in table 4.2 are discussed below.

Re-compacted Va

Sections 1,2 and 4 have re-compacted Va values within the desired range of (3-5)%. The section with the highest re-compacted Va of 7.4%, section 6, is the least rutted section and the section with the lowest re-compacted Va of 1.8%, section 3, is the second most rutted section. However, sections 5,6 and 7 have re-compacted Va values very much in excess of the desirable range of (3-5)%.

Re-compacted VMA

Only sections 2 and 3 have re-compacted VMA values within the desired range (for the respective nominal maximum aggregate sizes and re-compacted Va) .

Re-compacted VFA

Only sections 2 and 4 have re-compacted VFA values within the desired design range of 65-75%. Section 3, which has the highest re-compacted VFA value of 85.65% is the second most rutted section, whereas section 6, whih has the lowest re-compacted VFA value of 44.05% is the least rutted section.

Re-compacted Stability

The observed re-compacted stability values are all above the desired minimum value of 8KN, except for site 1, which has a re-compacted stability value of 7.19KN.

Re-compacted Flow

As expected, the flow values for all the sections are way below the design range of 8-14 — inch, as the sample mixtures were obtained from an already aged pavement. However, a defined trend was observed between re-compacted flow and rut depth,

Table 4:2 Marshall and Volumetric Properties of Re-compacted Cores

sampling location	sample ID	% AC by weigh of total mix (**Pb**)	%Va	%VMA	%VFA	Stability (KN) Adjusted	Flow (0.25mm)	Marshal quotient, MQ (KN/mm)
1	1L2	5.2	4.3	9.79	55.73	7.19	2.33	12.34
2	2R2	5.2	3.3	12.64	73.52	10.20	3.35	12.18
3	3C	5.7	1.8	12.26	85.65	14.45	3.02	19.14
4	4C	5.3	3.7	13.18	72.11	10.05	2.93	13.72
5	5L2	5.9	6.4	14.77	56.51	14.87	2.56	23.23
6	6L2	5.6	7.4	13.16	44.05	12.12	2.42	20.03
7	7C	5.9	6.7	13.86	51.43	12.22	2.89	16.92

4.2.3. Coarse Aggregate Angularity

Even though ERA Standard Technical Specification - 2002, stipulates that at least 95% of all particles used for bituminous surfacing mixtures shall have at least three fractured faces, the standard test procedure for the determination of coarse aggregate angularity as outlined in ASTM designation D 5821, uses the distinguishing criteria of 'one or more' or 'two or more' fractured faces. In all cases, the percent fractured faces for all the sections are all below the 95% margin. In fact, the site with the maximum percent of particles with two or more crushed faces is section two, which has a value of 91.6%.

Table 4:3 Percent Fractured Particles in Coarse Aggregates

sampling location	sample ID	% by weight of aggregates (19-9.5)mm	% of parcticles with 2 or more fractured faces	% by weight of aggregates (9.5 - 4.75)mm	% of parcticles with 2 or more fractured faces	Total % crushed particles
1	1L1	63.0%	83.3%	37.0%	91.4%	86.3%
2	2L2	61.3%	89.9%	38.7%	94.4%	91.6%
3	3L2	55.9%	81.9%	44.1%	93.3%	86.9%
4	4L1	64.7%	90.4%	35.3%	89.0%	89.9%
5	5L1	57.6%	83.7%	42.4%	88.7%	85.8%
6	6L1	66.0%	79.0%	44.0%	82.1%	88.3%
7	7L1	64.6%	79.9%	35.4%	81.7%	80.5%

4.2.4. Aggregate-Bitumen Coating and Stripping

Based on the AASHTO T-182-84, subjective evaluation test procedure, after immersion in water for 16-18hrs, all of the samples had coated area of less than 95%. However, subjectively evaluated (qualitatively and visually), the samples had varying degree of stripping, as shown in table 4.4 below.

Table 4:4 Coating - stripping of aggregate bitumen mixtures

Sampling location ID	Coated Area (%)	Rank in ascending degree of aggregate-bitumen stripping (visual)
1	< 95%	4
2	< 95%	6
3	< 95%	7
4	< 95%	3
5	< 95%	2
6	< 95%	1
7	< 95%	5

4.3. Analysis and Interpretation of Results

4.3.1. Effect of Aggregate Gradation

As described in the methodology section, effect of aggregate gradation (as plotted on the 0.45 Power gradation plots, as shown in Appendix J), on the rutting performance of the mixtures, has been evaluated on the basis of :-

i. the distance between the aggregate gradations and the maximum density line, for each respective maximum aggregate size-

ii. descriptive evaluation of the location of the aggregate gradations in relation to the maximum density line

As shown in table 4.5 and figure 4.1, based on the correlation analysis between the distance of aggregate gradations from the maximum density line and rut depth, even though there appears to be a haphazard trend and no linear correlation, the scatter plot data shows that the observed rut depths decreased with increase in the absolute distance from the maximum density line, upto the value of 49.9, and dramatically increased with increase in the absolute distance from the maximum density line, beyond that value; excluding the data point for the most severely rutted section-7. This is of course consistent to the theoretical expectations as outlined in the literature review.

Table 4:5 Evaluation of aggregate gradations in reference to the MDL

Sampling Location	Sample ID	Max. Aggr. Size (mm)	Nom. Max. Aggr. Size (mm)	Sum of Absolute Distance from the MDL	Average Distance from the MDL for Section	Descriptive Evaluation of Gradation in Reference to the MDL		
						Upto the 4.75mm sieve size	4.75 - 9.5mm sieve size	Beyond the 9.5mm sieve size
1	1L1	26.5	19	32.68	42.37	**close** but on the **coarser side**	**very close** but on the **finer side**	on the **finer side**
	1C	26.5	19	34.64		close but on the **coarser side**	**very close** but on the **finer side**	on the **finer side**
	1R1	26.5	19	59.79		on the **finer side**	on the **finer side**	on the **finer side**
2	2L1	26.5	19	32.04	41.57	**close** but on the **coarser side**	on the **coarser side**	on the **finer side**
	2L2	26.5	19	49.21		on the **coarser side**	on the **coarser side**	on the **finer side**
	2R1	26.5	19	43.45		close but on the **finer side**	**very close** but on the **finer side**	on the **finer side**
3	3L1	26.5	19	80.57	73.25	on the **finer side**	on the **finer side**	on the **finer side**
	3L2	26.5	19	54.78		on the **finer side**	on the **finer side**	on the **finer side**
	3R2	26.5	19	84.39		on the **finer side**	on the **finer side**	on the **finer side**
4	4L1	26.5	19	71.90	61.28	on the **finer side**	**very close** but on the **finer side**	on the **finer side**
	4R1	26.5	19	45.41		on the **finer side**	**very close** but on the **finer side**	on the **finer side**
	4R2	26.5	19	66.53		on the **finer side**	**very close** but on the **finer side**	on the **finer side**
5	5L1	26.5	19	41.57	41.95	on the **finer side**	**very close** but on the **finer side**	on the **finer side**
	5C	26.5	19	46.61		**coincident** wih the MDL	**coincident** wih the MDL	on the **finer side**
	5R1	26.5	19	37.67		**coincident** wih the MDL	**coincident** wih the MDL	on the **finer side**
6	6L1	26.5	19	33.31	49.93	**close** but on the **coarser side**	**coincident** wih the MDL	on the **finer side**
	6R1	37.5	26.5	61.79		**coincident** wih the MDL	**close** but on the **finer side**	on the **finer side**
	6R2	37.5	26.5	54.70		**coincident** wih the MDL	**coincident** wih the MDL	on the **finer side**
7	7L1	26.5	19	38.57	47.43	**coincident** wih the MDL	**coincident** wih the MDL	on the **finer side**
	7L2	37.5	26.5	66.87		on the **finer side**	on the **finer side**	
	7R2	26.5	19	36.85		**coincident** wih the MDL	**coincident** wih the MDL	on the **finer side**

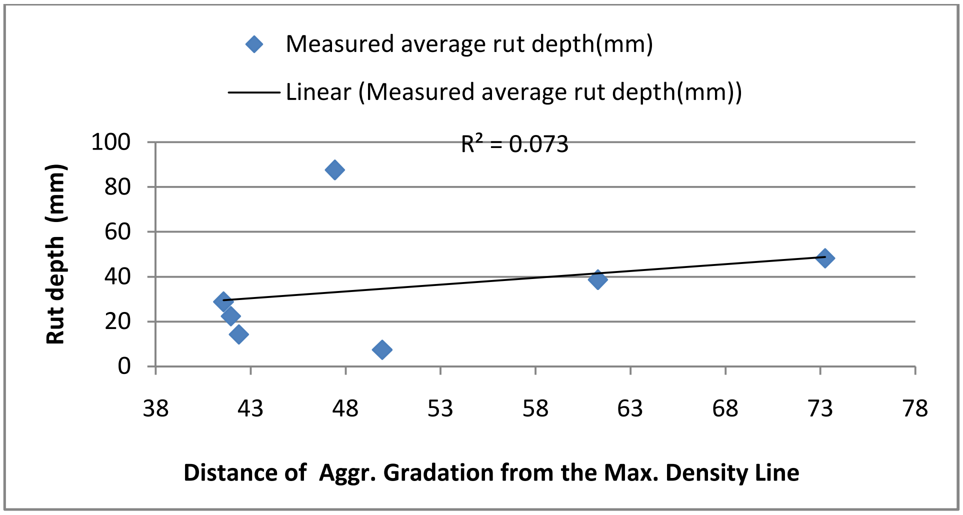

Figure 4-1 Distance from MDL of aggregate gradation Vs Rut depth

Based on the descriptive evaluation of the location of aggregate gradations (as plotted on the 0.45 Power Gradation Curves, for each sampling section, as shown in Appendix J), in relation to the maximum density line, it was observed :-

i. the most rutted sections (sections 7, 3 and 4), have aggregate gradations, which in the sieve size ranges finer than the 4.75mm sieve size, plot predominantly on the finer side of the maximum density line; and over the sieve size ranges between the 4.75 -9.5mm sieve sizes plot on the finer side to almost coincidental with the maximum density line.

ii. The least rutted sections (sections 6, 1 and 5) have aggregate gradations, which over the sieve size ranges finer than the 4.75mm sieve size, plot predominantly on the coarser side of the maximum density line; and over the sieve size ranges between the 4.75 - 9.5mm sieve sizes plot almost coincidental with the maximum density line.

iii. Over the range beyond the 9.5mm sieve size, all sections (most rutted as well as least rutted) have aggregate gradations which plot on the finer side of the maximum density line.

Moreover the filler contents and filler-asphalt ratios of each of the sampling sections have been summarized below.

Sampling section-1

- The filler contents of samples **1L1**, **1C** and **1R1** are 5.1%, 5.4% and 3.7% respectively
- The filler to asphalt ratios of samples **1L1**, **1C** and **1R1** are 0.96, 0.96 and 0.68 respectively

Sampling section-2

- The filler contents of samples **2L1** , **2L2** and **2R1** are 6.2, 5.7 and 7.9 respectively

- The filler to asphalt ratios samples **2L1** , **2L2** and **2R1** are 1.18, 0.85 and 1.76 respectively

Sampling section-3

- The filler contents of samples **3L1**, **3L2** and **3R2** are 10.8%, 6.5% and 9.9% respectively
- The filler to asphalt ratios of samples **3L1**, **3L2** and **3R2** are 1.67, 1.08 and 1.77 respectively

Sampling section-4

- The filler contents of samples **4L1**, **4R1** and **4R2** are 16.9%, 11% and 13.8% respectively
- The filler to asphalt ratios of samples **4L1**, **4R1** and **4R2** are 3.08 1.96 and 2.42 respectively

Sampling section-5

- The filler contents of samples **5L1**, **5C** and **5R1** are 9.0%, 5.1% and 4.2% respectively
- The filler to asphalt ratios of samples **5L1**, **5C** and **5R1** are 1.49, 0.82 and 0.66 respectively

Sampling section-6

- The filler contents of samples **6L1**, **6R1** and **6R2** are 1.8% 1.9% and 3.0% respectively
- The filler to asphalt ratios of samples **6L1**, **6R1** and **6R2** are 0.31, 0.33 and 0.5 respectively

Sampling section-7

- The filler contents of samples **7L1**, **7L2** and **7R2** are 7.6%, 5.5% and 4.3% respectively
- The filler to asphalt ratios of samples **7L1**, **7L2** and **7R2** are 1.24, 0.87 and 0.68 respectively

From the above results, as the maximum aggregate sizes, and gradations are varying for the different samples in the sampling sections, it is difficult to solely evaluate the effect of filler content and filler-asphalt ratios on rutting. However, these parameters will be evaluated along with the other volumetric properties below.

Summing up, the above evaluations, it can be plausibly concluded that mixtures, which have aggregate gradations, which, over the sieve size ranges finer than the 4.75mm, plot predominantly on the coarser side of the maximum density line; and over the sieve size ranges between the 4.75 -9.5mm sieve sizes plot almost coincidental with the maximum density line; and where the sum of absolute distance from the gradation plots to the maximum density line, is acceptably but not extremely higher, (preferably not exceeding 49.9), performed better with regards to rutting. It is noteworthy that, according to the literatures reviewed , the desired gradation with regards to better rutting performance is, where the grading is expected to be coarser in the finer fractions upto around the 2.36mm sieve size and finer in the coarser fractions from around 9.5mm sieve size upwards relative to the 0.45 Power Grading (Maximum Density Line).

4.3.2. Effect of In-place Marshall and Volumetric Properties

The effects of in-place Marshall and Volumetric parameters have been discussed below. Only the major correlation and significance test results have been depicted here. The regression outputs and residual analysis results used for identifying outliers have been presented in Appendix M.

4.3.2.1. Effect of In-place Va

As shown in figure 4.2, moderate correlation is observed between In-place Va and rutting, with a coefficient value 'r' of -0.5, which is not justified at 5% significance level, for the given sample size. However, a definite trend is not observed. The in-place Va values are all less than 3.5%, and most of the excessive depths of rut (>40mm) were observed for in-place Va values of approximately less than 2%. However, this trend is not "entirely" consistent as rut depths of less than 40mm were observed for in-place Va's of 1.0% and 1.5% at sections 2 and 5 respectively. The section with the maximum in-place Va of 3.4%(which is close to the desired in-place Va, after years of traffic), i.e., section 6 is the least rutted section. This is somewhat consistent with the minimum in-place VFA value and low filler- asphalt ratios of 0.31, 0.33 and 0.5. However, the same consistency is not observed for the section with the minimum in-place Va.

There is no clear trend observed between the in-place Va and in-place VMA values except for sites 7,4 and 5. Especially section 7 which has the second least in-place Va value and with the second least VMA value.

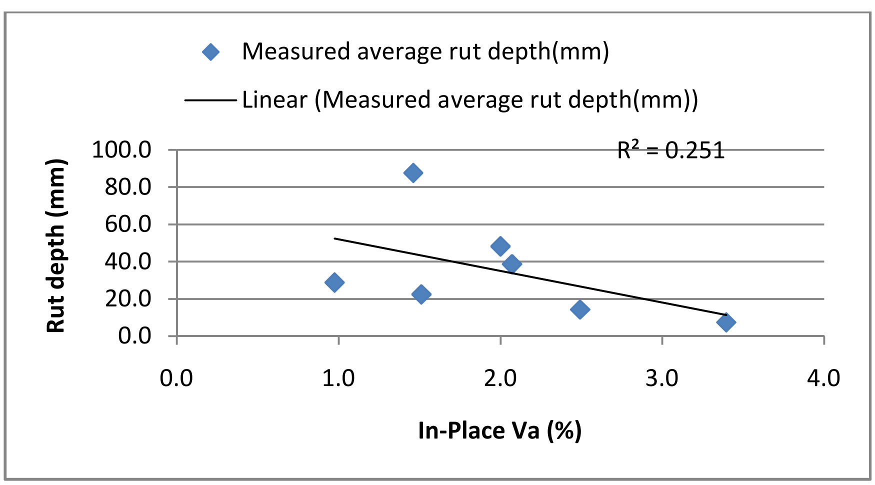

Figure 4-2 In-place Va Vs Rut depth

Table 4:6 Significance Test for Correlation of In-place Va Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
-0.50	7	-1.30	0.05	5	2.571	Accept Ho (there is no evidence of correlation

4.3.2.2. Effect of In-Place VMA

As shown in figure 4.3, very weak correlation is observed between In-place VMA and rutting, with a coefficient value 'r' of 0.14. However, a trend is observed from the scatter plot, which indicates that the rut depth increases with increase in in-place VMA(especially excluding the data point for section-7,which is a statistically justified outlier, figure 4.4). This is in complete contradiction with the theoretical expectation that the rut susceptibility decreases with increase in the VMA value (keeping other parameters aside). It is very difficult to draw plausible conclusion regarding the quantitative (and limiting) relations because the VMA requirement is dependent on the actual design Nominal Maximum Aggregate size and the actual design Va value, before any change in the aggregate fabric (interlock position) has occurred. However, there's a definite consistency between the maximum in-place VMA value and the maximum re-compacted VMA values for section 1, which is the 2nd least rutted section.

Note: As shown in figure 4.5, an intermediate trend has been observed between the observed in-place VMA values and the distance of aggregate gradation plots from the MDL, with a coefficient of correlation 'r' of 0.48, where the in-place VMA increases with increase in the distance of the aggregate gradation plot from the maximum density line; which is consistent with the theoretical expectations.

