EFFECT OF RECYCLED CEMENT CONCRETE CONTENT ON RUTTING BEHAVIOR OF ASPHALT CONCRETE

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Approval of the Thesis:

EFFECT OF RECYCLED CEMENT CONCRETE CONTENT ON RUTTING BEHAVIOR OF ASPHALT CONCRETE

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ABSTRACT

EFFECT OF RECYCLED CEMENT CONCRETE CONTENT
ON RUTTING BEHAVIOR OF ASPHALT CONCRETE

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Disposed waste materials remained from demolished buildings have been an environmental problem especially for developing countries. Recycled Cement Concrete (RCC) is one of the abundant components of waste materials that include quality aggregates. Use of RCC in asphalt concrete pavements is economically a feasible option as it not only helps in recycling waste materials but also preserves natural resources by fulfilling the demand for quality aggregate in pavement constructions. However, due to variability in RCC characteristics, a detailed evaluation of its effect on asphalt concrete performance is required.

In this study, effect of RCC content on rutting potential of asphalt concrete is investigated using laboratory prepared specimens. Rutting susceptibility of the specimens is determined using repeated creep tests performed in the uniaxial stress mode. Because of the aspect ratio requirements for the repeated creep test, the standard Marshall mix design procedures were modified based on the energy concept by changing the compactor device and the applied design number of blows. The modified specimens were tested to determine a number of parameters that can describe the rutting behavior of the tested mixes. The findings indicate that slope constant and flow number give relatively stronger relationships with rutting behavior as compared to the other rutting parameters. While increasing the

RCC content yields improved rutting performance for coarse graded specimens, it dramatically reduces the performance for fine graded specimens.

Key words: Rutting Potential, Recycled Cement Concrete, Repeated Creep Test

ÖZ

GERİ KAZANILMIŞ ÇİMENTO BETON İÇERİĞİNİN ASFALT BETONUN TEKERLEK İZİ DAVRANIŞI ÜZERİNDEKİ ETKİSİ

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Yıkıntı binalardan kalan atık malzemeler özellikle gelişmekte olan ülkeler için çevre problemi oluşturmaktadır. İçeriğinde kaliteli agrega bulunan geri kazanılmış çimento beton malzemesi atık malzemeler içerisinde en yaygın olanıdır. Ekonomik bir seçenek olarak geri kazanılmış çimento beton atıkların beton asfalt yapımında kullanımı, sadece atık malzemelerin değerlendirilmesinde değil aynı zamanda yol inşaatlarında gereken kaliteli agrega ihtiyacının karşılanmasına da yardımcı olmaktadır. Fakat, geri kazanılmış çimento betonun özelliklerinin değişkenlik göstermesi, bu malzemenin beton asfalt performansı üzerindeki etkilerinin detaylı araştırılmasını gerektirmektedir.

Bu çalışmada, geri kazanılmış çimento betonun, beton asfaltın tekerlek izi davranışı üzerindeki etkisi laboratuvar numunleri kullanılarak araştırılmaktadır. Numunelerin tekerlek izine karşı duyarlığı, tek yönlü basınç modunda yapılan tekrarlı sünme deneyleri kullanılarak belirlenmiştir. Tekrarlı sünme deneyinde gereken en-boy oranı için kompaktör makinası ve Marshall karışım tasarım yöntemleri enerji prensibine göre değiştirilmiştir. Bu numuneler kullanılarak yapılan deneylerden tekerlek izi davranışını belirleyen birçok parametre elde edilmiştir. Deney bulguları, eğim sabiti ve akma sayısının tekerlek izi davranışıyla diğer parametrelere kıyasla daha güçlü bir bağlantısı olduğunu göstermiştir. Geri kazanılmış çimento beton miktarındaki artışın kaba gradasyonlu numunelerin tekerlek izi performansını artırmasına karşın, ince gradasyonlu numunlerin performanslarını önemli ölçüde azalttığı görülmüştür.

Anahtar kelimeler: Tekerlek İzi Potansiyeli, Geri Kazanılmış Çimento Beton, Tekrarlı Sünme Deneyi

I wish to dedicate this work

To my IDEAL

and every person from whom I have learned anything in life

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TABLE OF CONTENTS

LIST OF TABLES

TABLES

LIST OF FIGURES

FIGURES

LIST OF SYMBOLS

AASHTO:	American Association of State Highway and Transportation Officials
ASCII:	American Standard Code for Information Interchange
ASTM:	American Society of Testing and Materials
CDAS:	Control and Data Acquisition System
FHWA:	Federal Highway Administration
HMA:	Hot Mix Asphalt
LVDT:	Linear Variable Displacement Transducers
METU:	Middle East Technical University
OAC:	Optimum Asphalt Content
RCC:	Recycled Cement Concrete
UMATTA:	Universal Materials Testing Apparatus
VTM:	Void in Total Mix

CHAPTER 1

INTRODUCTION

1.1 Background

Rutting is the formation of depressions along the road surface as a result of accumulated permanent deformations caused by the passage of heavy vehicles. Rutting is one of the major distress mechanisms in flexible pavements mainly dependent on asphalt mix compositions, i.e., gradation, binder type, air void content, and the degree of both construction compaction and traffic densification under service conditions. It also depends on pavement temperature, characteristics of traffic loading, i.e., number of load repetitions, axle weight, and properties of pavement layers, i.e., layer thicknesses, materials quality as well as subgrade support conditions. Hence, to prevent rutting distress, high quality materials together with sound construction techniques should be selected during the construction of asphalt pavements. On the other hand, use of high quality materials is limited, thus raising the need of utilization of quality materials in other sectors of the construction industry. Recycling waste materials has become a common practice in the last 20 years in various fields of both manufacturing and construction sectors. Recycled Cement Concrete (RCC) has been one of the feasible choices of recycling concrete aggregates in asphalt pavement constructions because of its various benefits; It is an economical option obtained from the debris of demolished buildings or wreckage of old rigid pavements. It generally contains good quality aggregates that can be effectively re-utilized in pavement structures. Besides, unlike fresh aggregates, obtaining RCC doesn't disturb the environment and vegetation by digging ground and making noisy queries.

Waste materials can be disposed of in three ways: (a) Through incineration, (b) Land filling, and (c) Recycling. Concrete cannot however, be incinerated. Disposing the waste material through land filling is not an easy process as well, because it causes further environmental problems. A feasible option that remains is to recycle these materials in the pavement constructions. RCC aggregates seemed to be a good option to recycle them in various parts of pavements layers for many researchers because it can provide not only high quality aggregate to the structural layers of pavements but also an economical solution to the disposing problems. The term crushed cement concrete is understood as the resulting material once the concrete debris has undergone a crushing process. This resulting material is: (i) Cement pieces made of Portland clinker, and (ii) Natural aggregates which either are rounded or crushed materials. The size of concrete waste materials produced from the demolition of construction elements can have large variations depending on the demolition technique used. The chemical composition of debris after the demolition process depends on the particular composition of the former components used in their production, as more than 75% of the total material contains natural aggregates, and the remaining portion made of cement hydration compounds, hydrated calcium silicates, and aluminates or calcium hydroxides. The acquirement of RCC materials follows two phases: (a) Demolition of structural elements, and (b) Transforming crushed debris into aggregates.

The demolition process can help to reduce the impurity present in the debris materials. The crushers used for RCC materials are quite similar to those of natural aggregates. These crushers can have a jaw, impact, or conic crushing mechanisms.

RCC materials have typical characteristics that make them different from the natural aggregates: RCC aggregates may present relatively high irregular shape, rougher surface and more porous texture than the natural aggregates. The specific gravity of RCC is generally smaller than that of natural aggregates. In addition, RCC aggregates have more absorption than natural aggregates. The chemical characterization of recycled concrete aggregates is similar to the waste material from which it is obtained. In the RCC materials, part of the cement hydration components remain adhered to the fines particles. Coarse RCC aggregates larger than 4.75 mm size show a decent bearing capacity and good resistance to abrasion and sulphate attacks.

1.2 Objective of the Study

In this study, rutting performance of HMA prepared with different percentages of RCC is investigated under repeated creep permanent deformation tests. The objective of this research is to (i) Modify the standard Marshall test procedure in order to prepare taller specimens (ii) Perform repeated load permanent deformation tests on the modified Marshall specimens to determine the rutting performance (iii) Investigate the effects of addition of RCC aggregate on the rutting performance of the test specimens (iv) Determine an empirical model and investigate the significant model parameters characterizing the rutting behavior.

1.3 Scope

The study was conducted in two phases. In the first phase, laboratory mixtures were prepared using two design variables, i.e., gradation and asphalt content, at two levels. The gradations were selected according to the ASTM D3515-01 specifications while the asphalt contents were adjusted to typical levels used in the standard Marshall mix design procedure. Both modified (tall) Marshall specimens and standard Marshall specimens were prepared and their compaction levels were compared based on the air void content and the energy per unit volume of the specimens. A design number of blows was then proposed which can be used for the modified Marshall specimens.

In the second phase, laboratory specimens were prepared using varying percentages of RCC aggregate based on the modified Marshall mix design procedure. Optimum asphalt content calculation was carried out in order to achieve around 4% air voids in all the test specimens. Repeated load creep tests were then conducted to determine the effect of recycling content on rutting performance of the specimens. Several empirical models and model parameters were calculated characterizing the rutting behavior. Based on the findings, a result and discussions section, and conclusions were presented accordingly.