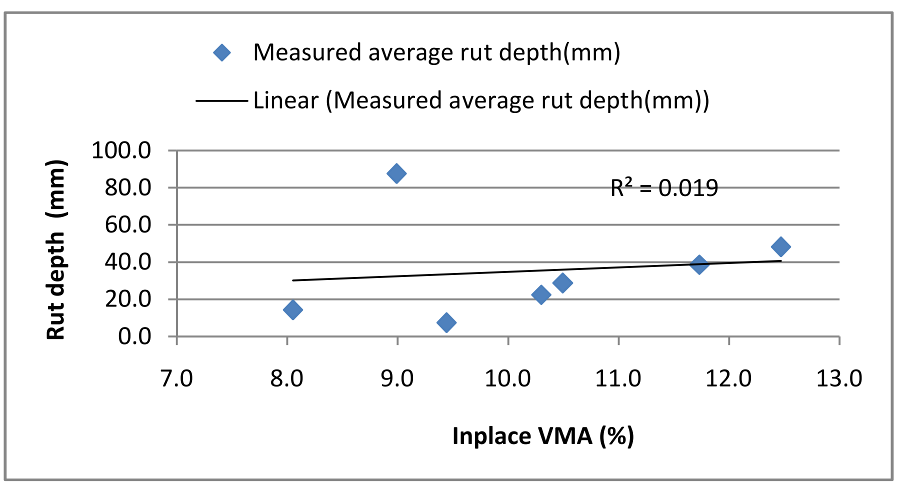

Figure 4-3 In-place VMA Vs Rut depth

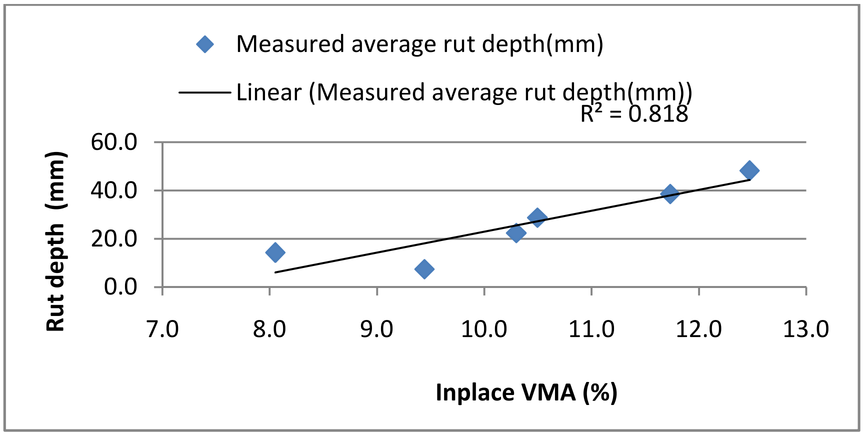

Figure 4-4 In-place VMA Vs Rut depth (excluding outlier data point - section 7)

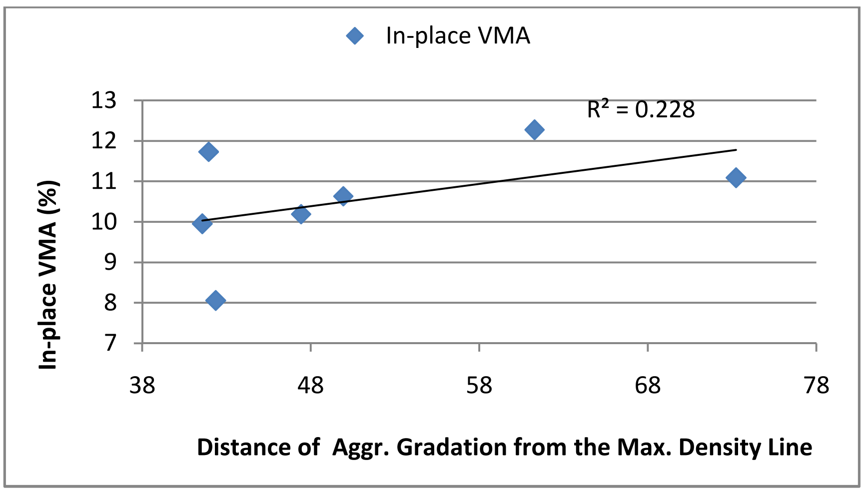

Figure 4-5 Distance from MDL of aggregate gradation Vs In-place VMA

4.3.2.3. Effect of In-place VFA

As shown in figure 4.7, excluding the statistically justified outlier data point - section 7, strong correlation is observed between in-place VFA and rut depth, with a coefficient of correlation of 0.71, which is however not justified at 5 % significance level, for the given sample size. A trend is somewhat observed, where the rut depth increases with increase in in-place VFA. As mentioned in the discussion of test results part, the in-place VFA values for most of the sections is above the desired design range of 65-75% (for traffic class of above 1 million ESA's), except for sites 1 and 6, which have average in-place VFA values of 69% and 64.1%, which are the 2nd and 1st least rutted sections, consistent with the theoretical expectations.

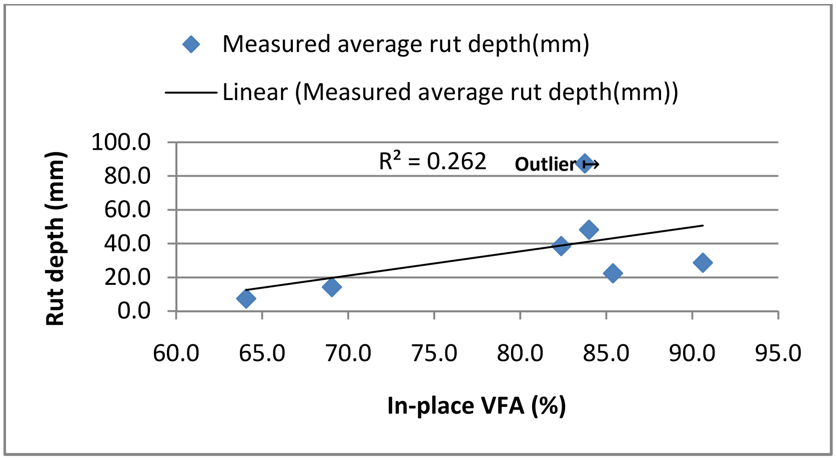

Figure 4-6 In-place VFA Vs Rut depth

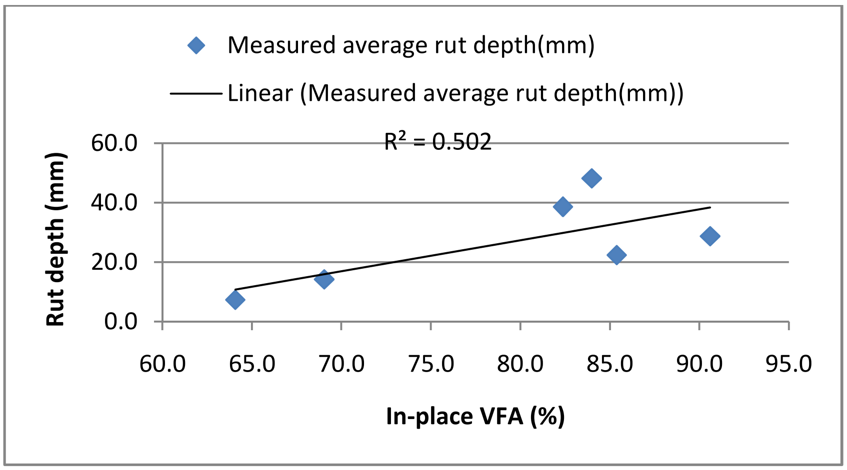

Figure 4-7 In-place VFA Vs Rut depth (excluding outlier data point - section 7)

Table 4:7 Significance Test for Correlation of In-place VFA Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, r	sample size, n	Test statistic, t	Significance level , α	Degrees of freedom, df = n-2	region of acceptance of H0 , tα/2,n-2	Acceptance/ Rejection of H0
0.71	6	2.01	0.05	4	2.776	Accept Ho (there is no evidence of correlation

4.3.2.4. Effect of In-place Stability

As shown in figure 4.8, moderate to weak correlation was observed between in-place stability and rut depth, with a coefficient of correlation of 0.4, which is still not justified at 5 % significance level, for the given sample size. However, A trend is somewhat observed, where the rut depth increases with decrease in in-place stability. As mentioned in the discussion of test results part, only sections 2,3,5 and 6 have stability values above the desired design value of 8KN. Section 7 , which is the least rutted section has the maximum in-place stability value of 14.3KN. However, this trend does not hold for the section with the minimum value of in-place stability- section 1.

Note: however that it is misleading to compare the in-place values with the design requirements in that, it is difficult to exactly understand (quantify) the aging effect of the in-service pavement as compared to the laboratory controlled testing procedures during mix design at 60^0C, as outlined in MS-2 Manual. It is not to be forgotten that due to the empiricism of the Marshall Mix Design and testing procedure, in the case of a variation in any of the standard test procedures the results obtained may not be expected be reliably explanatory.

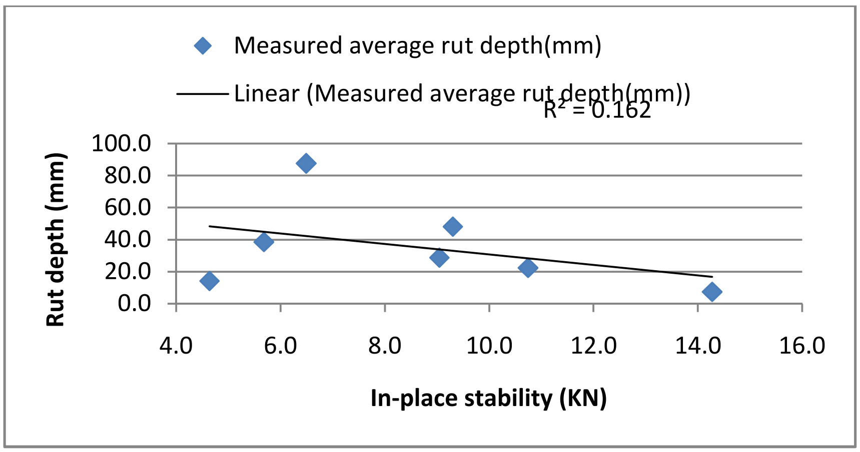

Figure 4-8 In-place Stability Vs Rut depth

Table 4:8 Significance Test for Correlation of In-place Stability Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
-0.40	7	-0.986	0.05	5	2.571	Accept Ho (there is no evidence of correlation

4.3.2.5. Effect of In-place Flow

As shown in figure 4.9, a strong correlation was observed between in-place flow and rut depth, with a coefficient of correlation of 0.95, which is justified at 5 % significance level, for the given sample size. A defined trend is observed where the rut depth increases with increase in in-place flow. As expected sections 7 and 6, which have the maximum and minimum flow values of 5.78 and 2.45 — inch respectively, are ranked as the most and least rutted sections.

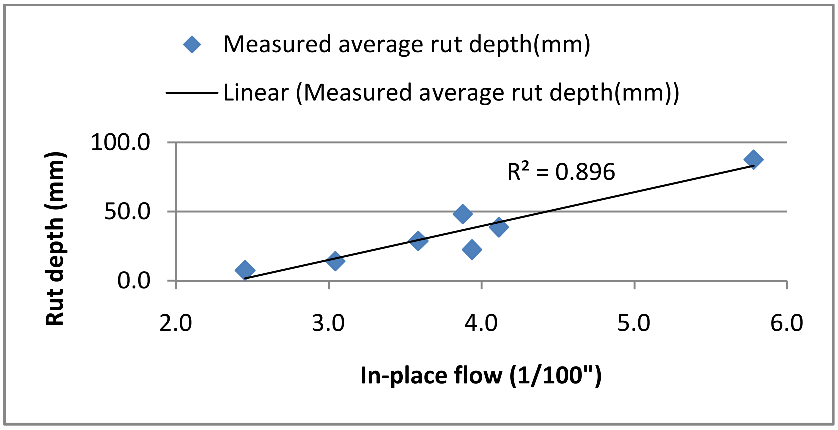

Figure 4-9 In-place Flow Vs Rut depth

Table 4:9 Significance Test for Correlation of In-place Flow Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, r	sample size, n	Test statistic, t	Significance level , α	Degrees of freedom, df = n-2	region of acceptance of H0 , tα/2,n-2	Acceptance/ Rejection of H0
0.95	7	6.577	0.05	5	2.571	Reject Ho (there is evidence of correlation)

4.3.2.6. Effect of In-place Marshall Quotient

As shown in figure 4.10, moderate correlation was observed between in-place Marshall quotient and rutting, with a coefficient of correlation 'r' value of 0.6, which was not justified at 5% significance level, for the given sample size. A trend is somewhat observed where the rut depth increases with a decrease in the in-place Marshall Quotient.

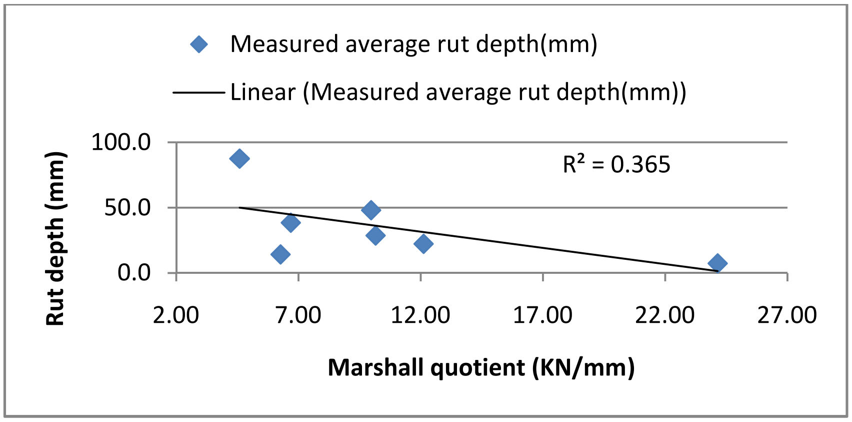

Figure 4-10 In-place Marshall Quotient Vs Rut depth

Table 4:10 Significance Test for Correlation of In-place Marshall Quotient Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
-0.60	7	-1.698	0.05	5	2.571	Accept Ho (there is no evidence of correlation

4.3.3. Effect of Re-compacted Marshall and Volumetric Properties

The effects of re-compacted Marshall and Volumetric parameters have been discussed below. Only the major correlation and significance test results have been depicted here. The regression outputs and residual analysis results used for identifying outliers have been presented in Appendix H.

4.3.3.1. Effect of Re-compacted Va

As shown in figure 4.12, a strong correlation was observed between re-compacted Va and rut depth, with a coefficient of correlation 'r' of 0.84, excluding the statistically justified outlier data point- section 7, which was justified at 5 % significance level. A defined trend is observed where the rut depth decreases with increase in re-compacted Va. Excessive depths of rut (> 40mm) were observed when the re-compacted Va values were less than approximately 3.3%. As mentioned in the test result discussion part, the section with the highest re-compacted Va value, section - 6, is the least rutted section, and the section with lowest re-compacted Va value, section 3, is the second most rutted section. However, sections 5,6 ad 7 all have re-compacted Va values very much in excess of the maximum desirable value of 4%.

Note: The section with the minimum rut depth of 7.4mm , section 6, has a re-compacted Va value of 7.4. This value is very high compared to what is expected from the Marshall mix design assumption that 4% Va is expected after years of traffic compaction (as simulated by the 75 blow Marshall compaction). This is also somewhat consistent with the slightly higher re-compacted VMA value of 13.16% for the same section. This may somewhat be justified with the minimum re-compacted VFA value of 44.05%, and low filler- asphalt ratios of 0.31, 0.33 and 0.5. This is also consistent with the in-place maximum in-place value of 3.4% and minimum in-place VFA alue of 64.1% for the same section (section 9).

Also note that sections 3,2 and 7 which have the highest, 2nd highest and 2nd least re-compacted Va values respectively, have the 2nd highest, 3rd highest and 2nd least re-compacted VMA values, respectively. This shows that a trend somewhat exists where the re-compacted Va increases with an increase in re-compacted VMA.

Another interesting observation made here is the correlation between the re-compacted Va and the distance of aggregate gradations from the maximum density line. As can be observed from figure 4.13, even though the linear trend line indicates that there is a negative correlation between the distance of aggregate gradation from the MDL and re-compacted Va, the scatter plot somehow indicates that there is a non-linear relationship, where the re-compacted Va increases with increase in the distance of aggregate gradation from the MDL upto around the 49.9 distance from MDL value and dramatically reduces beyond that; which is consistent with the correlation between the distance of aggregate gradations from MDL and rutting.

One question which could be raised here is why the vast difference between the in-place Va values and the re-compacted Va values. The most likely explanation for this would be the nature and effect of compactive effort. It is known that at the same asphalt content both the Va and VMA decrease with increase in compactive effort. In this case, the 75-blow marshall compaction (both sides), which is expected to be equivalent to traffic loading in excess of 1 million ESA's, was adpoted for re-compacting the HMA core samples. However, as discussed earlier, this compactive effort is significantly less compared to the actual traffic loading experienced by the pavement (up to the time of sampling), which is significantly greater than 2.6 million ESA's (the cumulative equivalent standard axle experienced by the road in over the years 2012 and 2013 only) This leads to the conclusion that the 75 blow , Marshall compactive effort, is not sufficient anymore, to simulate the current levels of high traffic loadings, which are very much in excess of one million ESA's. Of course, the effect of high pavement temperatures should not be underestimated in this regard. In such cases, in the absence (lack) of the more sophisticated superpave mix design methods, the need to adopt the density refusal approach (a modification of of the Marshall Mix Design method, which ensures that a mix designed with 75 blow compaction also does not compact to lower that approximately 3%, Va , when compacted further by traffic) becomes inevitable.

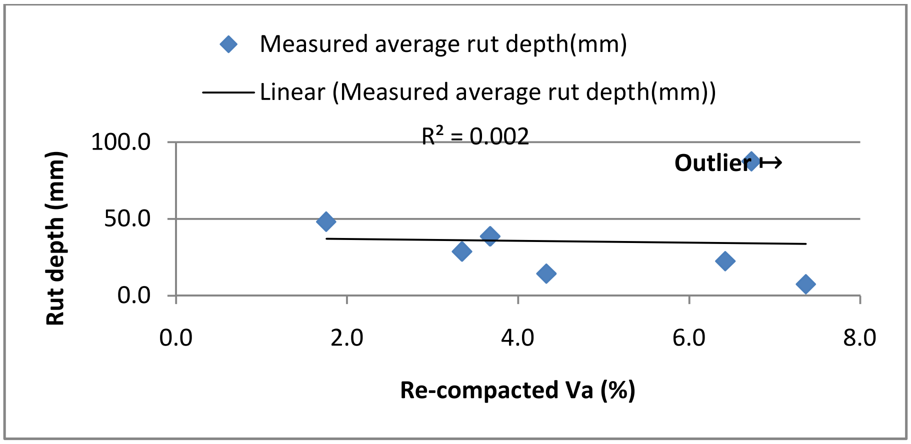

Figure 4-11 Re-compacted Va Vs Rut depth

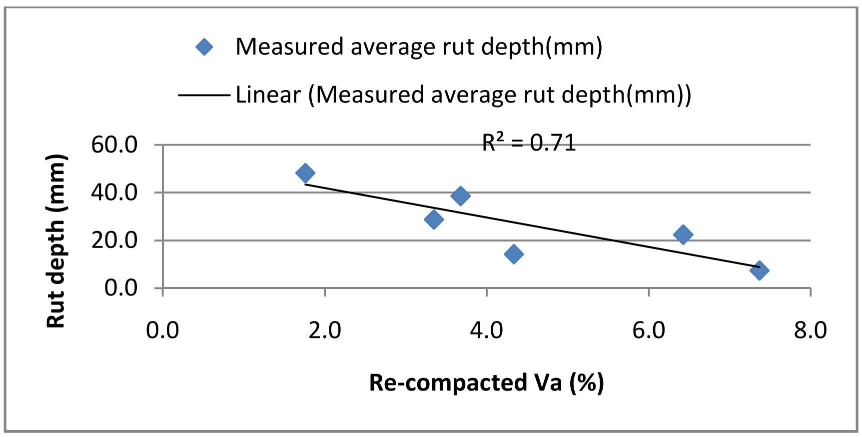

Figure 4-12 Re-compacted Va Vs Rut depth (excluding outlier data point - section 7)

Table 4:11 Significance Test for Correlation of Re-compacted Va Vs Rut depth (excluding outlier data point - section 7)

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
-0.84	6	-3.129	0.05	4	2.776	Reject Ho (there is evidence of correlation)

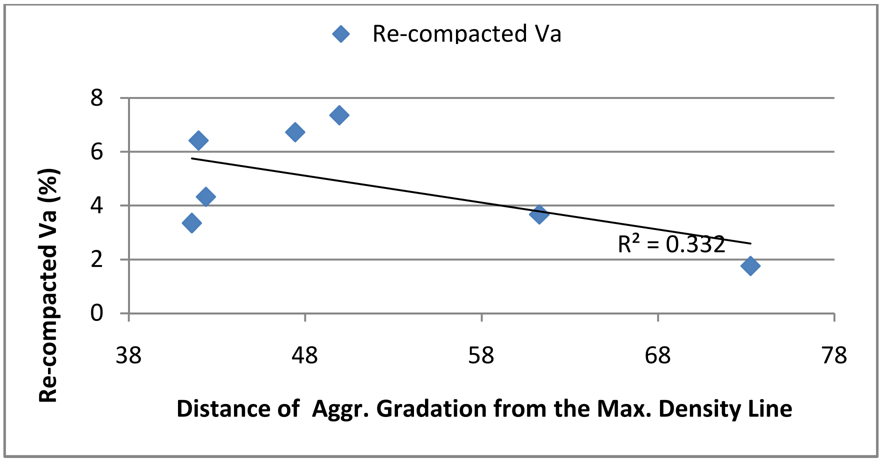

Figure 4-13 Distance from MDL of aggregate gradation Vs Re-compacted Va

4.3.3.2. Effect of Re-compacted VMA

As shown in figure 4.14, a weak correlation was observed between re-compacted VMA and rut depth, with a correlation coefficient 'r' of 0.31 , and no definite trend was observed, despite theoretical expectations.

As per the literatures reviewed, it is known that gradations which plot on either side of the maximum density line are capable of yielding higher VMA values. However, no correlation was observed between the re-compacted VMA values and the distance from the MDL of aggregate gradations.

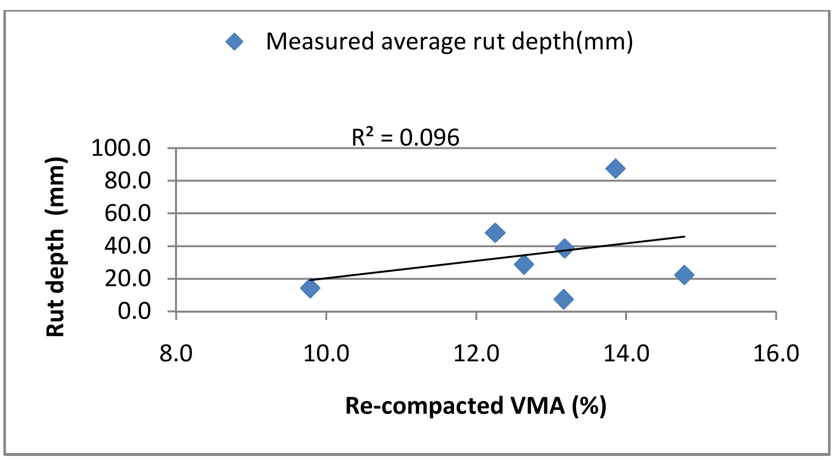

Figure 4-14 Re-compacted VMA Vs Rut depth

Table 4:12 Significance Test for Correlation of Re-compacted VMA Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
0.31	7	0.729	0.05	5	2.571	Accept Ho (there is no evidence of correlation

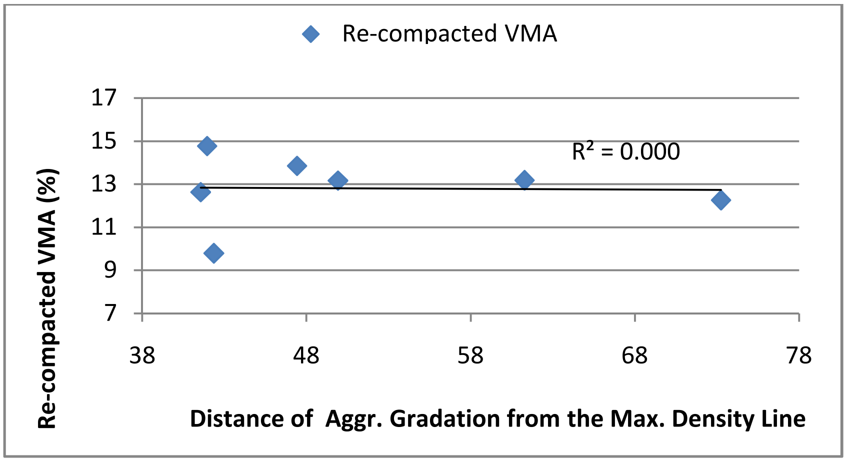

Figure 4-15 Distance from MDL of aggregate gradation Vs re-compacted VMA

4.3.3.3. Effect of Re-compacted VFA

As shown in figure 4.17, strong correlation was observed between re-compacted VFA and rut depth with a correlation coefficient 'r' of 0.96, excluding the statistically justified outlier data point - section 7, which was justified at 5 % significance level. Definite trend was observed where the rut depth increases with an increase in the re-compacted VFA. Excessive rut depths (>40mm) were observed for re-compacted VFA values of approximately 72.1%.

Note: that section 3, which has the highest re-compacted VFA value of 85.65% is the 2nd most rutted section, whereas section 6, which is with the lowest re-compacted VFA value of 44.5% is the least rutted section. The minimum values for the re-compacted and in-place VFA are also consistent for section 6. Excessive rut depths(>40mm) were observed where the re-compacted VMA was in excess of 72.1% (note that as per the MS-2 manual, the recommended range of mix design VFA values for traffic in excess of 1million EAL is 65-75%). It is also worthwhile noting that the VFA criteria is used to limit the maximum levels of VMA and maximum levels of asphalt content during mix design, and that the re-compacted properties are an estimate of the mix design properties.

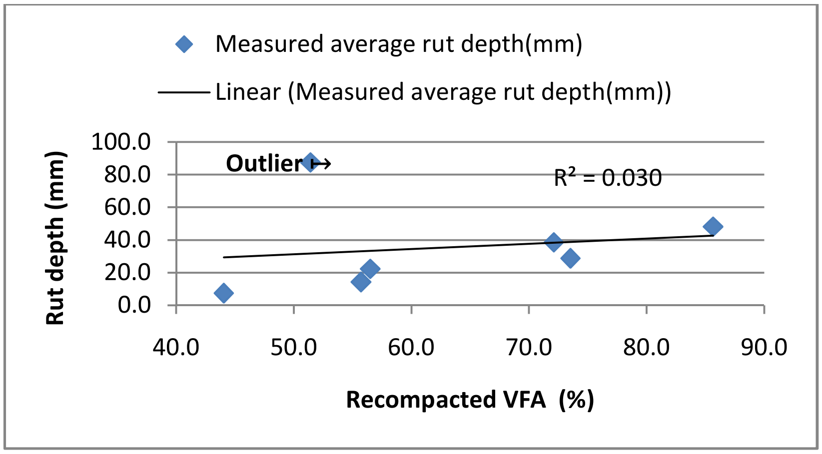

Figure 4-16 Re-compacted VFA Vs Rut depth

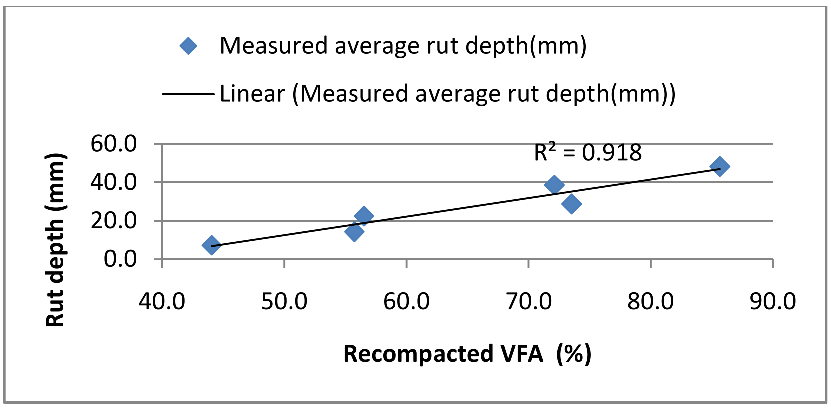

Figure 4-17 Re-compacted VFA Vs Rut depth (excluding outlier data point - section 7)

Table 4:13 Significance Test for Correlation of Re-compacted VFA Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
0.96	6	6.727	0.05	4	2.776	Reject Ho (there is evidence of correlation)

4.3.3.4. Effect of Re-compacted Stability

As shown in figure 4.18, weak correlation was observed between re-compacted Stability and rut depth with a correlation coefficient 'r' of 0.25, and no defined trend was observed. However the observed re-compacted stability values for all of sections, are all above the expected design limit of 8KN, except for section 1, (which has a re-compacted stability value of 7.2 KN). It is likely that the measured higher re-compacted stability values could in general be attributed to the higher air voids (especially for the good performing sections) and hence the higher rate of oxidation. Also note that Marshall Stability values from laboratory compacted samples during original construction are expected to be lower than that for aged mixes, on account of the inevitably expected, some extent of irreversible aging as a result of oxidation at molecular level of the asphalt binder.

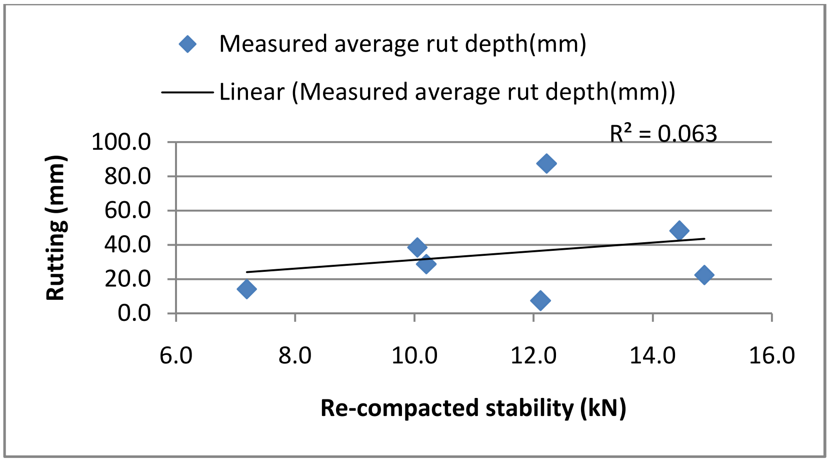

Figure 4-18 Re-compacted Stability Vs Rut depth

Table 4:14 Significance Test for Correlation of Re-compacted Stability Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
0.25	7	0.584	0.05	5	2.571	Accept Ho (there is no evidence of correlation

4.3.3.5. Effect of Re-compacted Flow

As shown in figure 4.20, strong correlation was observed between re-compacted flow and rut depth, excluding the statistically justified outlier data point - section 7, with a correlation coefficient 'r' of 0.72, which is however not justified at 5 % significance level. A trend is somewhat observed where rut depth increases with increase in re-compacted flow.

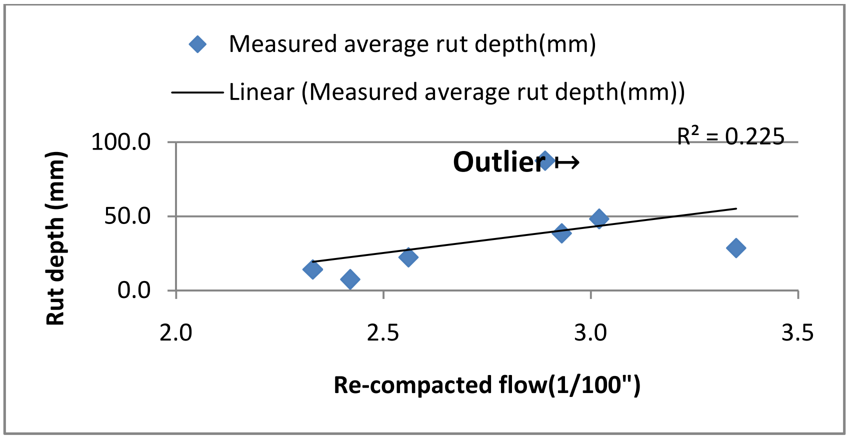

Figure 4-19 Re-compacted Flow Vs Rut depth

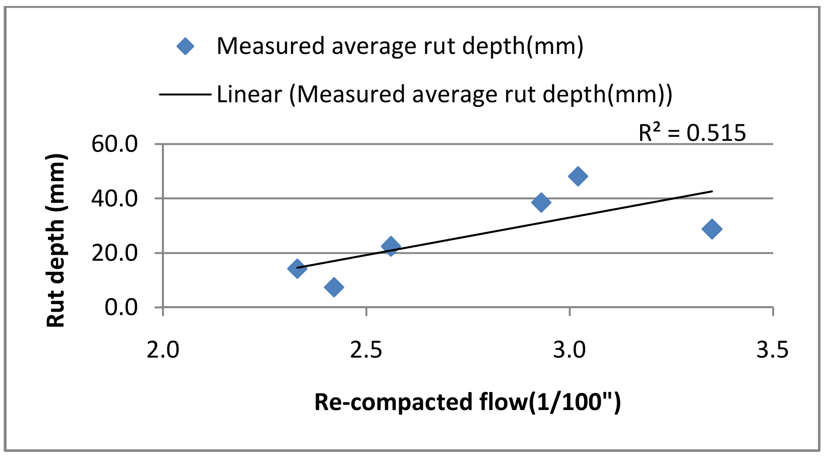

Figure 4-20 Re-compacted Flow Vs Rut depth (excluding outlier data point - section 7)

Table 4:15 Significance Test for Correlation of Re-compacted Stability Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : $\rho = 0$ (no correlation exists) , Ho : $\rho \neq 0$ (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
0.72	6	2.06	0.05	4	2.776	Accept Ho (there is no evidence of correlation

4.3.3.6. Effect of Re-compacted Marshall Quotient

As shown in figure 4.21, no correlation was observed between re-compacted Marshall Quotient and rut depth with a correlation coefficient 'r' of 0.03, and no trend what so ever.

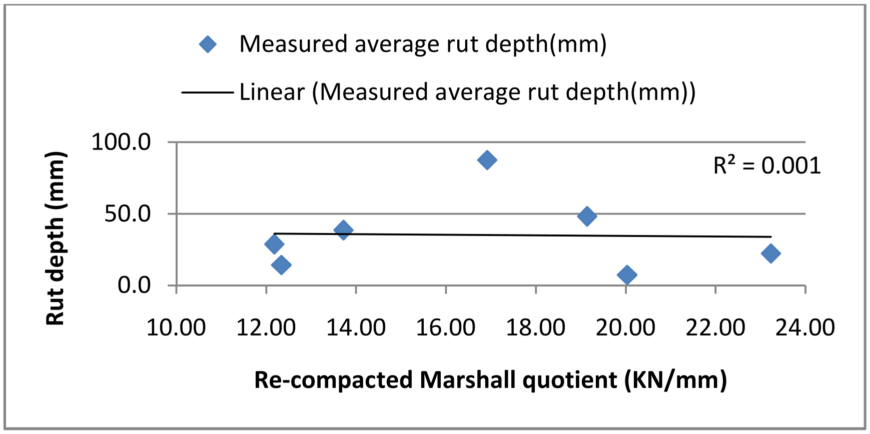

Figure 4-21 Re-compacted Marshall Quotient Vs Rut depth

Table 4:16 Significance Test for Correlation of Re-compacted MQ Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
-0.03	7	-0.075	0.05	5	2.571	Accept Ho (there is no evidence of correlation

4.3.4. Effect of Aggregate Angularity

As shown in figure 4.22, moderate correlation was observed between coarse aggregate angularity and rut depth with a correlation coefficient 'r' of -.064, which was however not justified at 5% significance level. A very ' vague' trend was observed which somehow shows that the rut depth decreases with increase in % coarse aggregate angularity (percentage of coarse aggregates with two or more crushed faces).

With regards to the correlation of VMA with angularity there is no clearly consistent trend. However, moderate correlation was observed in that:-

- section 4, which is ranked as the section with the 2nd highest percentage of coarse aggregate angularity, is also ranked as the section wit the second highest % in-place VMA and the 3rd highest % re-compacted VMA
- Section 1, which is ranked as the section with the 3rd lowest % coarse aggregate angularity is also the section with the least in-place VMA as well as least re-compacted VMA
- Section 7, which is ranked as the section with the least % coarse aggregate angularity is also the site with the 2nd least in-place VMA.

- Sections 2 and 5, also somehow conform with regards to the % coarse aggregate angularity and in-place VMA.

In General, percentage of coarse aggregate angularity and percent in-place VMA, show better trends of positive correlation.

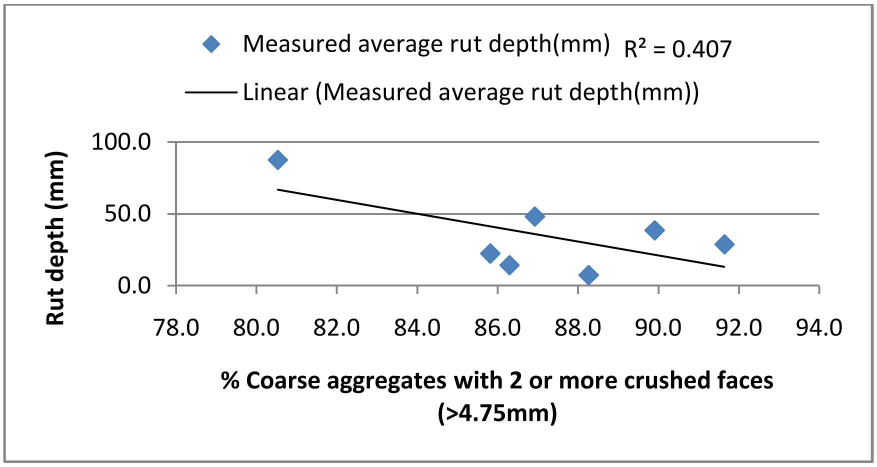

Figure 4-22 Coarse Aggregate Angularity Vs Rut depth

Table 4:17 Significance Test for Correlation of Coarse Aggregate Angularity Vs Rut depth

Significance test for correlation at 5% significance level ,{Ho : ρ = 0 (no correlation exists) , Ho : ρ ≠ 0 (correlation exists)}						
Sample correlation coefficient, **r**	sample size, **n**	Test statistic, **t**	Significance level , α	Degrees of freedom, **df = n-2**	region of acceptance of H_0 , $t_{\alpha/2,n-2}$	Acceptance/ Rejection of H_0
-0.64	7	-1.855	0.05	5	2.571	Accept Ho (there is no evidence of correlation

4.3.5. Effect of Aggregate-Bitumen Coating and Stripping

As shown in figure 4.23 clear 'linear' trend was not evident between degree of aggregate bitumen stripping and rutting. However, there's a general trend showing that the rut depth increases with increase in the rate of stripping (i.e. based on the qualitative ranking with respect to rate of stripping). The section with the lowest rate of stripping (section 6), is the section with the lowest depth of rut. Note the following in this regard:-

- the section with the lowest degree of stripping (section 6) is the section with the lowest depth of rut
- the section with the highest degree of stripping (section 3) is the section with the 2nd highest depth of rut

It is difficult to directly attribute rutting with the mere effect of degree of aggregate bitumen stripping. The fact that the test procedure is purely subjective (qualitative) evaluation also strengthens the validity of this very thought. However, in the case of moisture intrusion through liable areas such as worn-out shoulders, rutting can be partially attributed stripping, especially when this situation is coupled with localized shoving and aggregates of less angularity. Sections 7,3 and 2 which have exhibited higher degree of stripping, coincidental shoving and relatively lower coarse aggregate angularity(especially sections 7 an 3), are also the most rutted sections.

On the other hand, It is also difficult to directly correlate degree of stripping with percent fractured face of coarse aggregates. However, sections 4 and 1 have a 'nearly' consistent ranking with regards to their degree of stripping and percent crushed face of coarse aggregates.

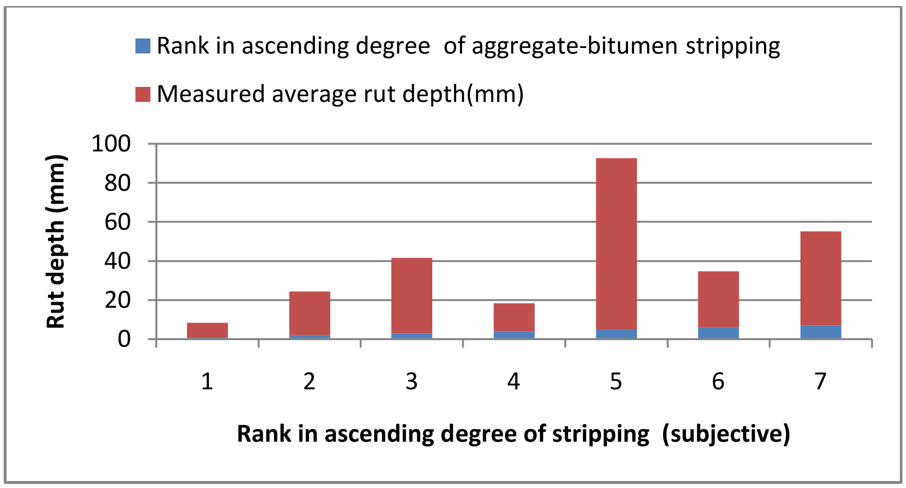

Figure 4-23 Rank in ascending degree of stripping (subjective) Vs Rut depth

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

The results of this study show that rutting is indeed a very complex phenomenon, which is affected by the various aggregate and mixture properties of hot-mix-asphalt. The test results indicate that:-

1. The 0.45 Power Gradation Plots give an indication, on the effect of aggregate gradations on the rutting performance of HMA mixtures. On the overall, HMA mixtures, which have aggregate gradations, which:-

    i. over the sieve size ranges finer than the 4.75mm, plot predominantly on the coarser side of the maximum density line;

    ii. over the sieve size ranges between the 4.75 -9.5mm sieve sizes plot almost coincidental with the maximum density line;

    iii. over the sieve size ranges exceeding 9.5mm plot on the finer side of the maximum density line;

    iv. and, where the sum of absolute distance from the gradation plots to the respective maximum density lines, is acceptably but not extremely higher (preferably not exceeding 49.9),

    generally performed better with regards to rutting.