The remaining part of the study can be summarized as follows: A comprehensive literature review regarding the use of recycled materials in pavement constructions, theory of rutting test and relevant previous works is presented in Chapter 2. Materials and methods used in the study are given in Chapter 3. In Chapter 4, results and discussions for the modification of standard Marshall mix design procedures are presented. Chapter 5 concentrates on the discussion of rutting tests performed on the modified Marshall specimens, the empirical models, and the significant model parameters to characterize the rutting behavior in relation to the RCC content. In Chapter 6, conclusions and recommendations for future study are provided.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

In this chapter, a review of literature from varying sources is presented. Initially, a description of the theory of rutting is briefly given along with different types of rutting that are common in asphalt concrete pavements such as surface consolidation, surface wear, plastic flow, and structural failures. The consequences of rutting for road users or travelers are then highlighted. Afterwards, the effect of specimen size on rutting is discussed and the use of different recycled materials, e.g., aggregates from recycled Portland cement concrete, construction debris, recycled plastics, waste roofing shingles, and recycled solid waste materials in asphalt concrete is introduced. Finally, the effect of recycled concrete aggregate on rutting performance of asphalt concrete is explained.

2.2 Rutting in Asphalt Concrete Pavements

Asphalt pavement rutting is a destructive distress mechanism in the form of a continuous longitudinal surface depression along the wheel path in flexible pavements. Ruts are generally formed in the direction of traffic making transverse waves on the pavement surface. Rutting is more widespread particularly in urban roads at intersection locations. It can also occur in all layers of the pavement structure especially in areas associated with commercial vehicles such as in curb lanes, bus bays, and turn lanes (Burlier, R. and Emery, J., 1997).

Rutting results from lateral distortion and densification and mostly occurs when traffic loading displaces the bituminous material on flexible pavements. The material is displaced either vertically or laterally from the wheel tracks toward the shoulder and centerline and between the wheel tracks. Hence, rutting is accompanied, in most cases, by pavement upheaval along the sides of the rut. Moreover, rutting represents a continuous accumulation of incrementally small permanent deformations formed from each load application. Generally, there are three causes of rutting in asphalt pavements: accumulation of permanent deformation in the asphalt surface, permanent deformation of subgrade, and wear of pavements caused by studded tires. In the past, subgrade deformation was considered to be the main cause of rutting and many pavement design techniques applied a limiting criterion on vertical stress at the subgrade level. However, a recent research indicates that most of the rutting occurs in the upper part of the asphalt pavements. These three causes of rutting can also act in combination, that is, rutting could be the sum of permanent deformations in all layers and wears from studded tires (Garba, 2002).

Permanent deformation in asphalt pavements is primarily dependent on the properties of asphalt mix, i.e., aggregate characteristics, gradation, asphalt cement stiffness, design methodology, binder type, and the degree of asphalt mix compaction. Other significant factors for rutting that are not related to the mix properties are pavement temperature, axle load, lateral wander of traffic, tire type, tire pressure, axle configuration, vehicle type, vehicle speed/stopping, etc. The road substructure characteristics, i.e., thickness, quality of materials in the base and subbase layers, and the subgrade bearing capacity also play a vital role in dealing with the rutting distress.

If rutting in a particular road is not controlled or prevented in the earlier stages, it might lead to serious failure in the road and hence economical loss due to major rehabilitation or a total reconstruction of the entire section. In order to prevent this distress, selection of a good quality of asphalt mix is required to limit the rut formation to a certain degree during the service life of the pavement. Rut-resistant hot-mix asphalt must be designed and placed with caution to avoid segregation and excessive compaction that may lead to “harsh” mixes. Asphalt mixes should be designed to achieve high resistance to permanent deformation in the field where the asphalt concrete is subjected to heavy and long duration traffic particularly during hot seasons without sacrificing resistance to fatigue distress and stripping problems (Piarc, 1995). The key to achieving satisfactory performance against rutting at intersections is to use materials and design methodologies that will prevent rutting during the service life of the pavement (Buncher, 2002). Since nearly all flexible pavements are vulnerable to rutting, one of the most vital aspects in pavement research is to set up reliable rutting prediction models for pavement design and management purposes.

2.3 Significance of Rutting Distress in Asphalt Pavements

Rutting of any degree or form is undesirable, as it results in many possible hazards for both drivers and the pavement structure. For instance, accumulation of water in the ruts can lead to pavement deterioration because of long standing and freezing water. Ponding of water in the ruts can also cause hydroplaning, in other words, aquaplaning, which is the loss of contact between the vehicle tire and the wet road surface, which might lead to fatal accidents. If the rut depression is deep enough, the wheel tracks alone can also pose problems. Controlling vehicles in such roads is more difficult due to large surface irregularities, which may result in unexpected change in the car’s direction. Besides, traveling on a rutted road is also quite uncomfortable for travelers, which significantly reduces the serviceability performance of the pavement.

A recent work by the Norwegian Public Road Administration (NPRA) shows that the accident rate increases with the amount of rut depth in the pavement as shown in Figure 2.1. It can be seen that the occurrence of accident becomes higher when driving on roads with a rut depth of more than 6.5 mm.

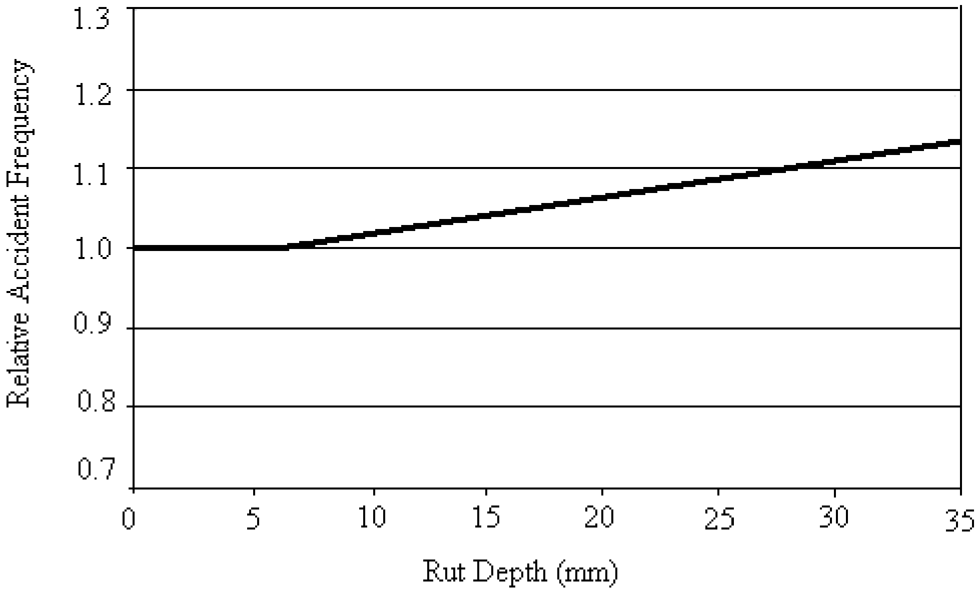

Figure 2.1 Rut Depth vs. Relative Accident Frequency (Optimal Road Maintenance and Operations – Results of New Research and Analysis in Norway" NORDIC Road and Transportation Research NPRA, Norway, 2002)

2.4 Rut Profile Definition and Measurement

Figure 2.2 shows a rut depth profile in which depressions are measured with respect to the ground level. The heave is determined by measuring the total height at which the pavement surface has raised from the ground level. The rut depth is measured as the sum of the depression and the heave. The severity of rutting can be described as follows; Low severity: rut depths less than 12 mm. Medium severity: rut depth between 12 - 25 mm. High severity: rut depths greater than 25 mm, at which the potential for hydroplaning is high. Rutting is measured by determining the depth of each rut over a square meter of surface area.

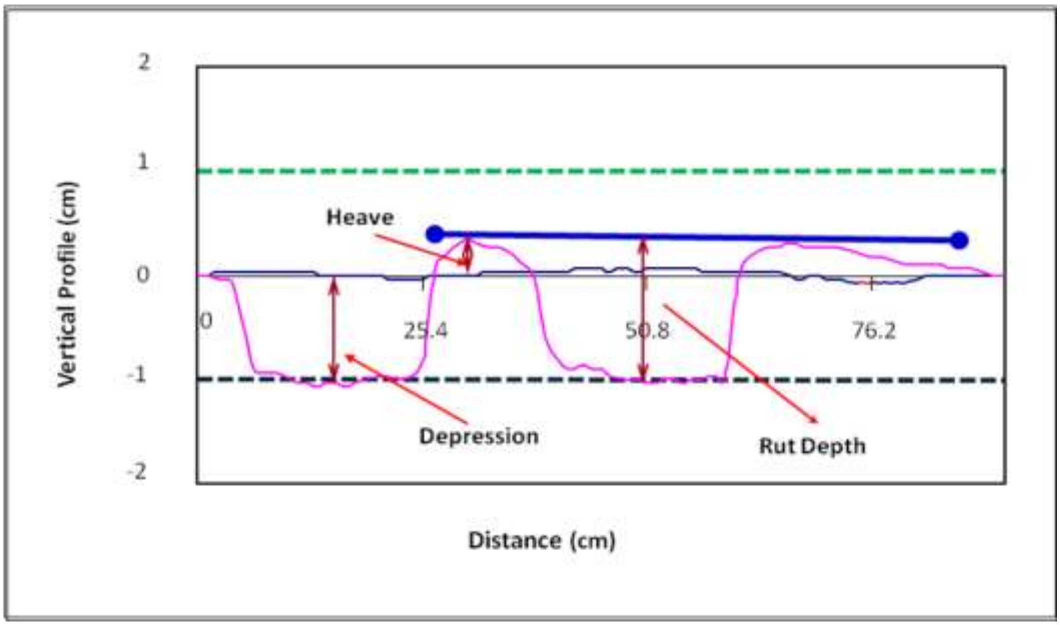

Figure 2.2 Rut profile example (Wu and Hossain, 2002)

On large roadways, it is often measured in the centerline distance of rutting. The severity of rutting is determined by measuring the average depth of each wheel path rut with a straight edge. The rut depths are measured over standard interval, such as every 20 m. The average rut depth is calculated by laying the straight edge across the rut and measuring its depth. Using the measurement taken along the length or the centerline of the rut, its depth is then computed in millimeters (Lavin, 2003).