2. Coarse aggregate angularity had an effect on the rutting performance of mixtures, where the rutting performance was generally improved with increase in coarse aggregate angularity. Moreover, increase in coarse aggregate angularity also increased the in-place VMA of HMA mixtures.

3. Even though the effect of in-place air voids (Va) on rutting was not as pronounced as theoretically expected, mixes with increased in-place air voids somehow performed better than mixes with lower in-place air voids.

4. The rutting performance of HMA mixtures, with lower in-place voids filled with asphalt (VFA) was generally better, especially those mixes which had in-place VFA's within the range 64-69%.

5. Despite the theoretical expectation that mixtures with reasonably higher in-place VMA values possess higher in-place Va's and hence better rutting resistance; in-place voids in mineral aggregate (VMA), according to this research, had almost no effect on the rutting performance of HMA mixtures. However, the in-place VMA of mixtures increased with

increase in the distance of aggregate gradation plots from the respective maximum density lines; which is consistent with the theoretical expectations.

6. Marshall Stability of in-place mixtures had no significant effect on the performance of HMA mixtures.

7. The Marshall flow value of in-place mixtures had a significant effect on the rutting performance. Mixtures with lower in-place flow values performed better with regards to rutting. However understandably, the flow values of all the in-place HMA mixtures were below the recommended design value of 8 — inch.

8. The Marshall quotient of in-place HMA cores had a notable effect on the rutting performance, where the observed rut depths decreased with increase in the in-place Marshall quotient .

9. Re-compacted air void (Va) content of mixtures (which is plausibly considered to be an estimate of the mix design air void content) significantly affected the rutting performance of HMA mixtures, where mixes with higher re-compacted Va content performed better in that respect. Excessive rut depths(>40mm) were observed for mixes which had re-compacted air voids less than 3.3%.

10. The distance of aggregate gradation plots from the respective maximum density lines had a 'non-linear' effect on the re-compacted Va of mixtures where, the re-compacted Va of mixtures increased with increase in the distance of aggregate gradation from the MDL upto around the 49.9 value and then dramatically reduced beyond that; which is consistent with the effect of distance from MDL of aggregate gradations on rutting.

11. Despite the theoretical expectation that mixtures with reasonably higher re-compacted VMA values possess higher re-compacted Va's and hence better rutting resistance; re-compacted voids in mineral aggregates (VMA), according to this research, had no direct effect on rutting performance of mixtures. However, it was generally observed that re-compacted Va of mixtues somehow increased with an increase in re-compacted VMA.

12. Mixes with lower re-compacted voids filled with asphalt (VFA) performed better with regards to rutting. The likelihood of experiencing excessive rut depths increased when the re-compacted VFA's were above 72.1%.

13. Re-compacted stability and re-compacted Marshall Quotient had almost no effect on the rutting performance of HMA mixtures.

14. Mixes with higher re-compacted flow values experienced higher rut depths.

15. Even though it was difficult to directly attribute rutting to the mere effect of aggregate-bitumen stripping phenomenon, it was observed that the increased degree of stripping coupled with coincidental shoving and reduced aggregate angularity, appreciably reduced the rutting performance of mixtures.

16. The mere 75 blow Marshall compaction approach to simulate traffic loadings in excess of one million ESA's, is not sufficient anymore to represent the current levels of high traffic loadings on highly trafficked trunk roads; as currently, the one million ESA's margin is reached within the first year (or two) of opening to traffic of such roads. The effect of such underestimated loading consideration is expected to be more pronounced in high pavement temperature environments.

17. The legal load limit of 10ton axles, beyond which pavement overloading is entailed, is being exceeded, and this has a detrimental effect on the rutting performance of pavements. Of course, the excessive axle loads can be taken into account during pavement design, to determine a structurally sufficient layer thickness, and during the HMA mix design, while selecting the appropriate Marshall compactive effort. However, by allowing higher axle loads (than the legal load limit), the HMA layer's life is expected to be substantially reduced with regards to rutting, especially taking into consideration the geometric nature of the subject road, the subsequently severe loading condition, and high pavement temperatures (exceeding 60^0C at times).

5.2. Recommendations

Hot-Mix-Asphalt mixtures can be designed and constructed to sustain the current levels of high traffic loading, by giving due attention to the various aggregate and mixture properties. Until the most advanced state-of-the-art mix design and testing procedures are familiarized with, Marshall Mix Design and Testing Method, coupled with the refusal-density mix design approach must be adopted to yield HMA mixtures, with aggregate gradations having sufficient voids in mineral aggregates, to ensure that the air void levels will not drop below 3-3.3%, after several years of traffic loading, on highly trafficked roads.

Available guidance customarily used for selecting initial gradations are typically vague, such as " choose a gradation somewhere near the center of the specification gradation band." However, making use of the 0.45 Power gradation chart and the maximum density line, to ensure that mixtures have sufficient voids in mineral aggregate and hence sufficient air void levels, aggregates with gradations which :-

- over the sieve size ranges finer than the 4.75mm, plot predominantly on the coarser side of the maximum density line;
- over the sieve size ranges between the 4.75 -9.5mm sieve sizes plot almost coincidental with the maximum density line;
- over the sieve size ranges exceeding 9.5mm plot on the finer side of the maximum density line; and,
- where the sum of absolute distance from the gradation plots to the respective maximum density lines, is acceptably but not extremely higher (preferably not exceeding 49.9),

must be used, of course in addition to using angular and rough textured aggregates.

Regarding the Marshall and volumetric properties, the mix design VFA, Flow value and Marshall Quotient of HMA mixtures, give a significant indication on the rutting performance of mixtures and must be closely evaluated. The voids filled with asphalt (VFA), beyond being an intrinsic criteria to limit the maximum levels of VMA and maximum levels of asphalt content during mix design, must be limited, preferably to the range 65-72.1%, to obtain better performing mixes with regards to rutting.

However, while emphasizing the need for sufficient voids to improve the rutting performance of HMA mixtures, the counter-effects of having relatively lower asphalt content, to increase the air voids, must be clearly understood, in that, such lowering of the asphalt content may lead to brittleness, accelerated oxidation and increased permeability. Moreover, high void contents are also frequently associated with mixes with high permeability, which by allowing the circulation of water and air may lead to premature hardening of the asphalt, raveling of aggregates or the possibility of stripping of the asphalt off the aggregates. and Hence, researches should be conducted to assess the conflicting void requirements of rutting on the one hand and durability, brittleness as well as increased permeability on the other hand.

On the **o**ther hand, taking into consideration the complex nature of the rutting phenomena, the need to resort to the more advanced Superpave mix design and performance testing method, for the design and evaluation of the rutting potential of HMA mixtures, is unquestionable; and it is high time our local pavement design manuals incorporate the same. But until then, substantial research efforts should be made on developing rut resistant mixtures, by comprehensively investigating the effect and suitability of different binders, along with the various material and mixture properties of hot-mix-asphalt mixtures, for the varying geometric, traffic and weather conditions in our country.

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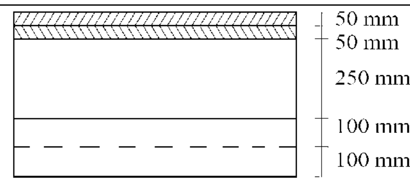
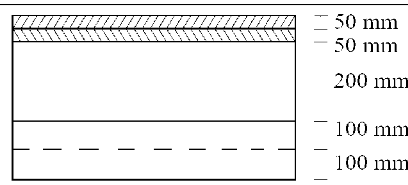
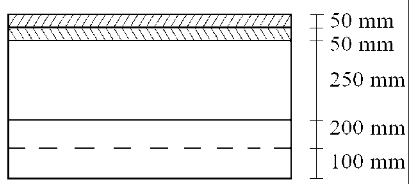
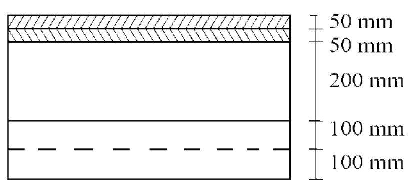
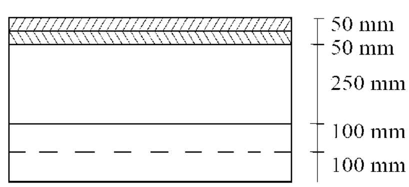
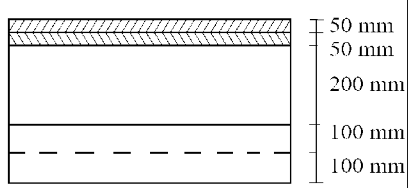
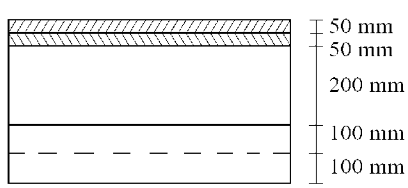
APPENDIX

APPENDIX A: Details of Sampling and Field Measurement Locations

Details of Sampling and Field Meaurement Locations

I.no	Sampling		Sampling location ID	Location of most severe rut-line from adjacent edge (m)	Rut depth measurment along rut-line (mm)		Core sample ID	Sampling time	Road surface temperature at the time of sampling (^{0}C)	Observed coincidental distresses	Remark
	Station	Approximate coordinate			measurement	Avg.					
1	3+200		1	2.8	23.0	14.3	1L1	4:40PM	45.3		cold and cloudy weather during sampling
		X=0416940		2.8	18.0		1L2				
				2.7	15.0		1C				
		Y=1108748		2.7	12.0		1R1				
				2.6	12.0		1R2				
2	6+680		2	1.0	25.0	28.8	2L1	5:05PM	45.7	corrugation transverse and longitudinal shoving	
		X=0416698		0.9	30.0		2L2				
				0.9	27.0		2C				
		Y=1110199		0.9	33.0		2R1				
				0.9	41.0		2R2				
3	13+280		3	1.3	51.0	48.2	3L1	6:20PM	42.0	very severe rutting with visible upheaval	
		X=0416177		1.4	49.0		3L2				
				1.4	49.0		3C				
		Y=1112250		1.4	46.0		3R1				
				1.4	46.0		3R2				
4	16+800		4	1.1	36.0	38.6	4L1	8:50AM	41.4	severe rutting visible bleeding on the upheaval zone	
		X=0414412		1.0	36.0		4L2				
				1.0	40.0		4C				
		Y=1111834		1.0	41.0		4R1				
				0.9	40.0		4R2				
5	21+940		5	1.2	24.0	22.4	5L1	9:50AM	52.5	bitumen reach surface minor bleeding	
		X=0410841		1.2	21.0		5L2				
				1.3	17.0		5C				
		Y=1114830		1.3	25.0		5R1				
				1.3	25.0		5R2				
6	28+700		6	1.2	4.0	7.4	6L1	11:12AM	58.8		
		X=0410849		1.2	10.0		6L2				
				1.1	8.0		6C				
		Y=1114845		1.2	7.0		6R1				
				1.2	8.0		6R2				
7	26+800		7	1.2	90.0	87.6	7L1	12:16PM	62.6	extremely severe rutting adjacent transversal shoving high upheaval visible bleeding	
		X=0410829		1.2	80.0		7L2				
				1.1	89.0		7C				
		Y=1114845		1.2	93.0		7R1				
				1.1	86.0		7R2				

APPENDIX B: Subgrade CBR and Pavement Section Details of Sampling Sections

Sampling section	Station	Subgrade CBR_d(%)	Pavement section	Description
1	3+200	5-6		Asphalt surface course (50mm) Asphalt binder course (50mm) Base course (250mm) Subbase course (100mm) Exsisting subbase course (100mm)
2	6+680	7-10		Asphalt surface course (50mm) Asphalt binder course (50mm) Base course (200mm) Subbase course (100mm) Exsisting subbase course (100mm)
3	13+280	3-4		Asphalt surface course (50mm) Asphalt binder course (50mm) Base course (250mm) Subbase course (200mm) Exsisting subbase course (100mm)
4	16+800	7-10		Asphalt surface course (50mm) Asphalt binder course (50mm) Base course (200mm) Subbase course (100mm) Exsisting subbase course (100mm)
5	21+940	5-6		Asphalt surface course (50mm) Asphalt binder course (50mm) Base course (250mm) Subbase course (100mm) Exsisting subbase course (100mm)
6	28+700	5-6		Asphalt surface course (50mm) Asphalt binder course (50mm) Base course (200mm) Subbase course (100mm) Exsisting subbase course (100mm)
7	26+800	5-6		Asphalt surface course (50mm) Asphalt binder course (50mm) Base course (200mm) Subbase course (100mm) Exsisting subbase course (100mm)

APPENDIX C: Specific Gravity and Absorption of Coarse Aggregates (AASHTO T-85)

SPECIFIC GRAVITY AND ABSORPTION OF COARSE AGGREGATE **(AASHTO T-85**)

sampling location	sample ID	Avg bulk specific gravity(oven-dry)	Avg bulk specific gravity (SSD)	Avg apparent specific gravity	Water Absorption (%)
1	1L1	2.784	2.819	2.884	1.2
2	2L2	2.835	2.859	2.904	0.8
3	3L1	2.822	2.846	2.892	0.9
4	4L1	2.794	2.810	2.841	0.6
5	5L1	2.756	2.775	2.807	0.7
6	6R1	2.770	2.790	2.828	0.7
7	7L1	2.757	2.772	2.798	0.5

APPENDIX D: Specific Gravity And Absorption Of Fine Aggregates (AASHTO T-84)

SPECIFIC GRAVITY AND ABSORPTION OF FINE AGGREGATE **(AASHTO T-84)**

sampling location	sample ID	Avg bulk specific gravity(oven-dry)	Avg bulk specific gravity(SSD)	Avg apparent specific gravity	Water Absorption (%)
1	1L1	2.429	2.481	2.563	2.1
2	2L2	2.619	2.693	2.829	2.8
3	3L2	2.711	2.734	2.791	1.1
4	4L1	2.758	2.783	2.827	0.9
5	5L1	2.708	2.745	2.811	1.4
6	6L1	2.569	2.621	2.711	2.0
7	7L1	2.599	2.642	2.715	1.6

APPENDIX E: Specific Gravity Of Filler (AASHTO T-100)

SPECIFIC GRAVITY OF FILLER **(AASHTO T-100)**

sampling location	sample ID	Avg specific gravity
1	1L1	2.732
2	2L2	
3	3L2	
4	4L1	
5	5L1	
6	6L1	
7	7L1	

APPENDIX F: Combined Specific Gravity Of Aggregates

COMBINED SPECIFIC GRAVITY OF AGGREGATES

sampling location	sample ID	% coarse	% fine	% filler	bulk specific gravity of combined aggregates
1	1L1	53.5	41.4	5.1	2.623
2	2L2	63.2	31.1	5.7	2.758
3	3L1	44.8	44.3	10.8	2.762
4	4L1	53.6	29.5	16.9	2.773
5	5L1	51.7	39.3	9.0	2.735
6	6R1	55.5	42.7	1.8	2.680
7	7L1	55.6	36.8	7.6	2.695

APPENDIX G: Percent Fractured Particles In Coarse Aggregates (ASTM D 5821)

PERCENT FRACTURED PACTICLES IN COARSE AGGREGATES **(ASTM D 5821**)

sampling location	sample ID	% by weight of aggregates (19-9.5)mm	% of parcticles with 2 or more fractured faces	% by weight of aggregates (9.5 - 4.75)mm	% of parcticles with 2 or more fractured faces	Total % crushed particles
1	1L1	63.0%	83.3%	37.0%	91.4%	86.3%
2	2L2	61.3%	89.9%	38.7%	94.4%	91.6%
3	3L2	55.9%	81.9%	44.1%	93.3%	86.9%
4	4L1	64.7%	90.4%	35.3%	89.0%	89.9%
5	5L1	57.6%	83.7%	42.4%	88.7%	85.8%
6	6L1	66.0%	79.0%	44.0%	82.1%	88.3%
7	7L1	64.6%	79.9%	35.4%	81.7%	80.5%

APPENDIX H: Coating And Stripping Of Bitumen Aggregate Mixtures (AASHTO T-182-84)

COATING AND STRIPPING OF BITUMEN AGGREGATE MIXTURES **(AASHTO T-182-84)**

sampling location	sample ID	Coated Area (%)	Subjective Ranking (in ascending rate of stripping)	Remark
1	1L1	< 95%	4	
2	2L2	< 95%	6	
3	3L2	< 95%	7	
4	4L1	< 95%	3	
5	5L1	< 95%	2	
6	6L1	< 95%	1	
7	7L1	< 95%	5	

APPENDIX I: Maximum Theoretical Specific Gravity Of Bituminous Mixtures (AASHTO T-209)

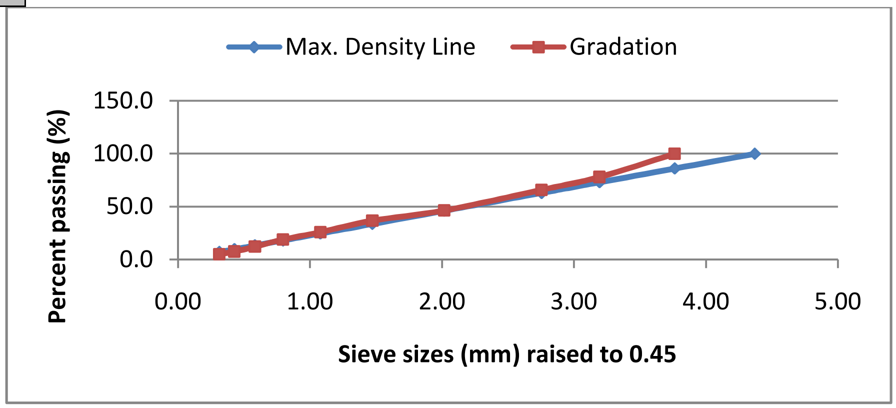
MAXIMUM THEORETICAL SPECIFIC GRAVITY OF BITUMINOUS MIXTURES **(AASHTO T-209)**

sampling location	sample ID	Average maximum specific gravity
1	1R2	2.608
2	2C	2.630
3	3R1	2.616
4	4L2	2.639
5	5R2	2.646
6	6C	2.661
7	7R1	2.645