2.5 Theory of Rutting in Asphalt Concrete Pavements

The resistance to permanent deformation is measured using repeated load test and the axial creep test. Researchers at the Shell Laboratory in Amsterdam conducted extensive studies using the unconfined creep test as the basis for predicting rut depth in asphalt concrete. It was reported that the creep test must be performed at comparatively low stress levels within the linear range of the material to obtain good comparisons between the rut depths observed in test tracks and those computed using creep test data. The need to use stress levels within the linear range has been credited to the fact that the loading time in the field is small compared to the loading time in the creep tests. Strain measured as a function of the loading time at a fixed test temperature is the usual output of the creep test. Results of the creep test are found to be independent of the shape and height to diameter ratio of the sample, provided that the specimen's ends are parallel, flat, and well lubricated.

Repeated load tests have been used to characterize permanent deformation response under more realistic conditions than those of the creep test. It has been argued that the permanent strain which gradually accumulates under repeated loading is essentially a creep phenomenon, i.e., it is the loading time rather than the number of load applications which controls the permanent strain. However, the pulse shape and duration were found to greatly influence the measurements. Rest period between load cycles does not influence the basic permanent strain against time relationship where the time refers to the time when the material is actually being loaded. The rate of accumulation of permanent deformation, i.e., the accumulated permanent deformation per cycle of load application is often referred to as creep rate or rutting rate. The creep rate is often calculated in the secondary creep range, the straight-line portion of the creep curve. However, this has proved to be difficult in many cases because generally there is no part in the creep curve with a constant slope. In addition, some specimens can fail or enter the tertiary creep range without showing any distinct secondary creep range and others undergo large deformation apparently in the primary creep range. In response to creep loading, both static and cyclic, asphalt concrete materials develop permanent deformation which accumulates with time or number of load repetitions. This accumulated permanent deformation is the cause of rutting in asphalt pavements. The plot of the accumulated strain against time is often called creep curve (Garba, 2002).

The creep of a material can be divided into three stages on a curve between creep strain and time, as shown in Figure 2.3. Before the start of the first stage, there is an instantaneous deformation at a constant time interval (t = 0). The first stage, or the primary creep, starts at a rapid rate and slows down with time. The second stage or the secondary creep has a relatively uniform rate. The third stage or the tertiary creep has an accelerating creep rate and terminates by failure of material at the time of rupture. If the load is continuously applied, the material continues to deform with a decreasing rate. In other words, the permanent strain rate slows down with time. The behavior is such that after unloading the material, a certain part of deformation is recoverable while the rest is irrecoverable. The physical damage process during the primary stage is called strain hardening. This damage occurs on the field due to the movement of dislocations in asphalt concrete under repeated traffic loading, thus resulting in increase in the plastic strain (Zhou and Scullion, 2002).

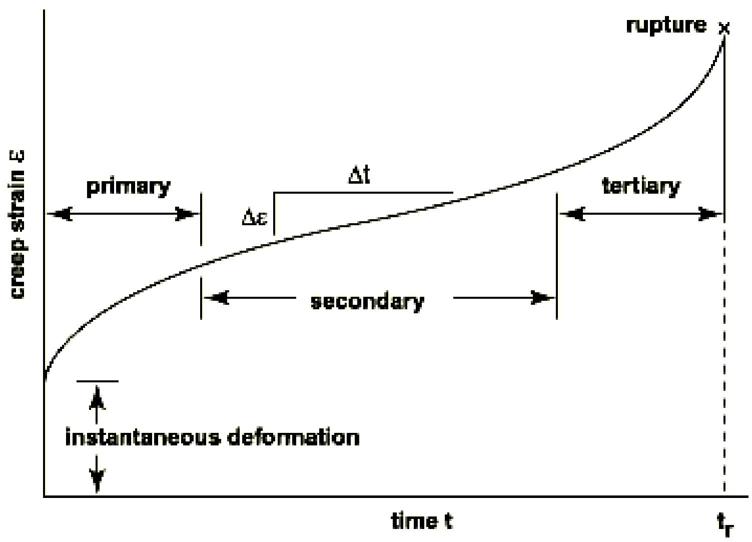

Figure 2.3 Creep Curve (Martin Tarr, "Stress and its effect on materials")

The haversine loading pattern for the permanent deformation test is shown in the graph between the load applied and the time interval in Figure 2.4. The loading period is 0.1 seconds and the rest period is 0.9 seconds. The contact load $P_{contact}$ remains constant throughout the test while the cyclic load P_{cyclic} starts from the minimum load and consistently increases at a uniform rate until it reaches the maximum load, hence this cycle continues throughout the test duration within only 0.1 seconds. During the rest period of 0.9 seconds, only the fixed load $P_{contact}$ is applied.

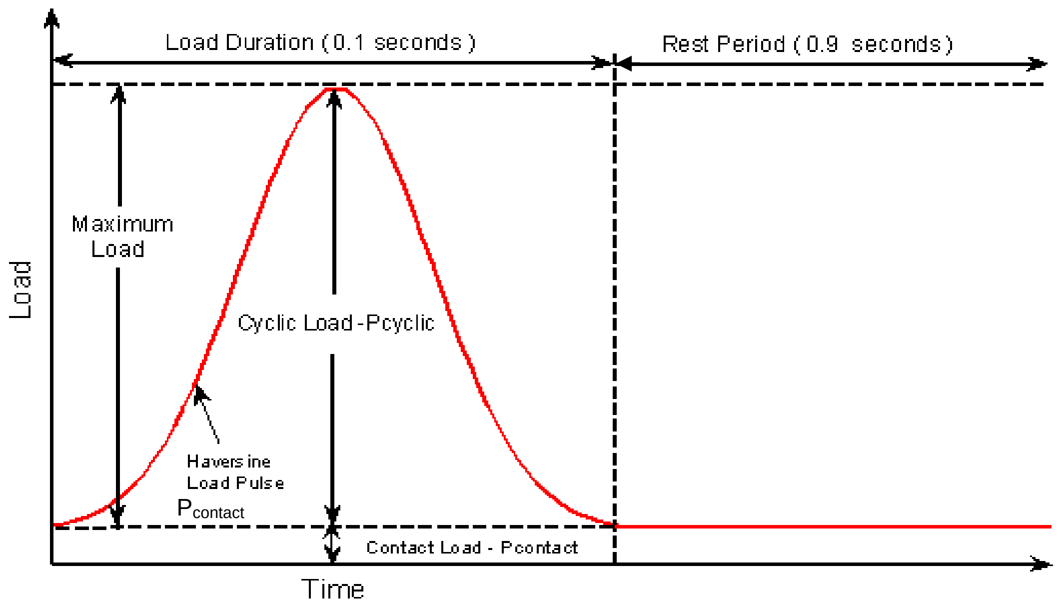

Figure 2.4 Haversine loading pattern applied in a permanent deformation test (National Cooperative Highway Research Program, Research Results Digest Number 285, 2004)

The plot in Figure 2.5 explains a relationship between permanent strain and number of cycles on a log–log scale. This curve also consists of three stages, i.e., primary, secondary, and tertiary stages, where the slope of the curve is calculated during the secondary stage using the formula (Kanitpong et al., 2006):

$$\log (\varepsilon (N)) = a + b \log (N) \tag{2.1}$$

Where N is the applied number of cycles, ε (N) is the permanent strain at cycle N, the intercept constant that can be calculated during the primary stage, and the slope constant b calculated for the second stage of the creep curve.

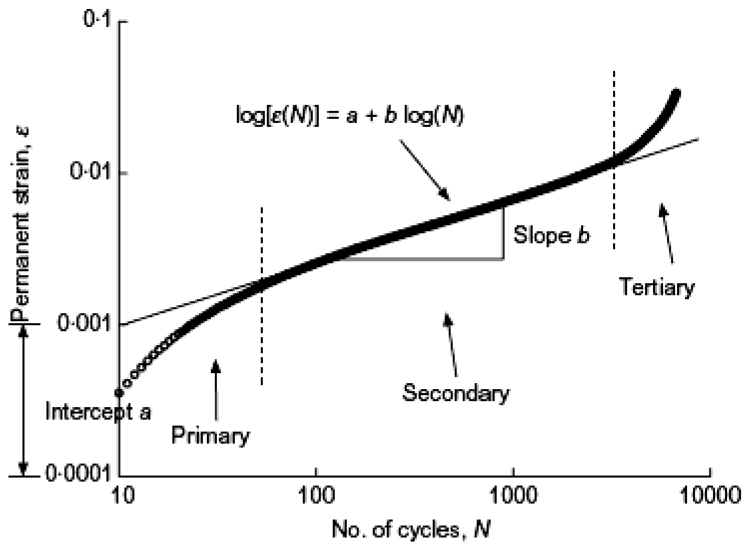

Figure 2.5 Permanent strain vs. number of cycles in log–log scale (Kanitpong et. al., 2006)

2.6 Types of Rutting

There are four basic causes of rutting which can develop on flexible pavements. These are consolidation, surface wear, plastic flow, and structural rutting.

2.6.1 Consolidation

Consolidation occurs when there is insufficient compaction during the construction of the pavement. A mix with insufficient density is prone to further compaction under traffic, especially in hot weather and at intersections where the traffic loads are slowly moving or static. Consolidation rutting occurs by forming a depression along the wheel paths without any humps on either side of the depression (Figure 2.6) (Buncher, 1999).

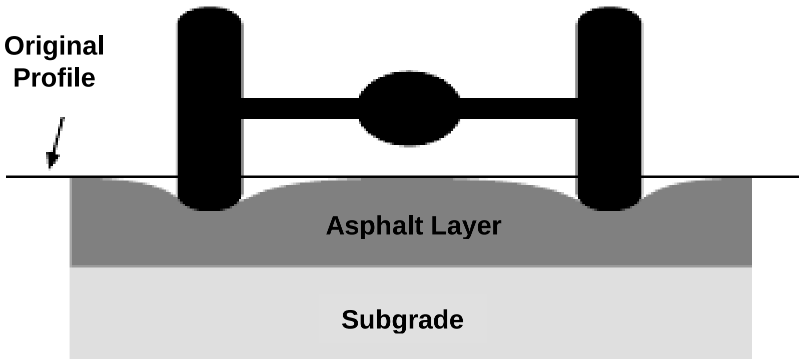

Figure 2.6 Rutting due to consolidation (Superpave Mix Design Superpave Series No. 2 (SP-2), Asphalt Institute)

2.6.2 Surface Wear

Surface wear takes place because of the surface abrasion of flexible pavements by studded tires during the winter session. The subsequent depression on the surface is similar to that caused by a consolidation rutting (Buncher, 1999). The wear rutting takes place because of progressive loss of coated aggregates from the asphalt pavement surface. The aggregate loss occurs, in general, due to the combined effect of environmental factors and traffic loading. The rate of wearing rutting may become higher by using abrasive sands for ice control in the winter time. It may also be accelerated when the pavement surface becomes disintegrable, by which aggregate particles move from their places as the bitumen becomes fragile by the environmental effects (Foo, 1994). Figure 2.7 shows a typical surface wear rutting in asphalt concrete without any deterioration in the underlying layers.

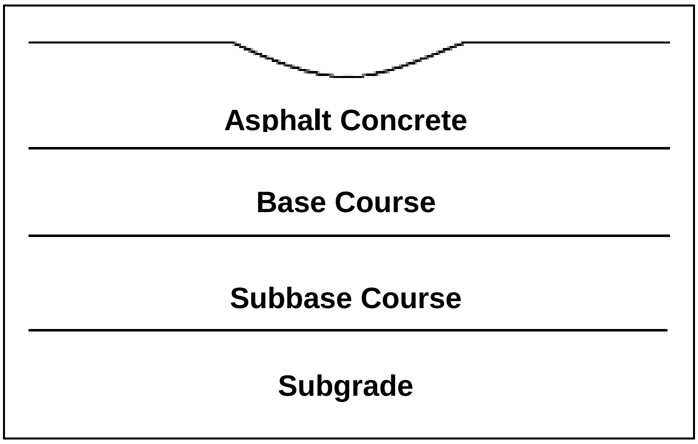

Figure 2.7 Rutting due to surface wearing (Ozturk, 2007)

2.6.3 Plastic Flow

Plastic flow results when there is insufficient stability in the asphalt pavement. Some of the more common reasons for mix instability are high asphalt content hence insufficient air voids, excessive amount of rounded aggregate and high percent of mineral filler passing No.200. The plastic flow occurs usually in the surface mix rather than the lower lifts of the pavement. Intersections are especially prone to this type of rutting due to vehicles either slowly moving or standing on the road surface. Plastic flow rutting will normally appear as longitudinal ruts with humps on either side of the depressions. The humps are created as the material is squeezed out from underneath the vehicle tires as depicted in Figure 2.8 (Buncher, 1999).

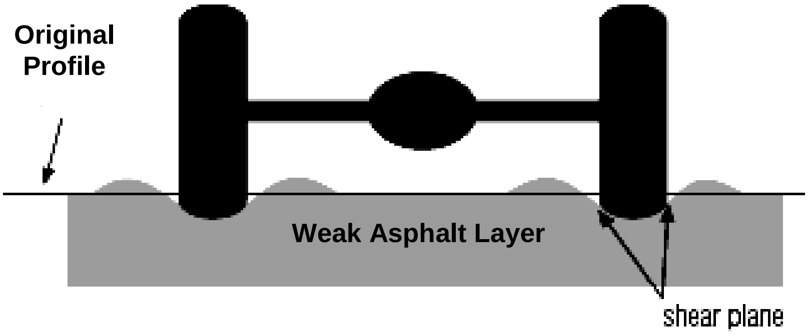

Figure 2.8 Rutting due to plastic flow (Superpave Mix Design Superpave Series No. 2 (SP-2), Asphalt Institute)

2.6.4 Structural Rutting

Structural rutting occurs due to vertical deformation of the pavement structure under repeated traffic loads. Permanent deformations can occur in one or more layers of the pavement structure. Surface cracks may occur in this type of rutting (Foo, 1994). A schematic of structural rutting is given in Figure 2.9.

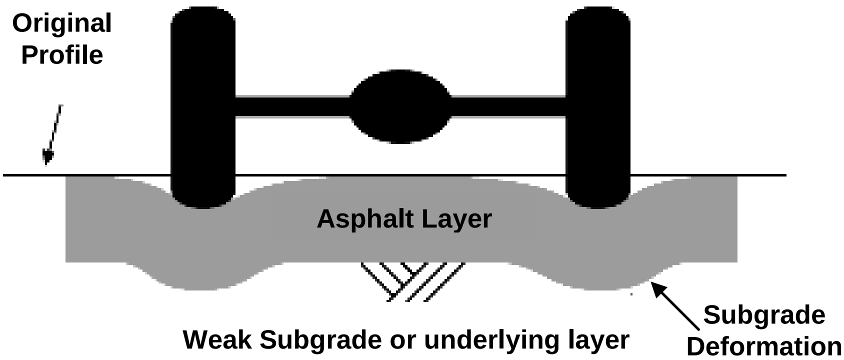

Figure 2.9 Structural rutting (Superpave Mix Design Superpave Series No. 2 (SP-2), Asphalt Institute)

2.7 Effect of Specimen Size on Permanent Deformation

The Superpave models team of the Arizona State University (Witczak et al., 1999) conducted a comprehensive study to determine the specimen size effect on various performance tests for asphalt concrete specimens prepared using the Superpave Gyratory Compactor. The analysis of permanent deformation tests results indicated that a minimum height to diameter ratio of 1.5 is required to offset the specimen size effects. The findings also showed that the test specimen should be large enough compared to the maximum size of individual aggregates. In a similar work, Quintus et al., (1994) recommended that the minimum test specimen diameter be 2 times the nominal aggregate size for field cores and 4 times the nominal aggregate size for cored laboratory specimens.

Figure 2.10 shows the effect of specimen diameter on permanent deformation at 2000 load cycles. It can be seen that as the specimen diameter increases, the strain at 2000 cycles decreases up to a minimum point, after which the strain is increased with an increase in the specimen diameter. Similarly, Figure 2.11 shows flow number vs. diameter relationship for the same specimen. Based on the comparison with Figure 2.10, it can be noted that an inverse relationship exists for the flow number in relation to the specimen diameter. The flow number increases up to a peak cycle and starts decreasing subsequently.

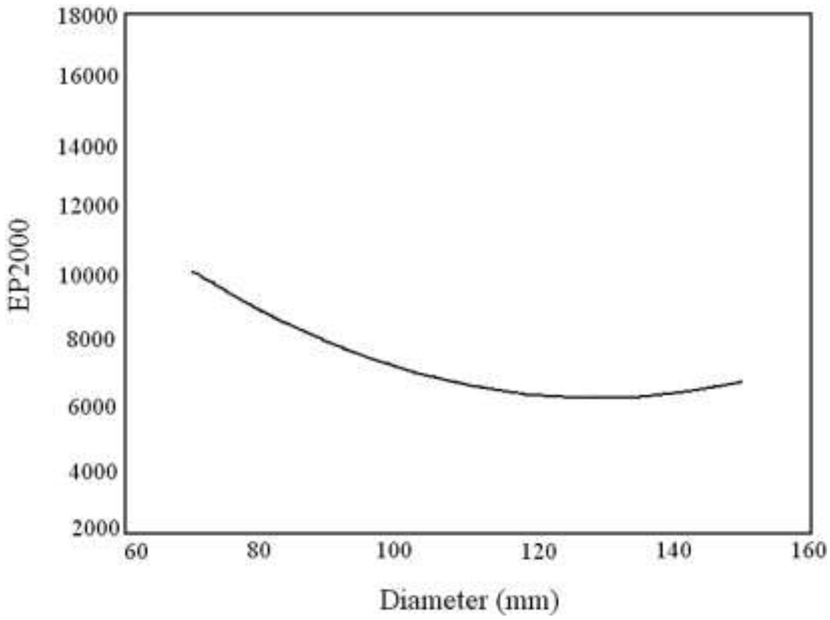

Figure 2.10 Change in permanent strain at 2000 cycles with specimen diameter (Witczak et al., 1999)