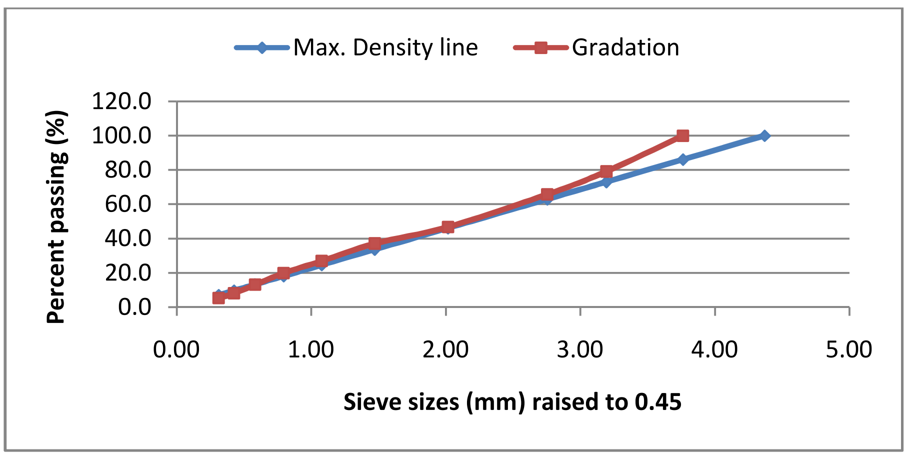
APPENDIX J: Aggregate Gradation Plots (0.45 Power Gradation Charts)