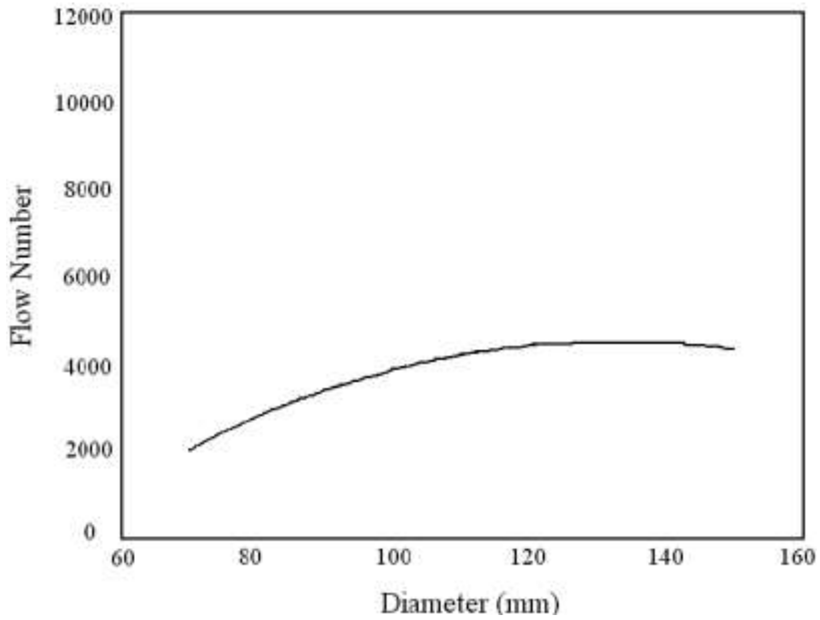

Figure 2.11 Effect of specimen diameter on flow number (Witczak et al., 1999)

These data indicated minimum of 100 mm diameter specimen to characterize both the secondary zone and the onset of tertiary flow in the permanent deformation tests. Figure 2.12 and Figure 2.13 show the effect of height to diameter ratio on the measured strain and the flow number, respectively. The onset of tertiary flow was found to be a critical parameter requiring a minimum height to diameter ratio of 1.5 (Witczak et al., 1999). As shown in Figure 2.12, there seems to be a negligible effect of height to diameter ratio on the permanent strain measured at 2000 cycles. On the other hand, as shown in Figure 2.13, there is a notable decrease in the flow number with an increase in H/D ratio, however, after the H/D ratio of 1.5, the flow number decreases at relatively a lower rate for the increasing H/D ratios.

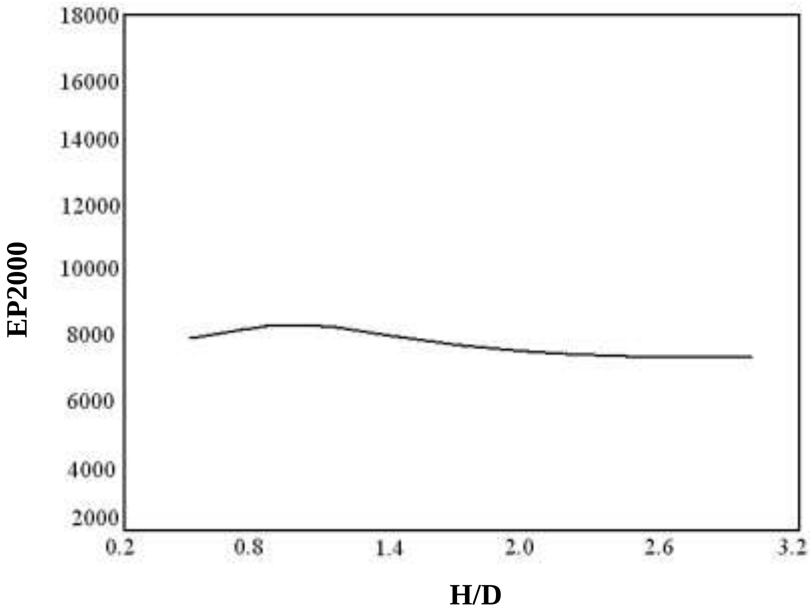

Figure 2.12 Change in permanent strain at 2000 cycles with H/D ratio (Witczak et al., 1999)

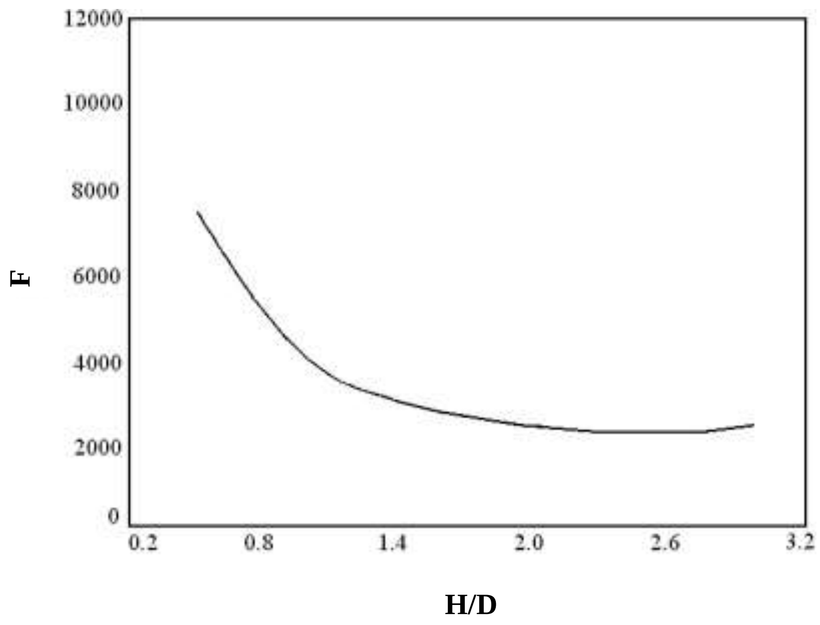

Figure 2.13 H/D ratio vs. flow number relationship (Witczak et al., 1999)

The study concluded that the specimen aspect ratio has a significant effect on the material properties measured in the uniaxial tests. A minimum of 100 mm diameter specimen with a height of 150 mm is required to yield material properties that are not affected by the specimen size or the maximum aggregate diameter. In both graphical and statistical analysis for permanent deformation tests, a minimum height to diameter ratio of 1.5 is necessary in order to accurately characterize the permanent deformation of specimens (Witczak et al., 1999).

2.8 Use of Recycled Materials in Asphalt Concrete

The material that was previously used in the construction of buildings and temporary works and then was re-used as a construction material without reprocessing is called reclaimed material. Reclaimed materials are mainly re-used in their original form even if they are cut to different sizes, cleaned up and refinished in any construction project (Salvo, 1995). Recycled material use is in fact an effort to preserve both materials and energy in any region, for which a variety of recycling plans have been approved by several public agencies. It is usually familiar to observe garbage collection bins spread all over the cities, wherein wastes are divided into different categories. In many developed countries an organized system is there for complete recycling programs to face the challenges related to reclaiming glass, aluminum cans, and newspapers. A recycled material must have some distinctive benefits of its use for its effective utilization in HMA. In fact, a vital feature of such a study is to consider the probability of potential liability when new, unusual materials are used in the asphalt mix production process. Nevertheless, it should be made sure that the design constraints and mix properties cannot be compromised to have room for waste materials. The reason for this consideration is the fact that an excellent mix performance is a product of good mix properties, also life cycle cost analysis of such a mix will need a considerable amount of time before the economic feasibility, and production processes for using waste materials in HMA can be completely assessed.

Two major issues should be considered for incorporating recycled material into hot mix asphalt. One issue is of “cost”; there needs to be an equilibrium between disposals of waste material through usual ways and its incorporation into the hot mix asphalt. The second matter is the effect of recycled material on the performance of HMA. It would not be appropriate to incorporate a waste material that considerably increases the cost of the HMA and at the same time shortens the service life, or increases the maintenance costs (Waller, 1993).

Several waste materials are a product of manufacturing process, service industries, sewage treatment plants, households and mining. In recent years, quite a lot of states have taken interest either to authorize the use of some waste materials or to examine the viability of such practice. The hot mix asphalt industry has been encouraged in recent years to add a wide range of waste materials into HMA pavements. This has raised some reasonable economic, engineering and environmental concerns. The waste materials include mining waste, industrial wastes, municipal/domestic wastes, and roofing shingles (Kandhal, 1993). Some of the materials having potential to be used as a recycled material are discussed in detail as follows;

2.8.1 Aggregates from Recycled Portland Cement Concrete

The breaking and crushing of Portland cement concrete results in the production of Recycled Portland Cement Concrete (RPCC). These crushed RPCC aggregates are then added to asphalt mixes prepared for road construction. According to Forster (1997), the cost and state requirements are the major external factors that bound the use of RPCC in any project. RPCC usually have a 10–15% lower specific gravity than natural aggregates. Hawkins (1996) studied the factors such as absorption, soundness, specific gravity, and gradation of aggregates, which limit the use of RPCC.

Due to lower density of recycled material, the costs of transportation are reduced, which is relatively a reasonable option than hauling natural aggregates (Wilburn and Goonan, 1998). Portland cement binder has a higher water absorption and lower specific gravity than the natural aggregates. Though RPCC is most often used as a fill material, however, soundness, solubility, and groundwater contamination remains a concern (Forster, 1997).

Before using RPCC as an aggregate in the concrete, the vital properties of the material used, which should be given importance, are angularity, specific gravity, gradation, absorption, freeze–thaw distress mechanism, compressive and flexural strength. The specific gravity and absorption of the RPCC are critical to the mix design and batching of the novel concrete. Specific gravity is a prime determinant of the batch weights that will be transformed to the same volumes in the concrete. The RPCCs higher water absorption rates requires controlling the water in the RPCC aggregate during the stockpiling and batching procedure so that the same quantity of water is there all over the concrete mix. Processed RPCC is very angular and has a rough exterior texture due to its mortar bond. These traits give processed coarse RPCC concretes positive mechanical properties such as good abrasion resistance and good bearing strength due to which it is able to effectively resist weathering and erosion (FHWA, 1993). Angularity and gradation of the RPCC increases the efforts required to compact granular material and may affect the workability of cement concrete (Forster, 1997).

RPCC has a compressive strength lower than non-recycled concretes due to weaker aggregate particles, though the flexural strength might be slightly higher due to stronger physical and chemical adhesion between the aggregate and the Portland cement binder (Forster, 1997). The FHWA carry out investigations on the types of mix or design alterations that are required for pavement mixes prepared with recycled concrete aggregate.

When using RPCC as a base, it is essential to consider its gradation, angularity, soundness, and solubility. Drainable bases need a dissimilar gradation than dense bases since drainable base gradations may require additional treatment of fines waste to stop its blockage (Robinson et al., 2004). As the base maintains the structure for a roadway, the soundness of the compacted RPCC must be precisely distinguished in order to make certain that RPCC-containing base fulfils the load-bearing necessities of the pavement structure in the beginning and in the future. The angularity of RPCC can add to the efforts desired to compact the granular material to dense base specifications. Some RPCC can dissolve in the water passing through the pavement structure. This dissolved material will increase the pH of the groundwater and may perhaps influence vegetation within the surrounding area of the road. When this water having dissolved concrete, comes across the external air, the carbon dioxide in the atmosphere will precipitate out calcium carbonate, which can possibly block a drainage system (Forster, 1997).

2.8.2 Construction Debris as a Recycled Aggregate

A lot of Federal and state highway contracts state the use of recycled materials in the construction of highways. The abundantly available potential replacements for natural aggregates in urban areas are Reclaimed Asphalt Pavement (RAP) and Reclaimed Portland Cement Concrete (RPCC). Recycling of construction debris not only helps in extending the life of natural resources but also lowers the waste discarding load on landfill regions (Wilburn and Goonan, 1998). Recycling and reprocessing of reclaimed asphalt concrete into new asphalt pavement aggregates, aggregate base and subbase for roads, and granular fills has now become very common. Approximately 20–30% of aggregate is used in road construction and repair works while only 10–15% is used to manufacture new asphalt pavements (FHWA, 1997)

The demolishment of road beds, bridge supports, buildings, airport runways, and curbing leads to their breaking up and crushing into recycled fill material, concrete aggregate, and aggregate base material. Recycled aggregate from RAP and RPCC competes in the construction market with natural aggregates. According to Robinson et al. (2004), the usage rate of recycled aggregate is influenced by its availability, engineering performance, and by financial and other marketplace inducements that support the use of RAP and RPCC as recycled aggregate

The engineering and quality specifications should be met by the aggregates, which are defined by standardized tests (ASTM, 2002, 2003; Barksdale, 2000). The high-quality source rocks and gravels that meet these specifications, as usually plentiful, are restricted in many regions (Langer and Knepper, 1995). Aggregate is a high-bulk, low unit-value, high place-value mineral commodity whose cost to the end user is highly affected by the price of transporting processed aggregate from the production site to the construction site (Bates et al., 1998).

The nonstop and rising demand for aggregate in many urban and developing areas (Robinson and Brown, 2002) produces a market for improved aggregate supply. The complexity of developing and allowing new sites of natural aggregate production increases transportation costs for natural aggregate to numerous construction sites (Robinson, 2004). This increasing transportation cost for natural aggregates and the increasing costs and decreasing availability of landfill options to dispose of construction and other wastes creates an economic incentive to market recycled aggregate materials reclaimed from local sources of construction debris (Wilburn and Goonan, 1998). The amount of construction-related debris that is potentially recyclable as aggregate is unknown, but presumably large. Aggregate resources are used mainly in proportion to the recognized transportation of a region (Robinson and Brown, 2002).

Reclaimed asphalt pavement and reclaimed Portland cement concrete are the easily obtainable of the potential substitutes for natural aggregates in urban areas. Trends in making and utilization of recycled aggregate are hard to verify because the volume and nature of debris that is generated throughout construction and demolition activities and the methods used to dispose of or reclaim this material are not tracked or checked by local state agencies. Wilburn and Goonan (1998) predicted that the production rate of recycled aggregate is less than 5% of annual production of natural aggregate. However, production on a tonnage basis is huge.

The Federal Highway Administration (FHWA, 1993) estimates that about 91 million metric tones of asphalt pavements are reclaimed every year, and that 80% of the reclaimed quantity is recycled into highway construction projects. The supply of Portland cement concrete debris that is produced by construction activities each year is not clearly identified. Approximations range from 26 million metric tones in 1992 (including an estimated 3 million metric tones of concrete pavement—(FHWA, 1993) to 100 million metric tones (Brown, 1997).

2.8.3 Recycled Solid Waste Materials in Asphalt Concrete

The huge amount of waste material collected all over the world is creating expensive disposal problems. These waste materials are either routine usage by consumers or by-product of industrial production processing. These waste materials can have harmful effects if their dumping pollutes the ground water used by common people. The primary aim of all the research work done on the solid waste material usage as a recycled material is to find out the techniques for processing the waste materials, to learn ways of accommodating waste materials in the hot mix asphalt, their influence on mix properties and performance, and to find out the consequential increase in the cost of the end product (Waller, 1993).

Other potentially good options of recycled materials to add in the hot mix asphalt pavements are steel slag, tires, and plastics. The selected recycled materials are subjected to the same requirements for property classification and testing as done in case of virgin aggregates (BSEN13043, 2002). It is the responsibility of the pavement engineers to define the categories for aggregates properties related to their specific uses. Selected requirements for aggregates in surface layers asphalt are shown in Table 2.1.

Table 2.1 Property requirements and test methods for aggregates in surface layers asphalt (PD6682-2, 2003)

Property Category	Test Method	Property Requirements
Geometric	BS EN933	Grading, fine content, flakiness index
Physical and mechanical	BS EN1097	Resistance to fragmentation, Polished Stone Value (PSV), Aggregate Abrasion Value (AAV)
Chemical	BS EN1744	Leaching
Thermal and weathering	BS EN1367	Water absorption, magnesium sulphate value

The selection of a type for the surface layers has to consider a huge number of issues together with traffic, climate, condition of existing surface, and economics. The toughest and the most expensive materials are used in pavement top layers of the road structures in order to endure tire and weather conditions. The characteristics they exhibit are essential to vehicles' protection and riding quality, these characteristics include strength, friction, noise and the capability to drain off surface water. The asphalt performance strongly depends on the kind of mixture used, with the exception of the nature of the binder and the aggregates. Not all the desired properties can be found altogether in any particular mix type, as in usual cases one property is improved at the expense of the other. Hence, the preference of one type over the other becomes hard when one property has to be picked at the cost of another.

The addition of steel slag in the asphalt mix can enhance the skid resistance and the stability of the asphalt mix. It can also be used as an alternate to the coarse aggregates due to its roughly textured surface, hardness and the angular outline. The aging vulnerability, the resistance to permanent deformation and the moisture susceptibility performance of asphalt containing slag aggregates improves the friction, texture, and resistance to rutting (Wu et al., 2007).

For the asphalt mixes containing recycled glass, the manufacturing equipment and paving methods designed for conventional asphalts mixes can also be used. Usually a hydrated lime about 2% is added as the anti-strip agent to maintain the stripping resistance of the pavement. In asphalt mixes if a higher quantity and larger size of the glass is used then it might cause a number of problems such as inadequate friction and bonding strength, and is considered more appropriate for use in lower courses of the pavement. The asphalt pavements having 10–15% crushed glasses in wearing course mixes have displayed reasonable performance (Su and Chen, 2002; FHWA, 1997; Airey et al., 2004; Maupin, 1998).

Plastic wastes can be significantly used as a fractional substitute of aggregates in asphalt concrete mixes to reduce the dead load of structures. In the tests carried out by Justo and Veeraragavan (2002), recycled low-density polyethylene replacing 15% aggregates in asphalt mix resulted in improved water and rutting resistance.

The recycled rubber can also be used in asphalt concrete mixes as a fractional substitute of virgin aggregates (McQuillen et al., 1998). The advantages of adding recycled rubber to the asphalt mix includes improved skid resistance under icy circumstances, enhanced flexibility and crack resistance, and reduced traffic noise. Many investigators have described the use of scrap tire/rubber in cement mortar and concrete, featuring the research on the use of scrap tire/rubber in concrete (Siddique et al., 2007). Tire rubber can be used in asphalt mixes through two discrete manners as follows. The crumb rubber can be dissolved in bitumen as a binder modifier, this is referred to as the ‘wet process’ and the modified binder from this process is termed as ‘asphalt rubber’. The second method is termed as the ‘dry process’, in which a portion of fine aggregates is substituted with the ground rubber to produce an asphalt mix termed as ‘rubberized asphalt’. According to FHWA (1997), the asphalt mix prepared through the ‘wet process’ contains rubber particles which reduces its resistance against permanent deformation. But the observations in Brazil and India where totally different, as the asphalt rubber mixture had lower rutting potential due to higher stiffness and tensile strength at high temperatures (Bertollo et al., 2004; Palit et al., 2004). According to Reyes et al. (2005) and Selim et al. (2005), some laboratory test results of "dry process" showed a reduced permanent deformation. Khalid and Artamendi (2006) elaborates that university of Liverpool had the allowable rubber content set at 10% of binder, as a result of which resistance to rutting was improved.