AGGREGATE GRADATIONS OF SAMPLING SECTION - 1

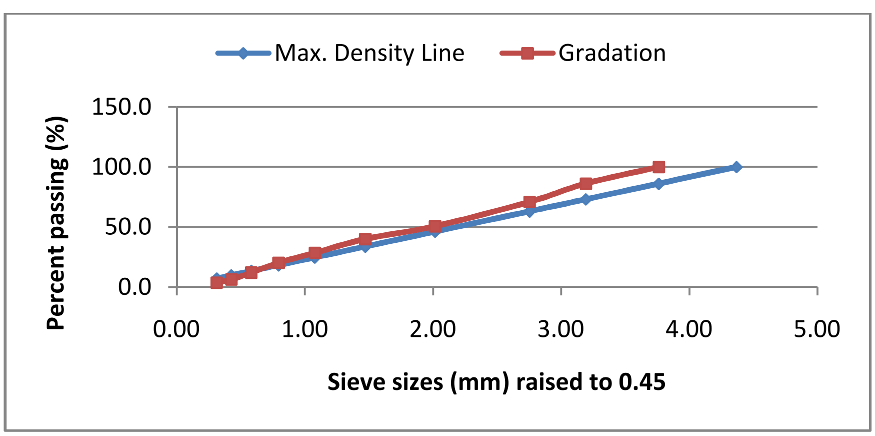
Site	1L1



Site	1C

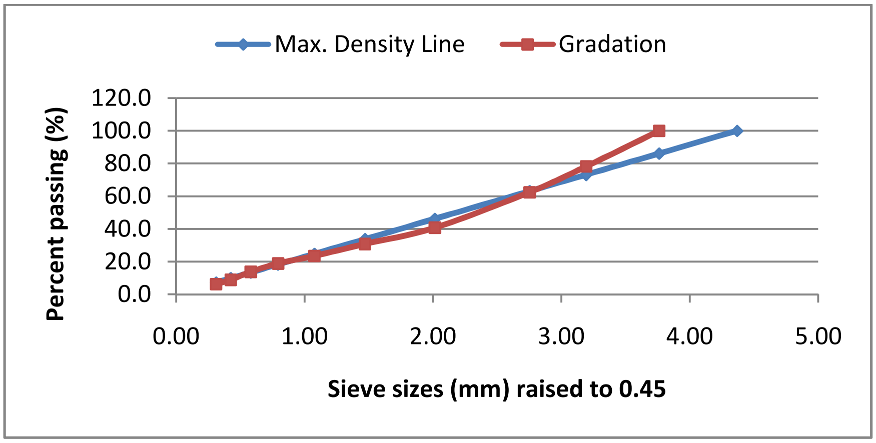


Site	1R1

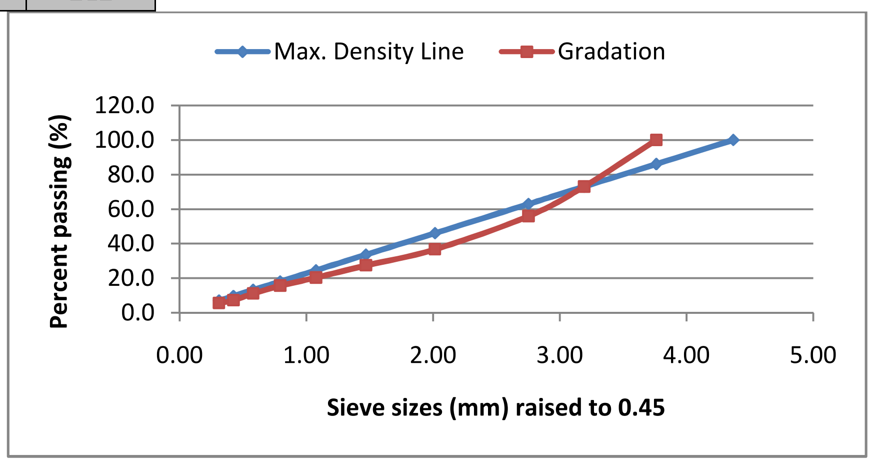


AGGREGATE GRADATIONS OF SAMPLING SECTION - 2

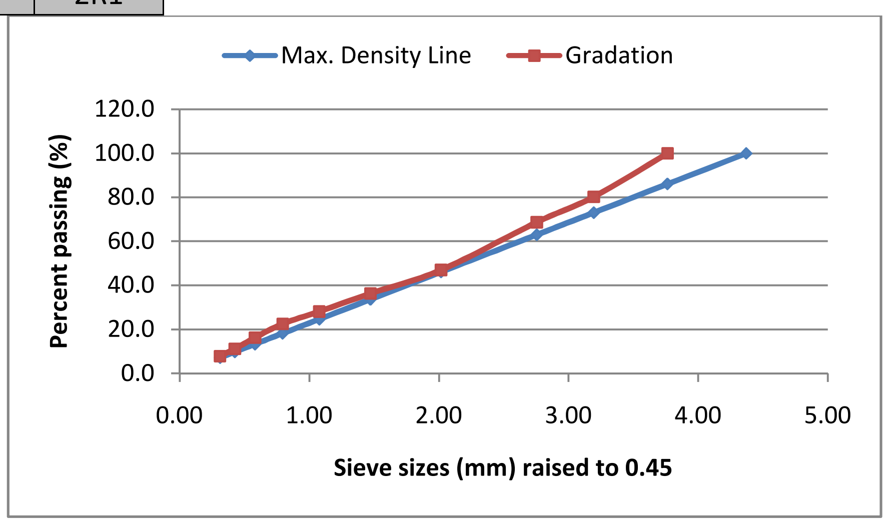
Site	2L1



Site	2L2

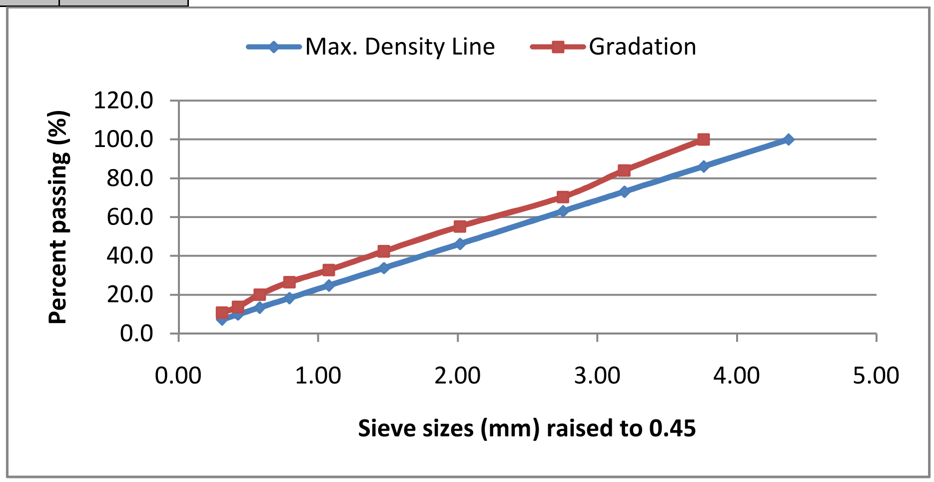


Site	2R1

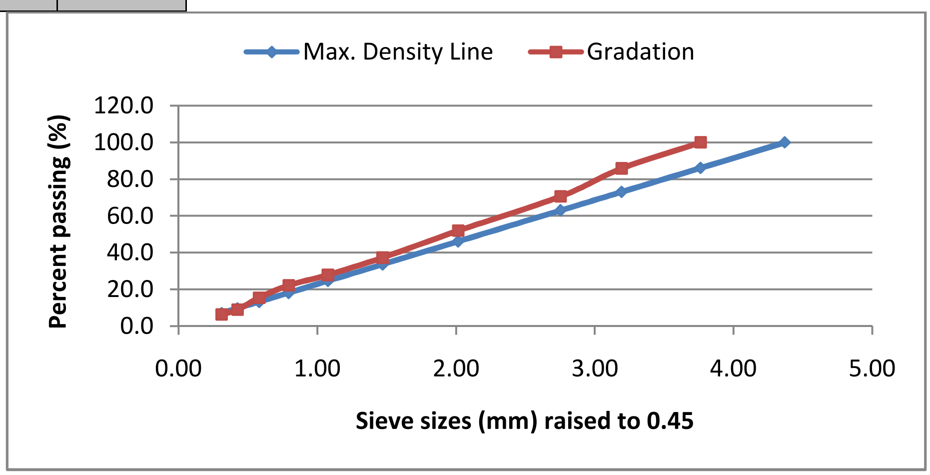


AGGREGATE GRADATIONS OF SAMPLING SECTION - 3

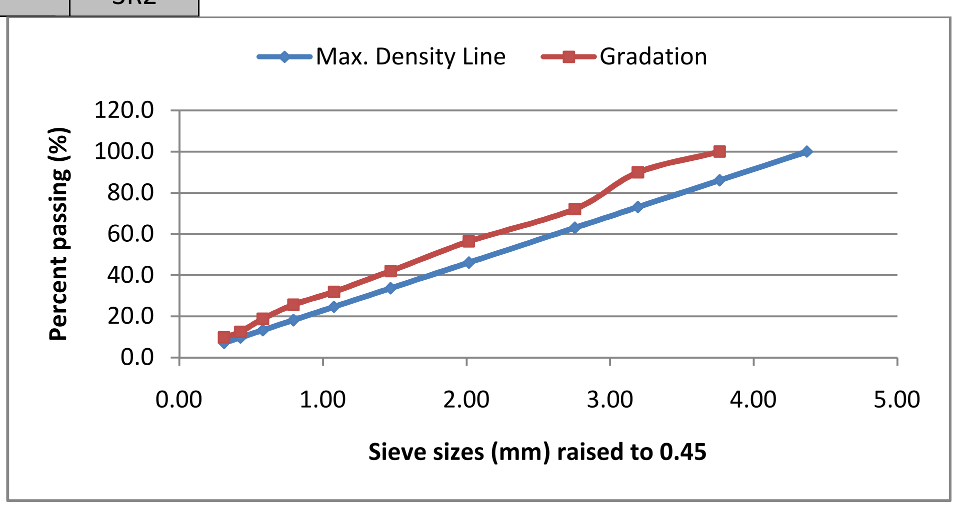
Site	3L1



Site	3L2

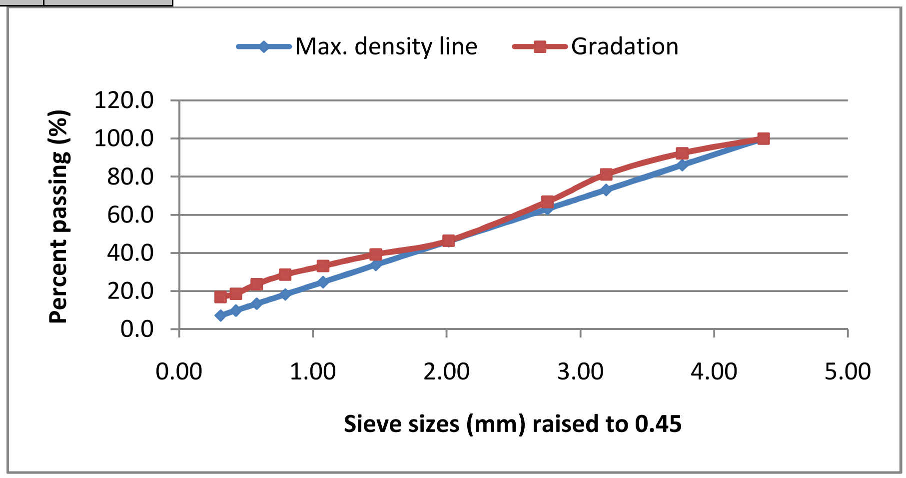


Site	3R2

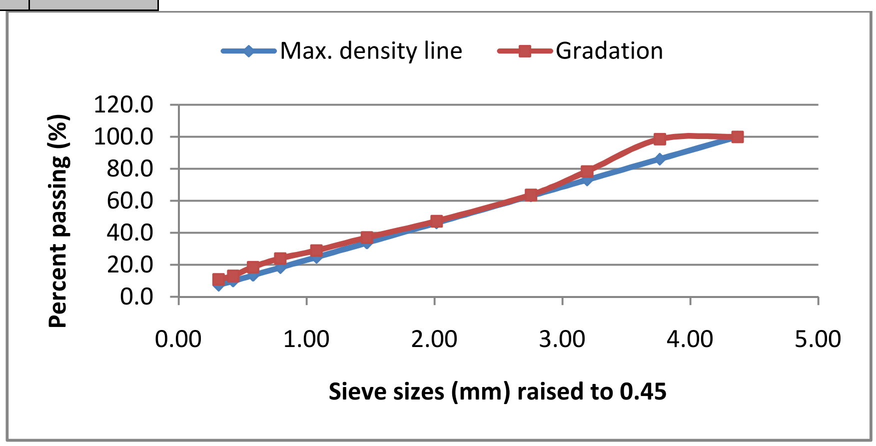


AGGREGATE GRADATIONS OF SAMPLING SECTION - 4

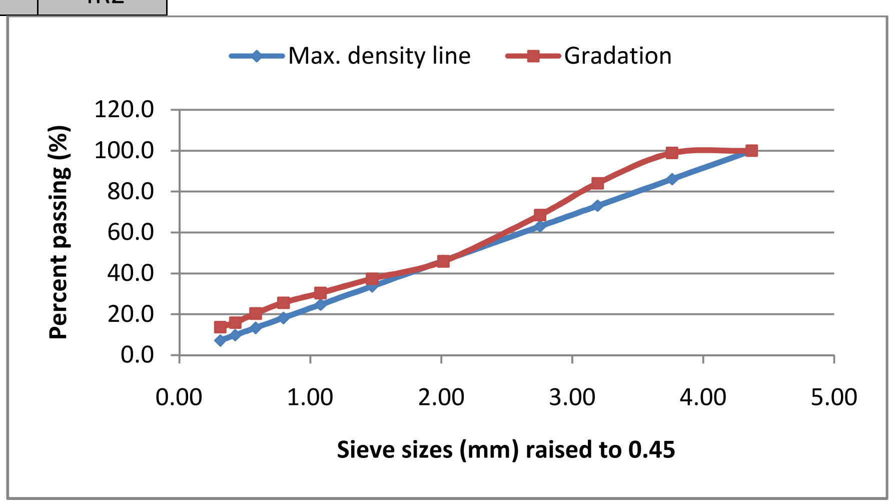
Site	4L1



Site	4R1

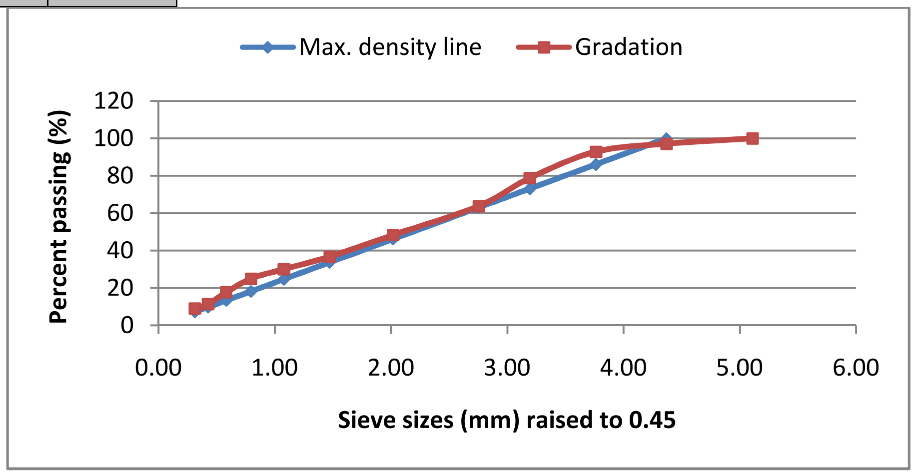


Site	4R2

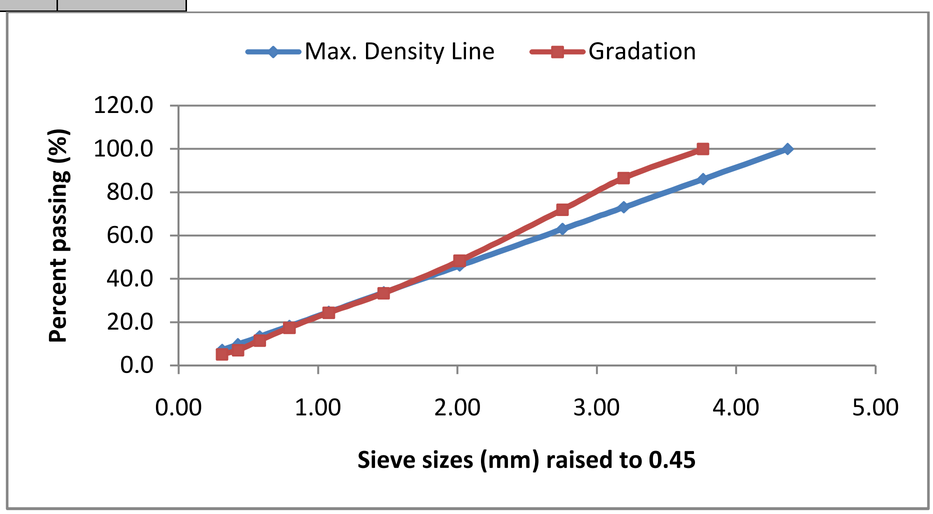


AGGREGATE GRADATIONS OF SAMPLING SECTION - 5

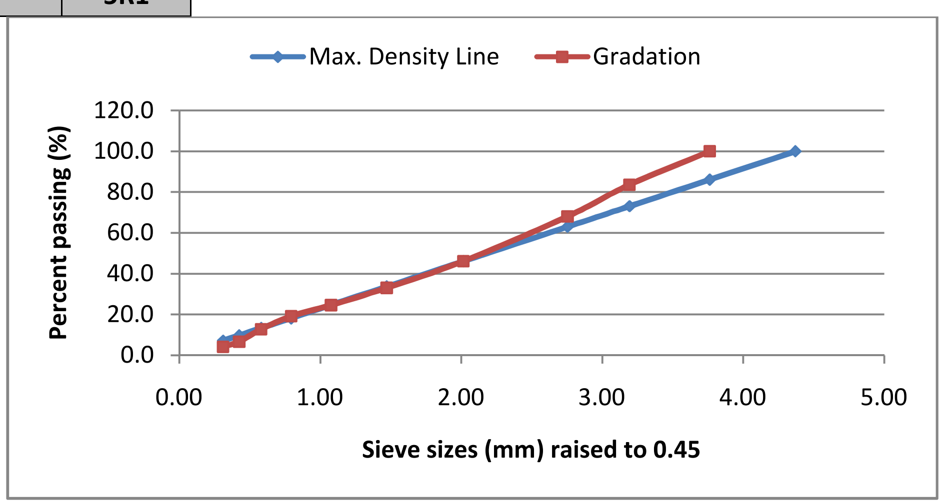
Site	5L1



Site	5C

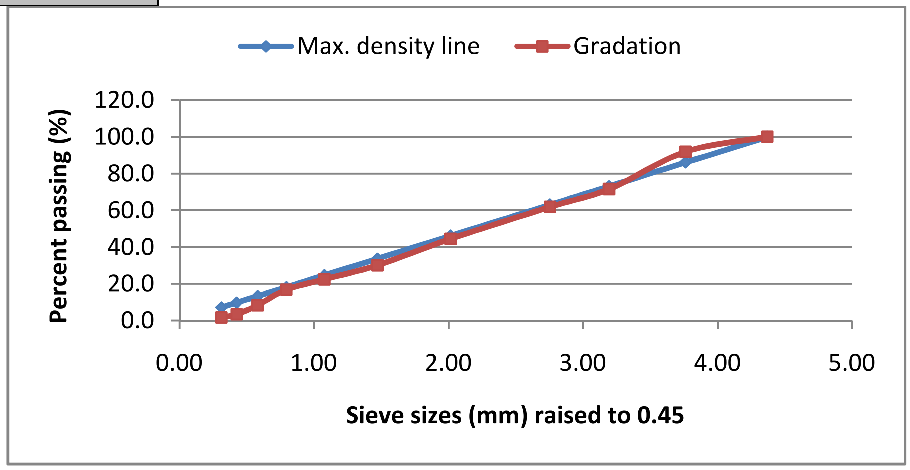


Site	**5R1**

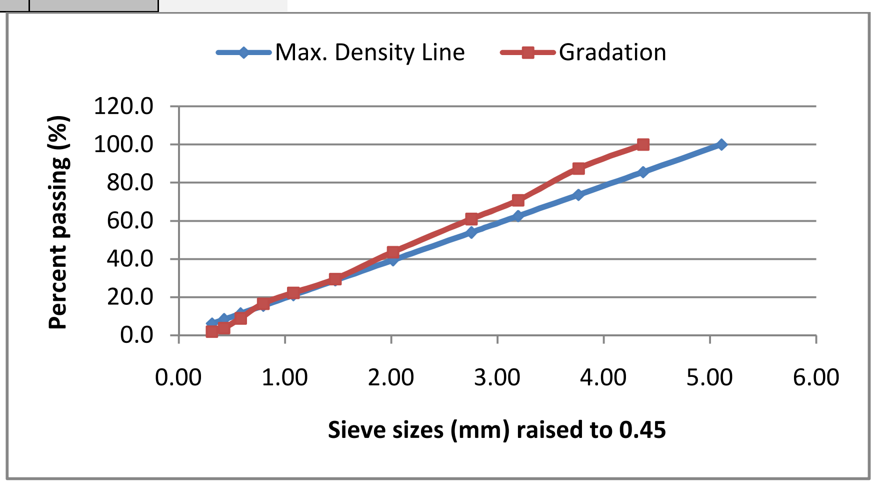


AGGREGATE GRADATIONS OF SAMPLING SECTION - 6

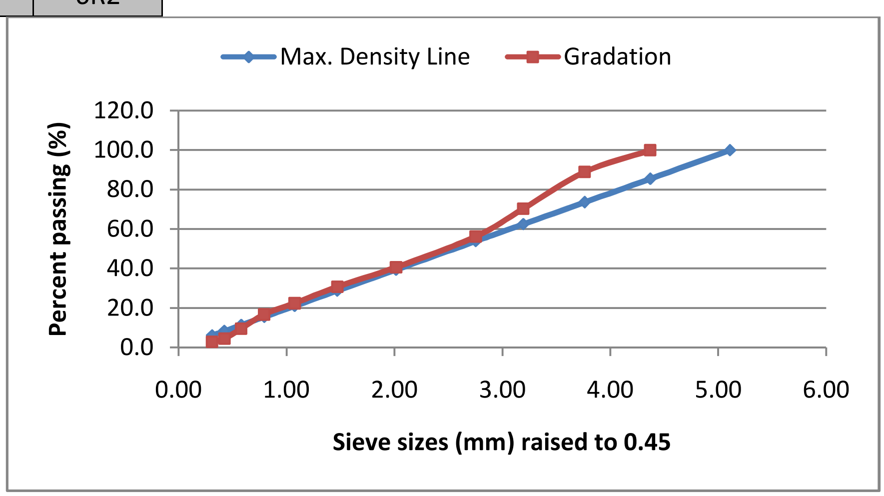
Site	6L1



Site	6R1

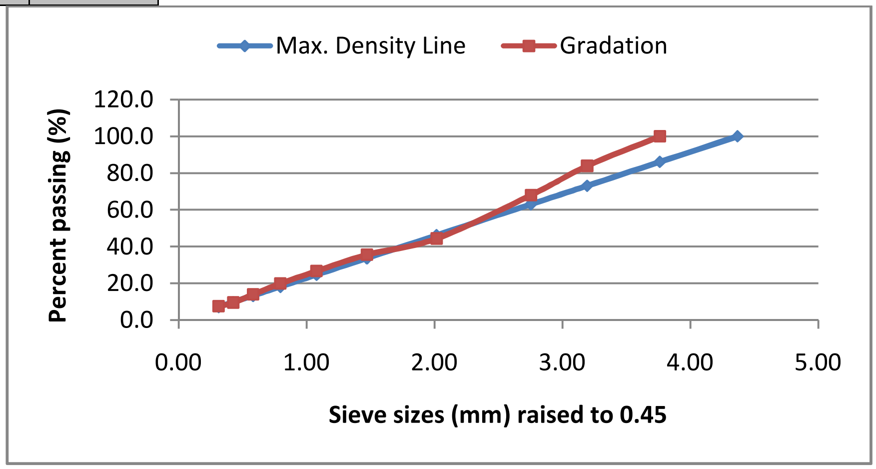


Site	6R2

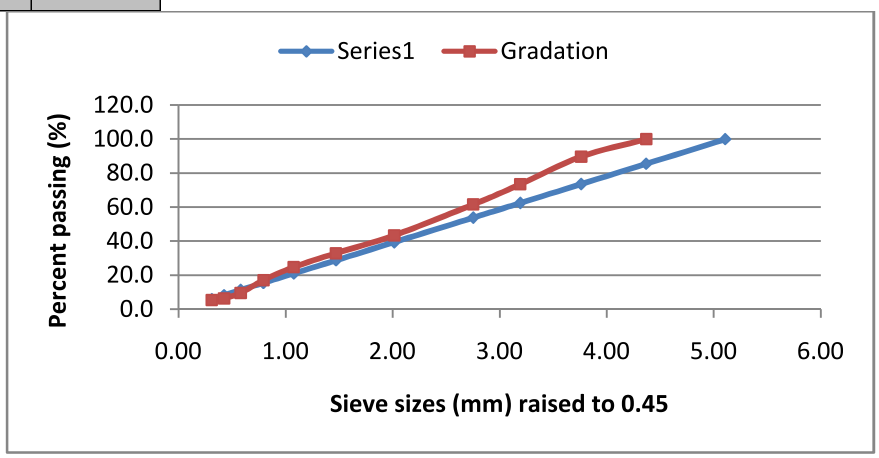


AGGREGATE GRADATIONS OF SAMPLING SECTION - 7

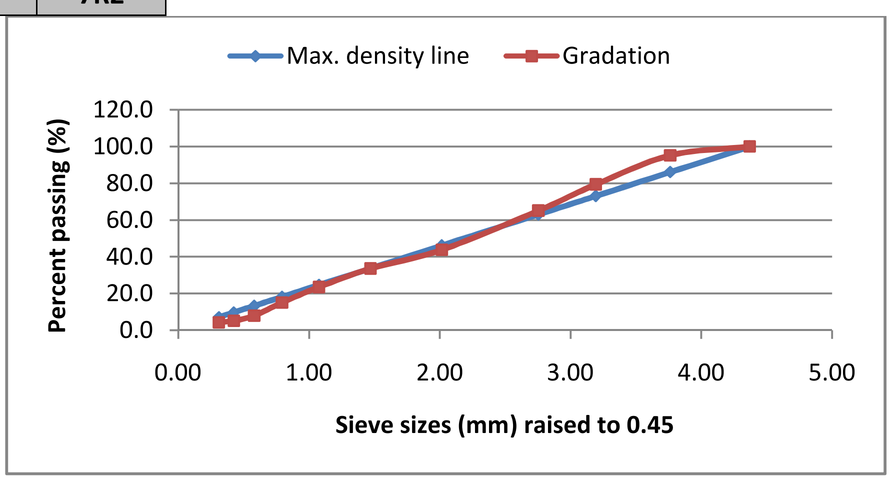
Site	7L1



Site	7L2



Site	7R2



APPENDIX K: Traffic Data

Traffic, Chancho-GohaTsion (source :'Traffic Survey and Safety Measures Report for Addis Ababa - Gohatsion Road Project, Detailed Engineering Design, Tender Document Preparation and Construction Supervision (2012)"

year	Small Cars	Land Rover/ Sta. Wagon/ Pick ups	Minibus (12 seats)	Small Buses 12- 25 seats	Mediu m Bus 25 - 45 seat	Large Bus >45 seat	Small Truck/ Isuzu	Medium Truck	Heavy Truck	Trucks & Trailer	Total
2012	53	255	377	16	20	130	181	150	133	149	**1464**
2013	56	276	404	17	21	139	198	164	146	163	**1,584**

APPENDIX L: Axle Load Data

Axle Load Survey : Source 'Traffic Survey and Safety Measures Report for Addis Ababa - Gohatsion Road Project, Detailed Engineering Design, Tender Document Preparation and Construction Supervision (2012) By Hitcon Engineering',

Time	Vehicle Type	Axle Config.	Direc. Of Travel	Freight Type	F1	F2	F3	F4	F5	F6
	D/T	1.22	DERBA	SOIL	49	29	25			
	TT	1.22+2.22	AA	SELITE	67	143	136	102	98	100
	MT	1.2	AA	CEMENT	70	123				
	ST	1.2	D.MARKOS	GOODS	29	63				
	TT	1.2-221	AA	ENPTY	54	43	54	50	20	
	MB	1.2	FITCHE	GOODS	22	38				
	LB	1.2	D.MARKOS	PASENGER	53	100				
	TT	1.22+2.22	AA	CEMENT	68	121	123	112	109	106
	LB	1.2	GONDER	PASENGER	50	97				
	ST	1.2	D.MARKOS	STEEL	29	56				
	LB	1.2	GONDER	PASENGER	43	89				
	MT	1.2	AA	RUSSE	38	84				
	MB	1.2	AA	PASENGER	40	72				
	DT	1.22	DERBA	SOIL	54	112	117			
	LB	1.2	G.GURACH	PASENGER	43	78				
	TT	1.22+2.22	FITCHE	CEMENT	82	122	120	113	104	98
	DT	1.22	DERBA	SOIL	39	109	110			
	LB	1.2	D.MARKOS	PASENGER	49	75				
	LB	1.2	D.MARKOS	PASENGER	65	91				
	DT	1.22	DERBA	SOIL	48	106	109			
	MT	1.2	AA	WOOD	40	97				
	ST	1.2	AA	SPICE	28	66				
	DT	1.22	DERBA	SOIL	45	103	106			
	TT	1.22-222	DERBA	POMICH	63	120	109	91	77	103
	MT	1.2	DERBA	GOODS	45	111				
	TT	1.22-222	DERBA	POMICH	68	104	97	115	109	110
	ST	1.2	G.T	GOODS	31	70				
	LB	1.2	FITCHE	PASENGER	41	65				
	TT	1.22+2.22	DERBA	POMICH	71	105	102	101	113	101
	DT	1.22	DERBA	SOIL	47	101	99			
	HT	1.21	DERBA	GOODS	70	98	26			
	ST	1.2	B.DARA	GOODS	32	65				
	MB	1.2	G.GURACH		42	62				
	LB	1.2	D.MARKOS		44	88				
	LB	1.2	GONDER	PASENGER	50	106				
	ST	1.2	FITCHE	GOODS	26	64				
	TT	1.22+2.22	AA	CEMENT	74	122	118	104	113	101
	ST	1.2	AA	ENPTY	28	52				
	TT	1.22+2.22	DERBA	POMICH	75	104	106	100	98	94
	FT	1.22+2.22	AA	CEMENT	74	136.0	132.0	96	128	119
	DT	1.22	DERBA	SOIL	56	99	98			
	ST	1.2	AA	SPICE	30	61				
	MT	1.2	AA	RUASE	44	126				
	SB	1.2	AA	PASSENGER	48	71				
	TT	1.22-222	AA	CEMENT	83	117	110	106	118	120
	TT	1.22-222	AA	CEMENT	78	115	109	114	122	123
	LB	1.2	D.MARKOS	PASSENGER	52	87				
	DT	1.22	DERBA	SAND	62	118	120			
	LB	1.2	D.MARKOS	PASSENGER	49	84				

	LB	1.2	D.MARKOS	PASSENGER	65	119				
	MB	1.2	G.GURACHA	PASSENGER	46	68				
	TT	1.22+2.22	DERBA	POMICHE	75	127	130	94	101	102
	MT	1.2	GONDER	GOODS	59	118				
	ST	1.2	AA	SPICE	26	61				
	SB	1.2	FITCHE	PASSENGER	36	64				
	ST	1.2	AA	SELITE	30	72				
	SB	1.3	AA	PASSENGER	24	40				
	SB	1.2	AA	PASSENGER	21	40				
	MT	1.2	AA	LEWZE	34	53				
	TT	1.22+2.22	DERBA	POMICHE	60	111	105	88	83	
	ST	1.2	AA	RUSSE	27	57				
	ST	1.2	AA	RUSSE	24	61				
	LB	1.2	GONDER	PASSENGER	54	97				
	FT	1.22+2.22	FITCHE	FUEL	75	116	111	98	102	109
	SB	1.2	AA	PASSENGER	41	72				
	ST	1.2	AA	SELITE	30	69				
	LB	1.2	GONDER	PASSENGER	55	95				
	ST	1.2	GONDER	SOLT	39	107				
	LB	1.2	D.MARKOS	PASSENGER	57	80				
	ST	1.2	GONDER	SOLT	31	63				
	ST	1.2	GONDER	SOLT	28	61				
	ST	1.2	AA	SPICE	30	69				
	ST	1.2	FITCHE	GOODS	27	56				
	ST	1.2	AA	SELITE	27	61				
	FT	1.22+2.22	AA	FUEL	84	129	124	99	118	121
	LB	1.2	GOJAM	PASSENGER	46	83				
	ST	1.2	AA	RUASE	27	63				
	DT	1.22	CHANCHO	SAND	66	122	129			
	ST	1.2	M.TURI	GOODS	23	37				
	DT	1.22	DERBA	SAND	51	124	124			
	MT	1.2	AA	WOOD	37	90				
	ST	1.2	AA	SOLITE	29	64				
	SB	1.2	FITCHE	PASSENGER	34	72				
	ST	1.2	AA	SELITE	28	57				
	ST	1.2	AA	SELITE	23	44				
	TT	1.22+2.22	AA	SELITE	80	130	125	106	110	101
	MT	1.2	AA	WOOD	50	120				
	TT	1.22+2.22	AA	SELITE	72	125	128	101	104	111
	TT	1.22+2.22	AA	SELITE	75	130	132	108	90	110
	ST	1.2	D.MARKOS	GOODS	28	66				
	MT	1.2	AA	TELBA	48	115				
	TT	1.22+2.22	AA	SELITE	81	130	127	102	110	113
	ST	1.2	FITCHE	MAZE	31	65				
	ST	1.2	FITCHE	GOODS	28	50				
	ST	1.2	G.GURACHA	STEEL	32	62				
	ST	1.2	AA	SPICE	28	58				
	ST	1.2	AA	SPICE	31	60				
	ST	1.2	AA	TELBA	29	62				
	ST	1.2	AA	TELBA	30	63				
	MT	1.2	AA	TELBA	30	68				
	MT	1.2	D.MARKOS	STEEL	44	101				
	ST	1.2	FITCHE	MAZE	27	61				
	ST	1.2	FITCHE	MAZE	27	61				
	ST	1.2	FITCHE	ONION	26	44				
	MT	1.2	AA	SELITE	45	111				
	ST	1.2	AA	SELITE	30	59				
	TT	1.22+2.22	AA	CEMENT	84	130	127	119	115	99
	ST	1.2	GONDER	SPICE	28	60				
	MT	1.2	GONDER	SPICE	42	103				

	ST	1.2	AA	TELBA	30	53				
	LB	1.2	D.MARKOS	PASSENGER	46	82				
	TT	1.22+2.22	AA	SELETE	77	132	130	100	108	107
	DT	1.22	AA	SAND	62	140	139			
	DT	1.22	AA	SAND	65	129	131			
	MT	1.2	AA	SSELETE	49	102				
	ST	1.2	AA	GOODS	21	24				
	TT	1.22+2.22	AA	SELETE	82	135	133	106	113	114
	ST	1.2	AA	EMPTY	19	16				
	ST	1.2	AA	EMPTY	19	15				
	DT	1.22	FITCHE	SAND	78	110	138			
	TT	1.2-221	AA	EMPTY	54	53	46	35	41	
	ST	1.2	AA	SELETE	20	37				
	MT	1.2	FITCHE	EMPTY	27	29				
	LB	1.2	AA	PASSENGER	49	99				
	SB	1.2	AA	PASSENGER	21	36				
	MT	1.2	AA	CEMENT	29	38				
	MT	1.2	AA	EMPTY	27	30				
	ST	1.2	CHANCHO	SAND	29	65				
	LB	1.2	AA	PASSENGER	52	104				
	MT	1.2	AA	GOODS	42	96				
	ST	1.2	AA	EMPTY	16	30				
	LB	1.2	D.MARKOS	PASSENGER	36	93				
	MT	1.2	AA	WOOD	36	97				
	TT	1.22-222	AA	EMPTY	60	48	46	39	39	47
	TT	1-22-222	AA	EMPTY	62	47	45	41	41	46
	ST	1.2	AA	TEFE	29	59				
	TT	1.22-222	DERBA	POMICHE	67	91	90	102	118	
	ST	1.2	CHANCHO	SAND	29	69				
	TT	1.22+2.22	AA	SELETE	72	147	137	109	113	112
	DT	1.22	G.T	EMPTY	51	52	51			
	ST	1.2	CHANCHO	FAGULOO	26	58				
	TT	1.22-222	G.T	SOIL	60	98	96	136	133	140
	TT	1.22-222	G.T	GOODS	47	71	71	83	62	71
	HT	1.2	GONDER	GOODS	77	134				
	TT	1.22+2.22	AA	CEMENT	79	1128	130	122	120	123
	ST	1.2	FITCHE	GOODS	44	112				
	FT	1.2+2.2	FITCHE	FUEL	95	139	96	95		
	TT	1.22-222	G.GURACHA	SOIL	64	117	103	106	131	133
	MT	1.2	AA	GOODS	33	58				
	TT	1.22+2.22	AA	EMPTY	53	38	35	22	31	31
	HT	1.2	CHANCHO	STONE	51	145				
	TT	1.2-221	AA	EMPTY	51	50	44	43	42	
	TT	1.22+2.22	BAH.D	FLOWER	78	135	131	100	115	112
	TT	1.22-222	BAH.D	CONTAINER	70	117	115	98	99	122
	HT	1.2	AA	WOOD	51	138				
	TT	1.22+2.2	CHANCHO	EMPTY	52	35	34	25	25	
	TT	1.22+2.22	D.MARKOS	FLOWER	70	125	123	106	118	110
	HT	1.22	D.MARKOS	METAL	78	134	132			
	TT	1.22-222	AA	CEMENT	72	114	110	115	112	129
	LB	1.2	FITCHE	PASSENGER	50	113				
	LB	1.2	D.MARKOS	PASSENGER	68	100				
	DT	1.22	CHANCHO	SOIL	64	100	106			
	LB	1.2	D.MARKOS	PASSENGER	51	101				
	LB	1.2	D.MARKOS	PASSENGER	57	98				
	ST	1.2	CHANCHO	GOODS	38	80				
	LB	1.2	D.MARKOS	PASSENGER	49	81				
	LB	1.2	D.MARKOS	PASSENGER	51	101				
	LB	1.2	D.MARKOS	PASSENGER	50	114				
	DT	1.22	AA	CEMENT	56	80	83	81		