2.8.4 Effect of Recycled Concrete Aggregate on Permanent Deformation

In pavement engineering, permanent deformation of asphalt concrete is of great importance. In the asphalt concrete pavements, the pavement response is judged by the development of distresses such as rutting. In the initial stages of a flexible pavement, the high rate of permanent deformation can be due to a number of reasons elaborated as follows. In the tests performed by Estakhri (1994), the reclaimed asphalt concrete was mixed together with bitumen emulsions and conventional mixtures. The recycled mixes with Hveem stability values greater than 24 resisted rutting successfully, while mixes with stability values lower than 12 failed due to permanent deformation. In the field conditions, at first higher permanent deformation rates were noticed on hot remixed roads as well as on hot remixed test sections subjected to accelerated pavement testing (Potter and Mercer, 1997).

2.8.4.1 Influence of Recycled Concrete Aggregate on Asphalt Concrete Properties

The presence of quality aggregates in the demolished buildings makes them a useful source for recycling as these aggregates can be re-used as helpful construction materials. The crushing process of the demolished concrete structures can become a helpful source of Recycled Concrete Aggregates (RCA), which was initially used only as a fill material. But later it was also accommodated in the road subbases to enhance their structural strength and resistance against rutting, threatened by the heavy traffic. The use of RCA in curbs, footpaths, parking areas etc. resulted from the investigations carried out by researchers such as McGrath (2001), Arm (2001), Guthrie and Mallett (1995), Roos and Zlich (1998) and Huang et al. (2002).

The presence of highly porous cement paste attached to the recycled concrete aggregate make them different from the fresh aggregates. Even after carrying out recycling process this paste remains on the face of the natural aggregates. This characteristic causes RCA to display relatively higher permeability and water absorption as compared to the virgin aggregates. Researchers such as Ravindrarajah (1996), Shayan et al. (1997), Gomez-Sobern (2002) and Zaharieva et al. (2002) concluded that RCA has a lot of variations in its source quality due to the existence of poor density materials and contaminations such as glass, rubber, cement, bricks, tiles and other flexible or brittle waste materials.

During the crushing process of RCA the mortar is detached from some of the aggregates, thus increasing the chances of stripping. In wet and dry states, RCA shows a lot of variation in strength. The air voids in the specimens accommodating RCA are usually found to be relatively higher than in control specimens. But the characteristics such as film thickness, bulk density, mineral aggregate voids, and voids filled with binder in the control mix specimens are higher than that in the RCA. The densities of both asphalt control mix and RCA enhances considerably with the addition of fine mineral aggregates. Apart from air voids all the volumetric properties and the creep values of the asphalt samples with fresh aggregates are relatively superior to that of parallel specimens having RCA as coarse aggregates.

The feeble cement mortar attached onto RCA particles is shattered by the mechanical mixing and compaction causing considerable alteration in the particle size distribution of RCA. The stiffness of the RCA asphalt mixtures is reduced by the increase in the binder content. In addition, the RCA asphalt mixes have lesser stiffness values in contrast to that of control mix. The reason for this could be the addition of low strength mortar and RCA to the mix.

In the mixes accommodating RCA the creep value decreases with the improvement in compaction but increases with higher binder content. At 50◦C the mixes containing RCA behaves similar to that of conventional mixes in regards to creep distress. The minimum creep value is higher in control mixes than in mixes containing RCA. For the same bitumen content, the film thickness values for RCA mixes are lower than for control mixes (Paranavithana and Mohajerani, 2004).

CHAPTER 3

MATERIALS AND METHODS

3.1 Introduction

The objective of this chapter is to discuss the materials and tests involved in this study. The procedure for modified Marshal mix design is presented along with materials selection criteria, sample preparation techniques, and testing methods. In the experimental design section, different parameters and their levels used for standard Marshall and modified Marshall specimens are mentioned. The selected gradation for aggregates, their blending, and mixing with selected asphalt contents at specific temperatures are described. The compaction process, the standard Marshall compactor modification method, the Rice test and bulk specific gravity test procedures are briefly described (Section 3.2). Before proceeding to the next phase, it was necessary to find out the number of blows for the modified Marshall specimens for which the energy per unit area must be the same as in the standard Marshall specimens. The criterion for selection of number of blows for modified Marshall specimens is explained in Chapter 4.

In the second phase (Section 3.3), the modified Marshall mix design procedure was used to prepare specimens with Recycled Cement Concrete (RCC). The specimens prepared at the Optimum Asphalt Contents (OAC) were then tested under the repeated creep rutting test to determine rutting potentials in relation to various experimental parameters. The type of recycled materials used, their gradations, aggregate crushing, and sample preparation processes are also explained. The section dealing with the experimental design for the repeated creep tests elaborates the various parameters used, and their justifications followed by the explanation of rutting program interface and the test equipment.

3.2 Modified Marshal Mix Design Procedure

As mentioned in Chapter 2, the standard Marshall test specimens were modified by increasing the specimen height so as to minimize the specimen size affects during the rutting tests which require at least an aspect ratio of 1.5. A minimum specimen diameter of 100 mm and a height of 150 mm are suitable options for the unconfined uniaxial repeated creep test to accurately characterize the permanent deformation of asphalt mixes both in the secondary zone and the onset of tertiary flow. Specimens with an aspect ratio of at least 1.5 also provide material properties that are not influenced by the maximum aggregate size (Witczak et al., 1999).

The limestone crushed aggregates were selected along with a 50-70 grade binder for preparing the mix samples. The aggregates were oven dried at 100ºC for 24 hours and were crushed by a jaw crusher to obtain the required gradation. The samples were blended, mixed, and compacted in accordance with the standard as well as the modified Marshall mix design methods. The mix of each combination in terms of gradation and asphalt content was compacted using a standard Marshall compactor device. The bulk specific gravity and the maximum theoretical density (Rice test) tests were performed on the compacted specimens. The energy per unit volume calculation of the specimens was then carried out in order to estimate a design number of blows for the modified Marshall specimens.

3.2.1 Materials Selected for Modified Marshall Mix Design Procedure

The selected bitumen was of grade 50-70, and its physical properties such as specific gravity, penetration and softening point were determined in the laboratory. A specific gravity (ASTM D70-03) of 1.024 g/cm^3, penetration (ASTM D5-05a) of 52.20 and a softening point (ASTM D36-06) of 48.40°C was measured according to the specified specifications.

Calcareous (composed of limestone) aggregates were selected for this study. The use of ASTM C127-80 specification for coarse aggregates lead to the results of 0.396 % absorption, 2.701 g/cm^3 saturated surface dry bulk specific gravity (G_{sb}), 2.72 g/cm^3 apparent specific gravity (G_{sa}) and a bulk specific gravity (G_{sb}) of 2.691 g/cm^3. In accordance with ASTM C128-79 specifications, for fine aggregates a bulk specific gravity (G_{sb}) of 2.601 g/cm^3, an apparent specific gravity (G_{sa}) of 2.733 g/cm3 and an absorption of 1.85 % were achieved after conducting laboratory tests.

3.2.2 Experimental Design

A total of 44 test specimens (see Table 3.1) were prepared by using the methodology as specified in ASTM D1559-92. The design variables selected are gradation and asphalt content at two levels. Each specimen was replicated with two specimens to reduce the amount of variability in the test results. While the gradation limits were determined according to the ASTM D3515, the asphalt contents were selected based on the experiences with the binder grade and the aggregate type used. The details of the number of blows applied to specimens of each combination are shown in Table 3.2. For the standard Marshal specimens, 50 blows were applied on each side of the test specimens, while a series of number of blows were used to determine the relationships between the degree of compaction and the air void or the energy per unit volume of the test specimens.

Table 3.1 Experimental design for standard and modified Marshall specimens

Design Variable	**Variable Level**			
Aggregate Type	Limestone (Crushed Aggregates)			
Asphalt Grade	50-70			
Asphalt Content (%)	5.2 (High)		4.7 (Low)	
Gradation	Coarse		Fine	
Sample Type	Standard	Modified	Standard	Modified
Compacted Specimens x Replicates	2 x 2	7 x 2	2 x 2	7 x 2
Weight of Compacted Specimen	1200 g	3200 g	1200 g	3200 g
Loose Samples No. of Replicates	2	2	2	2
Weight of Loose Samples	1500 g	1500 g	1500 g	1500 g
Sample Size (Diameter x Height)	100mm x 62mm	100mm x 152mm	100mm x 62mm	100mm x 152mm
Number of Blows	50	50,75,100, 150,175	50	50,75,100, 150, 175
Total Number of Specimens	44			

Table 3.2 Number of blows applied for each specimen

Type	Gradation	Specimen Weight (gms)	Asphalt (%)	Number of Blows
Modified Marshall Mix Design	Fine	3200	5.2	50
				75
				100
				150
			4.7	75
				100
				150
	Coarse	3200	5.2	75
				100
				150
			4.7	75
				100
				150
				175
Standard Marshal Mix Design	Fine	1200	5.2	50
			4.7	
	Coarse	1200	5.2	
			4.7	
Maximum Theoretical Density	Fine	1500	5.2	N/A
			4.7	
	Coarse	1500	5.2	
			4.7	

3.2.3 Specimen Preparation

A dense graded aggregate blend was chosen for the mix samples according to the ASTM D3515 gradation limits. As shown in Figure 3.1, the selected gradations (coarse and fine) close to the upper and lower limits of the specification limits. The gradation closer to the upper ASTM curve is termed as fine gradation and the gradation closer to the lower ASTM curve is termed as coarse gradation.