	LB	1.2	D.MARKOS	PASSENGER	48	107				
	LB	1.2	D.MARKOS	PASSENGER	47	96				
	TT	1.22+2.22	AA	CEALT	85	128	125	100	119	113
	DT	1.22	AA	GAPSEM	69	111	112			
	TT	1.22+2.22	AA	CEALT	73	131	131	103	110	106
	DT	1.22	AA	SAND	60	137	136			
	DT	1.22	AA	CEMENT	79	122	129			
	LB	1.2	AA	PASSENGER	57	102				
	LB	1.2	AA	PASSENGER	68	99				
	MT	1.2	DERBA	GOODS	38	42				
	ST	1.2	AA	SPICE	30	68				
	SB	1.2	AA	PASSENGER	39	75				
	TT	1.22+2.22	AA	CELETE	81	126	132	98	110	106
	TT	1.22+2.22	AA	CELETE	74	124	129	97	115	108
	DT	1.22	AA	GEPSEM	86	119	113			
	TT	1.22+2.22	AA	CELETE	76	126	121	104	113	103
	ST	1.2	AA	BANANA	30	75				
	TT	1.22+2.22	AA	CLINKAN	77	133	130	102	112	108
	TT	1.22+2.22	G.GURACHA	CLINKAN	76	128	131	107	116	105
	TT	1.2+2.2	GONDER	GOODS	60	112	81	92		
	DT	1.22	COSTIK	AA	77	138	135			
	DT	1.22	AA	CEMENT	76	129	132			
	TT	1.22-222	G.GURACHA	CLINKAN	61	105	104	143	130	127
	MT	1.2	GONDER	GOODS	39	116				
	FT	1.22+2.22	D.MARKOS	FUEL	72	130	129	98	110	119
	TT	1.22-222	G.GURACHA	CLINKAN	81	104	103	138	128	130
	TT	1.22+2.22	AA	CELETE	80	140	135	109	113	112
	TT	1.22+2.22	AA	CELETE	72	135	130	107	115	100
	ST	1.2	AA	GOODS	26	52				
	DT	1.22	AA	GEPSEM	75	123	120			
	HT	1.2	AA	SOIL	50	126				
	LB	1.2	AA	PASSENGER	44	99				
	DT	1.22	CHANCHO	SOIL	58	97	94			
	TT	1.22+2.22	BURIE	SELETE	77	136	131	112	105	104
	TT	1.22+2.22	BURIE	SELETE	74	140	133	100	110	107
	ST	1.2	FITCHE	MAZE	25	56				
	ST	1.2	G.T	GOODS	22	47				
	TT	1.22+2.22	D.MARKOS	BEAN	82	132	130	111	115	113
	MT	1.2	AA	METAL	39	96				
	DT	1.22	AA	GEPSEM	65	140	143			
	TT	1.22-222	FITCHE	EMPTY	46	45	42	42	40	45
	ST	1.2	FITCHE	ONION	30	59				
	DT	1.2	CHANCHO	STONE	55	128				
	DT	1.2	CHANCHO	STONE	53	131				
	TT	1.22+2.22	FITCHE	EMPTY	56	45	40	39	38	38
	FT	1.2-22	SUDAN	EMPTY	42	82	50	58		
	MT	1.2	D.MARKOS	GOODS	48	130				
	TT	1.22-222	AA	CEMENT	80	128	128	120	110	120
	ST	1.2	MEREHABE	MAZE	27	49				
	ST	1.2	CHANCHO	GOODS	31	66				
	TT	1.22+2.22	AA	CEMENT	62	100	106	128	130	131
	MT	1.2	D.MARKOS	GOODS	43	121				
	MT	1.2	AA	WOOD	38	94				
	ST	1.2	AA	WOOD	28	68				
	TT	1.22-222	DERBA	AOMICH	66	102	100	130	129	131
	TT	1.22+2.2	AA	CEMENT	83	126	121	111	110	80
	TT	1.22+222	AA	SELETE	64	130	130	100	109	108
	DT	1.22	CHANCHO	SOIL	68	100	98			
	TT	1.22+2.22	AA	SELETE	72	133	130	105	113	107
	FT	1.22+2.22	AA	FUEL	78	138	135	112	108	110

	FT	1.22+2.22	AA	FUEL	77	137	134	109	110	110
	TT	1.22+2.22	AA	CEMENT	76	130	126	105	115	112
	FT	1.2+2.22	AA	FUEL	88	141	112	119		
	TT	1.22-222	FITCHE	EMPTY	53	41	37	30	42	36
	DT	1.22	CHANCHO	SOIL	56	98	103			
	ST	1.2	AA	TEFE	34	68				
	ST	1.2	CHANCHO	GOODS	29	56				
	DT	1.22	DEREBAN	EMPTY	48	57	44	43		
	ST	1.2	D.MARKOS	GOODS	32	45				
	ST	1.2	DERBAN	EMPTY	19	18				
	MT	1.2	G.GURACHA	TELBA	28	89				
	ST	1.2	G.GURACHA	GOODS	28	68				
	MT	1.2	AA	GEPSEM	61	132	135			
	MT	1.2	AA	CEMENT	30	78				
	TT	1.22+2.22	AA	GEPSEM	86	130	129	106	113	108
	TT	1.22+2.22	AA	SELETE	75	120	120	113	120	117
	TT	1.22-222	AA	GEPSEM	65	109	108	140	130	127
	TT	1.22+2.22	AA	SELETE	79	137	140	109	112	111
	HT	1.2	AA	SELETE	49	138				
	ST	1.2	AA	SELETE	26	60				
	HT	1.22	AA	BELOKE	78	140	138			
	TT	1.22+2.22	AA	SELETE	80	130	124	100	120	116
	TT	1.22-222	AA	GEPSEM	68	103	98	131	127	136
	MT	1.2	AA	RUASE	50	115				
	TT	1.22-222	AA	EMPTY	57	47	45	39	36	49
	FT	1.22+2.22	D.MARKOS	FUEL	75	126	122	107	130	121
	ST	1.2	AA	GOODS	30	62				
	FT	1.2+2.2	AA	EMPTY	48	46	31	33		
	MT	1.2	CHANCHO	SOFT DRINK	69	97				
	TT	1.22-222	AA	EMPTY	57	47	45	39	36	49
	FT	1.22+2.22	D.MARKOS	FUEL	75	126	122	107	130	121
	ST	1.2	AA	GOODS	30	62				
	FT	1.2+2.2	AA	EMPTY	48	46	31	33		
	MT	1.2	CHANCHO	SOFT DRINK	69	97				
	DT	1.22	DEREBAN	SOIL	65	105	109			
	TT	1.22+2.22	AA	SELETE	79	133	123	101	12	113
	TT	1.22-222	DEREBAN	CLINTEN	61	115	116	124	120	133
	TT	1.22-222	D.MARKOS	MACHINE	68	108	107	111	113	140
	TT	1.2+22	AA	EMPTY	57	46	40	50	52	
	HT	1.2	AA	WOOD	55	130				
	TT	1.2-22	AA	EMPTY	53	60	58	55		
	MT	1.2	AA	LETEHR	48	113				
	TT	1.22+1.22	AA	CEMENT	85	140	130	115	122	117
	ST	1.2	AA	TELBA	28	61				
	TT	1.2-221	AA	EMPTY	50	50	53	50	34	
	ST	1.2	AA	SPICE	28	64				
	MT	1.2	AA	LEWZE	50	115				
	TT	1.22-222	G.T	CONTAINER	59	65	60	80	89	113
	TT	1.22+2.22	AA	EMPTY	57	44	38	30	32	33
	TT	1.22-222	DERBAN	POMICHE	65	101	101	115	115	136
	ST	1.2	FITCHE	SAND	25	46				
	MT	1.2	MUKTARI	CEMENT	28	111				
	DT	1.22	DERBAN	SOIL	48	109	110			
	TT	1.22-222	AA	CEMENT	61	96	90	140	133	140
	TT	1.2+2.2	AA	CEMENT	59	145	97	98		
	DT	1.22	AA	CEMENT	73	126	127			
	DT	1.22	AA	CEMENT	80	124	127			
	TT	1.22-222	DERBAN	CLINKAN	67	104	105	140	133	140
	TT	1.22+2.22	AA	SELETE	84	130	129	101	113	1117
	DT	1.22	AA	CEMENT	75	120	122			

	TT	1.22+2.22	AA	SELETE	90	133	128	114	120	110
	DT	1.22	AA	GEPSEM	62	135	133			
	TT	1.22-222	DERBAN	POMICHE	65	108	105	122	120	132
	DT	1.22	DERBAN	SOIL	57	107	100			
	DT	1.22	DERBAN	POMICHE	83	116	119			
	TT	1.22+2.22	AA	E.BOTTEL	65	94	90	70	78	90
	TT	1.22-222	AA	DOZEN	79	116	112	157	139	133
	MT	1.2	GONDER	SPICE	45	115				
	TT	1.22-222	DERBAN	CLINKAN	68	99	98	130	120	138
	ST	1.2	FITCHE	SAND	28	62				
	DT	1.22	DERBAN	SOIL	56	103	101			
	MT	1.2	AA	RUASE	45	118				
	TT	1.22+2.22	AA	GEPSEM	90	127	126	107	115	119
	TT	1.22-222	AA	GEPSEM	64	102	107	135	139	132
	TT	1.22-222	AA	GEPSEM	64	111	109	121	130	140
	ST	1.2	AA	MEZE	28	61				
	ST	1.2	AA	TEFE	27	59				
	ST	1.2	FITCHE	SAND	28	60				
	TT	1.22+2.22	AA	EMPTY	57	44	40	33	29	27
	TT	1.22+2.22	AA	EMPTY	59	43	44	37	30	29
	ST	1.2	M.TURI	GOODS	29	68				
	TT	1.22-222	DERBAN	CLINKAN	69	109	108	130	121	140
	ST	1.2	AA	BELOKE	26	59				
	TT	1.22+2.22	AA	SELETE	74	140	137	111	123	110
	MT	1.2	AA	SELETE	48	111				
	TT	1.22-222	G.GURACHA	CLINKAN	66	109	105	121	130	140
	TT	1.22-222	G.GURACHA	CLINKAN	64	111	109	125	135	139
	TT	1.22-222	G.GURACHA	CLINKAN	66	105	109	120	131	142
	TT	1.22+2.22	AA	SELETE	83	130	133	119	127	130
	MT	1.2	AA	SPICE	48	116				
	TT	1.22+2.22	AA	SELETE	80	140	136	133	122	136
	TT	1.22+2.22	AA	SELETE	82	136	130	118	129	137
	TT	1.22-222	DERBAN	CLINKAN	65	108	107	125	120	132
	MT	1.2	D.MARKOS	COFFEE	42	109				
	DT	1.22	DERBAN	SOIL	65	99	109			
	TT	1.22+2.22	AA	BEER	70	128	126	107	121	112
	HT	1.2	AA	WOOD	53	133				
	TT	1.22+2.22	AA	SELETE	80	120	122	117	125	127
	ST	1.2	D.MARKOS	BANANA	30	63				
	ST	1.2	D.MARKOS	BANANA	29	70				
	ST	1.2	AA	CAW	25	44				
	FT	1.22+2.22	AA	FUEL	80	124	122	114	116	111
	FT	1.22+2.22	AA	FUEL	82	119	117	109	113	108
	TT	1.22-222	DERBAN	CLINKAN	69	110	111	132	130	140
	DT	1.22	AA	CEMENT	85	141	138			
	TT	1.22+2.22	AA	SELETE	76	133	133	111	120	114
	DT	1.22	AA	GEPSEM	59	141	145			
	TT	1.22-222	DERBAN	CLINKAN	68	110	108	120	129	131
	DT	1.22	G.GURACHA	EMPTY	58	50	46			
	TT	1.22+2.22	AA	SELETE	69	141	135	140	128	117
	TT	1.22+2.22	AA	SELETE	75	144	145	112	130	121
	DT	1.22	AA	GEPSEM	64	130	122			
	ST	1.2	D.MARKOS	ONION	25	62				
	DT	1.22	AA	GEPSEM	63	134	139			
	ST	1.2	AA	SELETE	26	45				
	TT	1.22+2.22	AA	CEMENT	84	145	141	114	128	112
	DT	1.22	AA	GEPSEM	67	130	127			
	TT	1.22+2.22	FITCHE	EMPTY	54	42	35	27	24	26
	ST	1.2	CHANCHO	BEER	29	61				
	ST	1.2	G.GURACHA	EMPTY	19	18				

	ST	1.2	AA	MEZE	28	62				
	TT	1.22+2.22	G.GURACHA	EMPTY	57	42	41	32	24	50
	MT	1.2	GONDER	SPICE	43	107				
	DT	1.22	DERBAN	SOIL	52	94	93			
	DT	1.22	DERBAN	SOIL	54	96	101			
	DT	1.22	DERBAN	SOIL	51	95	96			
	TT	1.2+2.2	CHANCHO	CEMENT	60	147	100	89		
	TT	1.22-222	AA	CEMENT	72	1122	119	140	126	122
	ST	1.2	AA	MAZE	27	62				
	MT	1.2	AA	SELETE	42	106				
	ST	1.2	AA	CLINKAN	20	35				
	DT	1.22	AA	GEPSEM	74	138	125			
	DT	1.22	DERBAN	SOIL	50	83	87			
	TT	1.22+2.22	AA	SELETE	77	136	137	105	116	100
	TT	1.22+222	DERBAN	CLINKAN	67	111	115	136	121	135
	ST	1.2	GONDER	CANDLE	25	46				
	TT	1.22-222	DERBAN	CLINKAN	68	108	104	121	121	130
	ST	1.2	GONDER	COSMOTIC	28	60				
	FT	1.22+2.22	AA	FUEL	85	102	103	91	93	94
	TT	1.22-222	DERBAN	POMICHE	66	110	107	109	115	128
	TT	1.22-222	DERBAN	CLINKAN	68	110	113	140	122	133
	TT	1.22-22	AA	GONTAINER	60	53	52	50	50	
	TT	1.2-222	AA	CONTAINER	57	48	48	50	49	
	TT	1.22+2.22	GOJAM	GOODS	59	71	68	79	98	100
	ST	1.2	GONDER	ONION	29	69				
	TT	1.22-222	DERBAN	CLINKAN	68	111	108	138	128	140
	TT	1.22-222	DERBAN	POMICHE	64	94	90	109	120	131
	DT	1.22	DERBAN	SOIL	50	87	89			
	ST	1.2	GONDER	SPICE	30	51				
	ST	1.2	D.MARKOS	GOODS	24	35				
	ST	1.2	CHANCHO	MAZE	29	62				
	ST	1.2	DERBAN	GOODS	25	30				
	ST	1.2	D.MARKOS	METAL	24	60				
	MT	1.2	D.MARKOS	STEEL	54	109				
	ST	1.2	FITCHE	GOODS	24	34				
	ST	1.2	CHANCHO	SAND	27	48				
	ST	1.2	D.MARKOS	SAND	25	60				
	TT	1.22+2.22	AA	SELETE	73	134	135	109	107	100
	MT	1.2	AA	TEFE	47	105				
	ST	1.2	FITCHE	ONION	26	45				
	ST	1.2	D.MARKOS	BANANA	28	67				
	ST	1.2	CHANCHO	GOODS	25	31				
	ST	1.2	GOGAME	SALTE	26	62				
	ST	1.2	FITCHE	GOODS	33	63				
	TT	1.22-222	DERBAN	CLINKAN	67	103	100	112	120	134
	TT	1.22+2.22	GOJAM	MACOT	69	134	122	99	116	110
	TT	1.22-222	AA	CABUE	55	53	50	89	87	81
	HT	1.22	AA	CABUE	70	77	74			
	HT	1.22	AA	CABUE	75	78	77			
	MT	1.2	UMERA	SPICE	41	109				
	ST	1.2	AA	SELETE	29	64				
	TT	1.22+2.22	AA	COTEN	73	63	84	86	80	81
	TT	1.22-222	DERBAN	CLINKAN	67	111	106	130	114	134
	TT	1.22+2.22	POMICHE	CLINKAN	75	108	109	100	112	110
	DT	1.22	DERBAN	SAND	120	125				
	TT	1.22-222	DERBAN	CLINKAN	68	103	105	116	119	130
	TT	1.22-222	DERBAN	CLINKAN	70	109	107	127	117	133
	HT	1.22	AA	LODER	65	120	117			
	TT	1.22+2.22	AA	SELETE	74	130	132	115	124	115
	TT	1.22+2.2	AA	CEMENT	77	103	100	130	115	128

	DT	1.2	CHANCHO	SOIL	59	122				
	TT	1.22-222	DERBAN	POMICHE	66	106	103	133	115	124
	TT	1.22+2.22	AA	SELETE	74	123	125	113	116	109
	HT	1.22	AA	BOTTLE	55	58	57			
	MT	1.2	AA	WOOD	38	103				
	TT	1.2-111	AA	GOTONE	64	84	67	57	65	
	TT	1.2-111	AA	COTONE	63	82	67	67	65	
	TT	1.22-222	DERBAN	CLINKAN	70	102	103	135	125	130
	TT	1.22+2.2	AA	CEMENT	81	139	133	100	93	
	MT	1.2	GONDER	GOODS	42	109				
	ST	1.2	FITCHE	MAZE	30	60				
	ST	1.2	CHANCHO	GOODS	28	64				
	TT	1.22-222	DERBAN	CLINKAN	66	105	107	133	120	132
	ST	1.2	M.TURI	CEMENT	28	60				
	HT	1.22	AA	SELETE	80	141	140			
	DT	1.22	AA	CEMENT	82	128	127			
	DT	1.22	FITCHE	SAND	71	129	126			
	ST	1.2	CHANCHO	GOODS	24	37				
	ST	1.2	CHANCHO	GOODS	27	51				
	HT	1.22	AA	SELIT	80	140	134			
	ST	1.2	FITCHE	BRIKRE	28	59				
	ST	1.2	D.MARKOS	CEMENT	31	62				
	ST	1.2	D.MARKOS	GOODS	26	66				
	ST	1.2	D.MARKOS	OIL	30	53				
	TT	1.22+2.22	AA	CEMENT	85	130	123	108	95	88
	TT	1.22+2.22	DERBAN	CLINKAN	80	109	104	99	107	92
	ST	1.2	GOJAM	GOODS	28	41				
	ST	1.2	FITCHE	SALTE	24	76				
	ST	1.2	FITCHE	MAZE	25	38				
	ST	1.2	GEZU	MAZE	32	58				
	MT	1.2	HUMERA	SPICE	44	108				
	MT	1.2	FITCHE	BEER	40	102				
	ST	1.2	FITCHE	SAND	28	45				
	ST	1.2	FITCHE	SPICE	25	50				
	DT	1.22	CHANCHO	SAND	59	120	125			
	TT	1.22+2.22	D.MARKOS	COCADA	69	131	126	97	120	100
	MT	1.2	D.MARKOS	GOODS	24	37				
	ST	1.2	FITCHE	CAWS	29	64				
	ST	1.2	FITCHE	SALTE	27	63				
	ST	1.2	FITCHE	GOODS	21	35				
	TT	1.22+2.22	AA	MAZE	84	140	133	105	117	98
	MT	1.22	D.MARKOS	CO.MATER	30	68				
	ST	1.2	D.MARKOS	BANANA	29	66				
	ST	1.2	DERA	ONION	30	56				
	ST	1.2	DERA	BEER	29	63				
	ST	1.2	D.MARKOS	SALTE	27	69				
	ST	1.2	D.MARKOS	GOODS	27	67				
	DT	1.22	AA	CEMENT	77	128	126			
	TT	1.22+2.22	AA	SELPHER	77	132	130	102	104	105
	DT	1.22	AA	GEPSEM	62	124	126			
	ST	1.2	FITCHE	BEER	34	60				
	ST	1.2	AA	SELETE	31	62				
	TT	1.22+2.22	D.MARKOS	CEMENT	82	130	128	100	114	105
	ST	1.2	AA	EMPTY	19	18	1			
	TT	1.22+2.22	DEGEN	CEMENT	83	135	129	99	116	107
	TT	1.22-222	DERBAN	POMICHE	62	91	89	107	102	119
	HT	1.2	AA	SOIL	59	132				
	TT	1.22-222	DERBAN	CLINKAN	72	89	87	89	87	80
	DT	1.22	AA	GEPSEM	60	123	118			
	DT	1.22	DERBAN	SOIL	48	102	99			

	HT	1.2	FITCHE	GOODS	64	139				
	FT	1.22-22	SUDAN	EMPTY	54	45	42	46	54	
	FT	1.22-22	SUDAN	EMPTY	55	44	41	52	53	
	MT	1.2	AA	RUASE	52	110				
	DT	1.22	DERBAN	SOIL	54	84	82			
	DT	1.22	AA	COSMOTIC	52	86	89			
	DT	1.22	DERBAN	SOIL	49	89	92			
	DT	1.22	AA	CEMENT	74	120	119			
	TT	1.22-222	AA	CEMENT	66	99	98	119	122	135
	DT	1.22	DERBAN	SOIL	52	93	95			
	ST	1.2	AA	RUASE	27	60				
	TT	1.22+2.22	DEGEN	EMPTY	56	43	42	35	38	30
	DT	1.22	AA	GEPSEM	71	136	132			
	ST	1.2	AA	SPICE	31	59				
	TT	1.22+2.22	DERBAN	CLINKAN	68	95	95	97	96	94
	ST	1.2	AA	WOOD	27	50				
	MT	1.2	AA	RUASE	40	107				
	HT	1.22	AA	WOOD	40	145				
	ST	1.2	AA	LEWZE	31	63				
	MT	1.2	AA	LEWZE	44	108				
	MT	1.2	AA	RUASE	47	107				
	TT	1.22+2.22	DERBAN	CLINKAN	68	96	95	95	98	89
	ST	1.2	D.MARKOS	GOODS	21	41				
	ST	1.2	AA	AL	28	63				
	TT	1.22+2.22	AA	SELITE	75	130	131	117	125	119
	TT	1.22+2.22	DERBAN	CLINKAN	75	110	103	89	110	90
	FT	1.22+2.22	FITCHE	FUEL	74	150	127	116	127	118
	MT	1.2	D.MARKOS	WATER	44	100				
	TT	1.22+2.22	DERBAN	CLINKAN	75	106	105	114	107	98
	TT	1.22+2.22	DERBAN	CLINKAN	75	85	87	92	95	89
	TT	1.22+2.22	DERBAN	CLINKAN	73	97	92	99	97	90
	TT	1.22+2.22	D.MARKOS	SOFT DRINK	71	133	128	84	119	102
	TT	1.22-222	AA	GEPSEM	63	110	110	94	90	120
	TT	1.22+2.22	AA	SELITE	85	135	132	107	125	117
	DT	1.22	DERBAN	SAND	66	130	124			
	TT	1.22+2.22	DERBAN	CLINKAN	74	94	91	93	94	90
	TT	1.22+2.22	AA	SELITE	75	145	142	120	124	118
	TT	1.22+2.22	DERBAN	CLINKAN	69	105	103	96	96	98
	TT	1.22+2.22	DERBAN	CLINKAN	75	115	103	91	99	90
	FT	1.22+2.22	FITCHE	FUEL	74	153	128	113	127	118
	MT	1.2	D.MARKOS	GOODS	43	105				
	TT	1.22+2.22	DERBAN	CLINKAR	74	94	91	93	94	97
	TT	1.22+2.22	AA	SELITE	75	144	140	120	124	118
	TT	1.22+2.22	G.GURACHA	CLINKAR	80	125	117	96	109	112
	MT	1.2	D.MARKOS	GOODS	54	139				
	MT	1.2	AA	WOOD	35	91				
	TT	1.22-22	GONDER	MACHINE	61	126	137	140	120	
	TT	1.22+2.22	AA	SELITE	85	137	117	104	125	127
	TT	1.22+2.22	AA	SELITE	88	129	133	113	123	130
	TT	1.22-222	G.GURACHA	CLINKAN	63	107	100	116	122	133
	ST	1.2	GONDER	GOODS	28	61				
	ST	1.2	D.MARKOS	FERNITHER	36	40				
	DT	1.22	AA	GEPSEM	63	134	135			
	HT	1.22	DERBAN	EMPTY	64	54	51			
	ST	1.2	GONDER	COMICAL	36	63				
	ST	1.2	FITCHE	GOODS	27	51				
	MT	1.2	B.DARE	ONION	45	116				
	MT	1.2	GONDER	SPICE	39	112				
	MT	1.2	GONDER	SPICE	41	108				
	ST	1.2	G.GURACHA	LEWZE	29	58				

	FT	1.22-222	AA	FUEL	63	111	102	109	120	135
	TT	1.22-222	AA	CLINKAR	64	98	95	125	120	126
	TT	1.22+2.22	AA	BEER	70	130	122	96	122	112
	ST	1.2	GONDER	GOODS	27	70				
	DT	1.22	G.GURACHA	SAND	72	134	134			
	DT	1.22	FITCHE	SAND	60	132	135			
	DT	1.22	CHANCHO	SAND	51	138	136			
	TT	1.2+2.22	FITCHE	EMPTY	59	43	43	31	27	27
	ST	1.2	AA	SPICE	19	37				
	MT	1.2	D.MARKOS	DRWGE	49	53				
	TT	1.22+2.22	D.MARKOS	STEEL	70	121	123	95	113	96
	ST	1.2	D.MARKOS	GOODS	19	57				
	FT	1.22-22	DERBAN	FUEL	58	130	120	135	139	
	ST	1.2	D.MARKOS	SPICE	26	61				
	DT	1.22	AA	GEPSEM	79	134	124			
	DT	1.22	FITCHE	SAND	54	122	120			
	TT	1.22+2.22	G.GURACHA	CEMENT	84	135	133	125	130	118
	ST	1.2	GONDER	STEEL	22	51				
	FT	1.22+2.22	SUDAN	EMPTY	55	56	51	43	48	40
	TT	1.22+2.22	GONDER	FLOWER	73	135	132	113	116	
	ST	1.2	D.MARKOS	STEEL	26	59				
	ST	1.2	D.MARKOS	GOODS	40	61				
	TT	1.22+2.22	AA	CEMENT	90	138	132	125	122	118

APPENDIX M: Regression Outputs and Residual Analysis

RUTTING VS IN-PLACE PROPERTIES

Rutting vs In-Place Va

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.50
R Square	0.25
Adjusted R Square	0.10
Standard Error	25.50
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	1092.73	1092.73	1.68	0.25
Residual	5.00	3250.68	650.14		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	69.15	27.82	2.49	0.06	-2.37	140.67	-2.37	140.67
Va (%)	-17.03	13.14	-1.30	0.25	-50.79	16.74	-50.79	16.74

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	26.71	-12.46	-0.5
2.00	52.53	-23.78	-1.0
3.00	35.08	13.12	0.6
4.00	33.87	4.73	0.2
5.00	43.41	-21.01	-0.9
6.00	11.34	-3.94	-0.2
7.00	44.26	43.34	1.86

Rutting vs In-Place VMA

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.14
R Square	0.02
Adjusted R Square	-0.18
Standard Error	29.19
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	82.34	82.34	0.10	0.77
Residual	5.00	4261.07	852.21		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	10.77	79.73	0.14	0.90	-194.17	215.72	-194.17	215.72
VMA (%)	2.40	7.73	0.31	0.77	-17.47	22.28	-17.47	22.28

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	30.12	-15.87	-0.6
2.00	36.00	-7.25	-0.3
3.00	40.74	7.46	0.3
4.00	38.97	-0.37	0.0
5.00	35.52	-13.12	-0.5
6.00	33.46	-26.06	-1.0

Rutting vs In-Place VFA

SUMMARY OUTPUT with outlier

Regression Statistics	
Multiple R	0.51
R Square	0.26
Adjusted R Square	0.11
Standard Error	25.32
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	1137.98	1137.98	1.78	0.24
Residual	5.00	3205.43	641.09		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	-79.51	86.71	-0.92	0.40	-302.42	143.40	-302.42	143.40
VFA (%)	1.44	1.08	1.33	0.24	-1.34	4.21	-1.34	4.21

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	19.73	-5.48	-0.2
2.00	50.72	-21.97	-1.0
3.00	41.21	6.99	0.3
4.00	38.89	-0.29	0.0
5.00	43.21	-20.81	-0.9
6.00	12.58	-5.18	-0.2
7.00	40.87	46.73	2.02

SUMMARY OUTPUT without outlier

Regression Statistics	
Multiple R	0.71
R Square	0.50
Adjusted R Square	0.38
Standard Error	11.98
Observations	6.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	579.72	579.72	4.04	0.11
Residual	4.00	574.26	143.57		
Total	5.00	1153.99			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	-56.01	41.40	-1.35	0.25	-170.96	58.94	-170.96	58.94
VFA (%)	1.04	0.52	2.01	0.11	-0.40	2.48	-0.40	2.48

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	15.97	-1.72	-0.2
2.00	38.45	-9.70	-0.9
3.00	31.55	16.65	1.6
4.00	29.86	8.74	0.8
5.00	32.99	-10.59	-1.0
6.00	10.78	-3.38	-0.3

Rutting vs In-Place Stability

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.40
R Square	0.16
Adjusted R Square	0.00
Standard Error	26.97
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	706.98	706.98	0.97	0.37
Residual	5.00	3636.44	727.29		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	63.42	30.28	2.09	0.09	-14.41	141.25	-14.41	141.25
Stability (KN)	-3.27	3.32	-0.99	0.37	-11.79	5.25	-11.79	5.25

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	48.26	-34.01	-1.4
2.00	33.86	-5.11	-0.2
3.00	33.01	15.19	0.6
4.00	44.84	-6.24	-0.3
5.00	28.29	-5.89	-0.2
6.00	16.75	-9.35	-0.4
7.00	42.19	45.41	1.84

Rutting vs In-Place Flow

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.95
R Square	0.90
Adjusted R Square	0.88
Standard Error	9.49
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	3893.38	3893.38	43.26	0.00
Residual	5.00	450.04	90.01		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	-58.57	14.72	-3.98	0.01	-96.41	-20.74	-96.41	-20.74
Flow (1/100")	24.53	3.73	6.58	0.00	14.94	34.12	14.94	34.12

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	16.09	-1.84	-0.2
2.00	29.42	-0.67	-0.1
3.00	36.53	11.67	1.3
4.00	42.34	-3.74	-0.4
5.00	38.00	-15.60	-1.8
6.00	1.61	5.79	0.7
7.00	83.22	4.38	0.51

Rutting vs In-Place Marshall Quotient

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.60
R Square	0.37
Adjusted R Square	0.24
Standard Error	23.47
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	1588.28	1588.28	2.88	0.15
Residual	5.00	2755.14	551.03		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	61.61	17.85	3.45	0.02	15.73	107.48	15.73	107.48
Marshall quotient (kN/mm)	-2.49	1.47	-1.70	0.15	-6.26	1.28	-6.26	1.28

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	46.01	-31.76	-1.5
2.00	36.34	-7.59	-0.4
3.00	36.78	11.42	0.5
4.00	44.95	-6.35	-0.3
5.00	31.44	-9.04	-0.4
6.00	1.52	5.88	0.3
7.00	50.16	37.44	1.75

RUTTING VS RE-COMPACTED PROPERTIES

Rutting vs Re-compacted Va

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.05
R Square	0.00
Adjusted R Square	-0.20
Standard Error	29.44
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	8.97	8.97	0.01	0.92
Residual	5.00	4334.45	866.89		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	38.15	29.99	1.27	0.26	-38.96	115.25	-38.96	115.25
Va (%)	-0.59	5.80	-0.10	0.92	-15.49	14.31	-15.49	14.31

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	35.59	-21.34	-0.8
2.00	36.17	-7.42	-0.3
3.00	37.11	11.09	0.4
4.00	35.98	2.62	0.1
5.00	34.36	-11.96	-0.4
6.00	33.80	-26.40	-1.0
7.00	34.18	53.42	2.0

SUMMARY OUTPUT (without outlier)

Regression Statistics	
Multiple R	0.84
R Square	0.71
Adjusted R Square	0.64
Standard Error	9.15
Observations	6.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	819.31	819.31	9.79	0.04
Residual	4.00	334.68	83.67		
Total	5.00	1153.99			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	54.30	9.61	5.65	0.00	27.63	80.97	27.63	80.97
Va (%)	-6.18	1.97	-3.13	0.04	-11.66	-0.70	-11.66	-0.70

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	27.53	-13.28	-1.6
2.00	33.63	-4.88	-0.6
3.00	43.44	4.76	0.6
4.00	31.59	7.01	0.9
5.00	14.61	7.79	1.0
6.00	8.80	-1.40	-0.2

Rutting vs Re-compacted VMA

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.31
R Square	0.10
Adjusted R Square	-0.08
Standard Error	28.02
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	417.41	417.41	0.53	0.50
Residual	5.00	3926.00	785.20		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	-32.96	94.24	-0.35	0.74	-275.22	209.29	-275.22	209.29
VMA (%)	5.33	7.31	0.73	0.50	-13.46	24.13	-13.46	24.13

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	19.21	-4.96	-0.2
2.00	34.41	-5.66	-0.2
3.00	32.37	15.83	0.6
4.00	37.29	1.31	0.1
5.00	45.80	-23.40	-0.9
6.00	37.22	-29.82	-1.2
7.00	40.90	46.70	1.8

Rutting vs Re-compacted VFA

SUMMARY OUTPUT with outlier

Regression Statistics	
Multiple R	0.17
R Square	0.03
Adjusted R Square	-0.16
Standard Error	29.02
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	132.30	132.30	0.16	0.71
Residual	5.00	4211.12	842.22		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	15.26	51.77	0.29	0.78	-117.82	148.34	-117.82	148.34
VFA (%)	0.32	0.81	0.40	0.71	-1.75	2.39	-1.75	2.39

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	33.08	-18.83	-0.7
2.00	38.77	-10.02	-0.4
3.00	42.65	5.55	0.2
4.00	38.32	0.28	0.0
5.00	33.33	-10.93	-0.4
6.00	29.35	-21.95	-0.8
7.00	31.71	55.89	2.1

SUMMARY OUTPUT without outlier

Regression Statistics	
Multiple R	0.96
R Square	0.92
Adjusted R Square	0.90
Standard Error	4.84
Observations	6.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	1060.25	1060.25	45.25	0.00
Residual	4.00	93.73	23.43		
Total	5.00	1153.99			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	-35.55	9.45	-3.76	0.02	-61.78	-9.31	-61.78	-9.31
VFA (%)	0.96	0.14	6.73	0.00	0.56	1.36	0.56	1.36

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	18.07	-3.82	-0.9
2.00	35.19	-6.44	-1.5
3.00	46.86	1.34	0.3
4.00	33.83	4.77	1.1
5.00	18.82	3.58	0.8
6.00	6.83	0.57	0.1

Rutting vs Re-compacted Stability

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.25
R Square	0.06
Adjusted R Square	-0.12
Standard Error	28.52
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	277.56	277.56	0.34	0.58
Residual	5.00	4065.86	813.17		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	5.97	51.37	0.12	0.91	-126.07	138.02	-126.07	138.02
Stability (KN)	2.53	4.33	0.58	0.58	-8.61	13.68	-8.61	13.68

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	24.18	-9.93	-0.4
2.00	31.81	-3.06	-0.1
3.00	42.57	5.63	0.2
4.00	31.42	7.18	0.3
5.00	43.62	-21.22	-0.8
6.00	36.67	-29.27	-1.1
7.00	36.93	50.67	1.95

Rutting vs Re-compacted Flow

SUMMARY OUTPUT with outlier

Regression Statistics	
Multiple R	0.48
R Square	0.23
Adjusted R Square	0.07
Standard Error	25.93
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	980.73	980.73	1.46	0.28
Residual	5.00	3362.68	672.54		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	-62.38	81.49	-0.77	0.48	-271.87	147.11	-271.87	147.11
Flow (1/100")	35.07	29.04	1.21	0.28	-39.58	109.73	-39.58	109.73

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	19.33	-5.08	-0.2
2.00	55.10	-26.35	-1.1
3.00	43.53	4.67	0.2
4.00	40.37	-1.77	-0.1
5.00	27.40	-5.00	-0.2
6.00	22.49	-15.09	-0.6
7.00	38.97	48.63	2.05

SUMMARY OUTPUT without outlier

Regression Statistics	
Multiple R	0.72
R Square	0.52
Adjusted R Square	0.39
Standard Error	11.82
Observations	6.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	594.75	594.75	4.25	0.11
Residual	4.00	559.24	139.81		
Total	5.00	1153.99			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	-49.61	37.27	-1.33	0.25	-153.08	53.85	-153.08	53.85
Flow (0.25mm)	27.53	13.35	2.06	0.11	-9.53	64.59	-9.53	64.59

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	14.53	-0.28	0.0
2.00	42.61	-13.86	-1.3
3.00	33.53	14.67	1.4
4.00	31.05	7.55	0.7
5.00	20.86	1.54	0.1
6.00	17.01	-9.61	-0.9

Rutting vs Re-compacted Marshall Quotient

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.03
R Square	0.00
Adjusted R Square	-0.20
Standard Error	29.46
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	4.85	4.85	0.01	0.94
Residual	5.00	4338.57	867.71		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	38.87	48.89	0.80	0.46	-86.81	164.55	-86.81	164.55
Marshall quotient (kN/mm)	-0.21	2.83	-0.07	0.94	-7.50	7.07	-7.50	7.07

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	36.26	-22.01	-0.8
2.00	36.29	-7.54	-0.3
3.00	34.82	13.38	0.5
4.00	35.97	2.63	0.1
5.00	33.95	-11.55	-0.4
6.00	34.63	-27.23	-1.0
7.00	35.29	52.31	1.95

Rutting vs Coarse Aggregate Angularity

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.64
R Square	0.41
Adjusted R Square	0.29
Standard Error	22.68
Observations	7.00

ANOVA

	df	SS	MS	F	Significance F
Regression	1.00	1771.12	1771.12	3.44	0.12
Residual	5.00	2572.29	514.46		
Total	6.00	4343.41			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	457.74	227.83	2.01	0.10	-127.91	1043.39	-127.91	1043.39
% coarse aggregate with 2 or more crushed faces (>4.75mm)	-4.85	2.62	-1.86	0.12	-11.57	1.87	-11.57	1.87

RESIDUAL OUTPUT

Observation	Predicted Measured average rut depth(mm)	Residuals	Standard Residuals
1.00	39.00	-24.75	-1.2
2.00	13.06	15.69	0.8
3.00	35.94	12.26	0.6
4.00	21.49	17.11	0.8
5.00	41.31	-18.91	-0.9
6.00	29.45	-22.05	-1.1
7.00	66.95	20.65	1.00