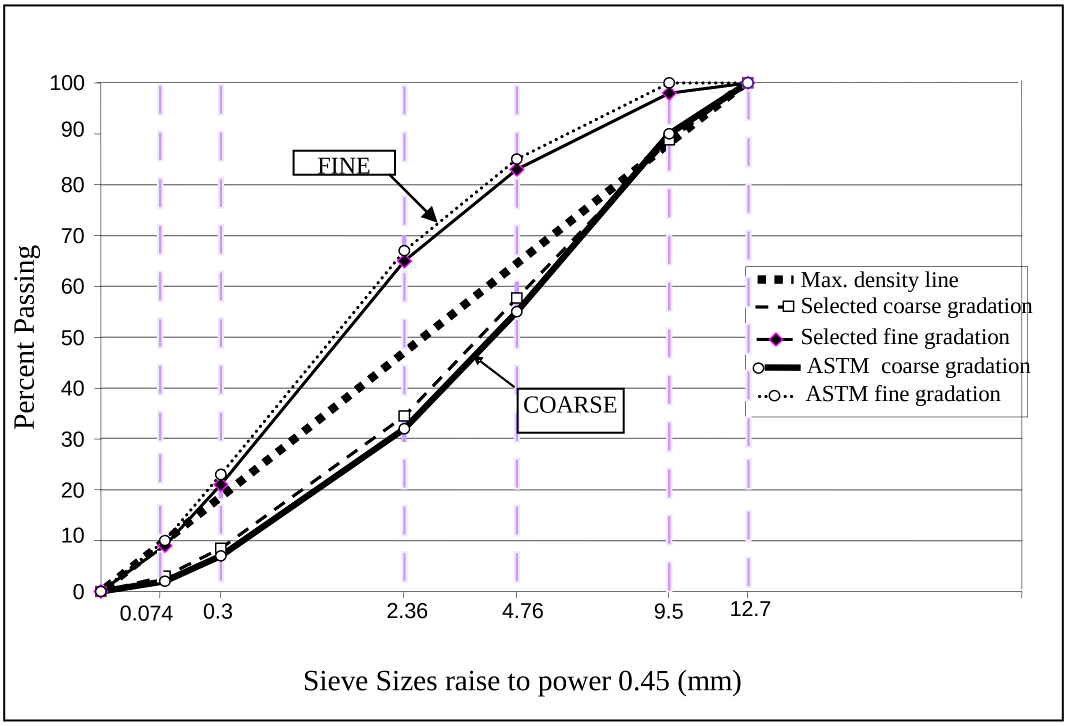

Figure 3.1 Gradations selected in accordance with ASTM D3515

Table 3.3 Selected coarse and fine gradations

Sieve Size	ASTM D3515		Selected Gradations	
	Minimum Limit (%)	Maximum Limit (%)	Coarse Gradation (%)	Fine Gradation (%)
12.7 mm	100	100	100	100
9.5 mm	90	100	91.5	98
4.76 mm	55	85	57.7	83
2.36 mm	32	67	34.7	45
0.3 mm	7	23	8.5	21
0.074 mm	2	10	3	8

The selected gradations are shown in Table 3.3 and final mix designs for both standard and modified Marshall specimens are given in Table 3.4. The standard Marshall coarse graded specimens were prepared in two categories, i.e. coarse gradation with high asphalt content (CH) and coarse gradation with low asphalt content (CL). In the naming convention, the first alphabet denotes gradation (coarse or fine) while the second represents asphalt content level (high or low) to identify different specimens.

Table 3.4 Mix design used for the standard and modified Marshall specimens

<table>
<tr><th>Sieve Size</th><th>Coarse Gradation (%)</th><th>Fine Gradation (%)</th><th colspan="2">Asphalt Content (%)</th></tr>
<tr><td>12.7 mm</td><td>100</td><td>100</td><td rowspan="6">5.2</td><td rowspan="6">4.7</td></tr>
<tr><td>9.5 mm</td><td>91.5</td><td>98</td></tr>
<tr><td>4.76 mm</td><td>57.7</td><td>83</td></tr>
<tr><td>2.36 mm</td><td>34.7</td><td>45</td></tr>
<tr><td>0.3 mm</td><td>8.5</td><td>21</td></tr>
<tr><td>0.074 mm</td><td>3</td><td>8</td></tr>
</table>

Similarly, the standard Marshall fine graded specimens were also prepared as fine gradation with high asphalt content (FH) and fine gradation with low asphalt content (FL). The total weight of both coarse and fine graded standard Marshall specimens was 1100 g. without the asphalt concrete.

The modified Marshall coarse graded specimens were prepared as coarse gradation with high asphalt content (CH) and coarse gradation with low asphalt content (CL). The total weight of aggregate for the coarse graded modified Marshall specimens with high AC (%) was 3040 g and with low AC (%) was 3056 g without the addition of asphalt cement. The total weight of mix for both coarse and fine specimen is 3200 g. The modified Marshall fine graded specimens were prepared as fine gradation with high asphalt content (FH) and fine gradation with low asphalt content (FL). The total weight of both coarse and fine graded modified Marshall specimens was the same as for the coarse specimens.

The aggregates were sieved to achieve the desired gradations. Samples were also prepared for the Rice tests for the calculation of maximum specific gravities of the selected mixes.

The standard Marshall mix design procedures together with a list of miscellaneous equipments were used in the preparation of the test specimens: A small mixing trowel, a large spatula and a scoop for batching aggregates. Oven and hot plate, for heating the mixture and equipment, compaction pedestal and a flat bottom metal pans for heating aggregates, containers were used. Gill-type tins, beakers, pouring pots, or saucepans for heating asphalt, round metal pans and mechanical mixture for mixing asphalt and aggregates, a balance sensitive to 1 g were used. Marking crayons for identifying specimens, compaction mould, compaction hammer, mould holder, paper disks, steel specimen, gloves for handling hot materials and equipment, electric thermometers with digital read out are recommended. In addition, a holder for specimen mold, mold cylinders for test specimen and a test specimen extruder are requirements for performing Marshall tests.

The aggregates were dried to a constant weight at 105 to 110°C and then sieved to the required fractions. After blending, each sample blend was placed in the oven for 3 hours at 150°C while bitumen was heated for two hours at the same temperature. After heating, the required quantity of bitumen was added to the aggregate blend and mixed with the help of a mixer in a container as shown in Figure 3.2.

Figure 3.2 Mixer used for mixing bitumen with blended aggregates

During the mixing process, a spatula was used for removing the asphalt particles sticking at the corners and sides of the container in order to make sure that no fine particles were lost during the mixing process. The speed of the mixer should be adjusted in such a way that it is neither too slow causing the mix to cool down nor so fast that its movement may result in throwing of asphalt particles out of the container. The different stages of the mixing procedure are shown in Figure 3.3.

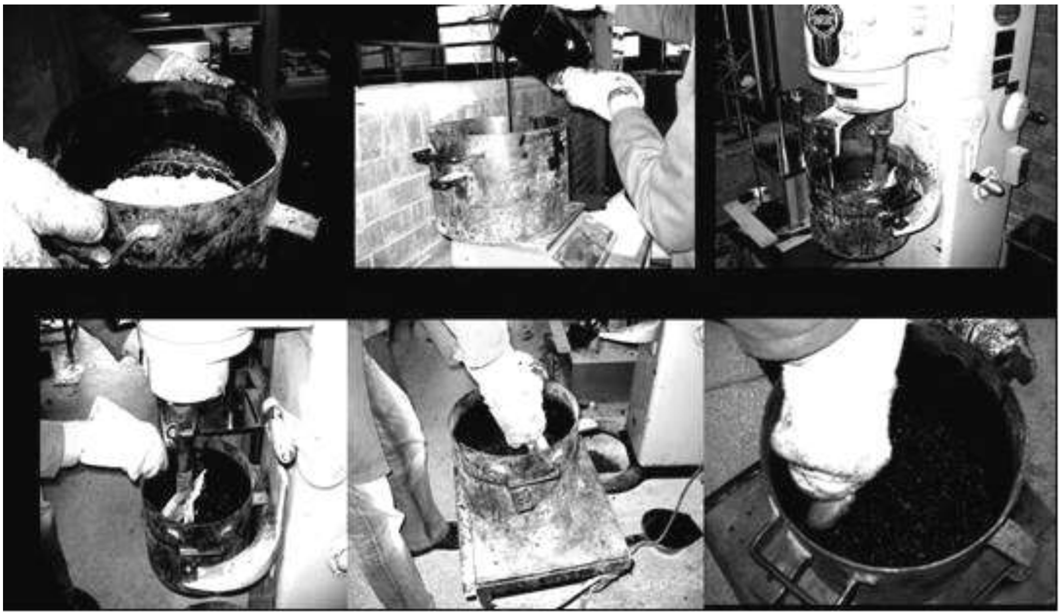

Figure 3.3 Mixing of asphalt and aggregate before compaction

After mixing the blended aggregates with bitumen, asphalt was placed in a tray, which was then heated in an oven for two hours at a temperature of 140°C. Meanwhile, the Marshall moulds were also heated to a temperature of 140°C. Attention was given to make sure that the temperature remains constant throughout the heating time. The moulds were then placed on the pedestal of the compactor. The specimen was taken out and poured in the mould. A filter paper was placed in the mould before and after pouring the asphalt mix. The collar was removed and the surface of the mix was smoothened with a trowel. The collar was placed again, and the mould assembly was placed on a compaction pedestal in the mould holder. The required number of blows was applied. The mould was overturned and the same number of blows was applied again. Figure 3.4 shows the compactor used for the standard Marshall specimens.

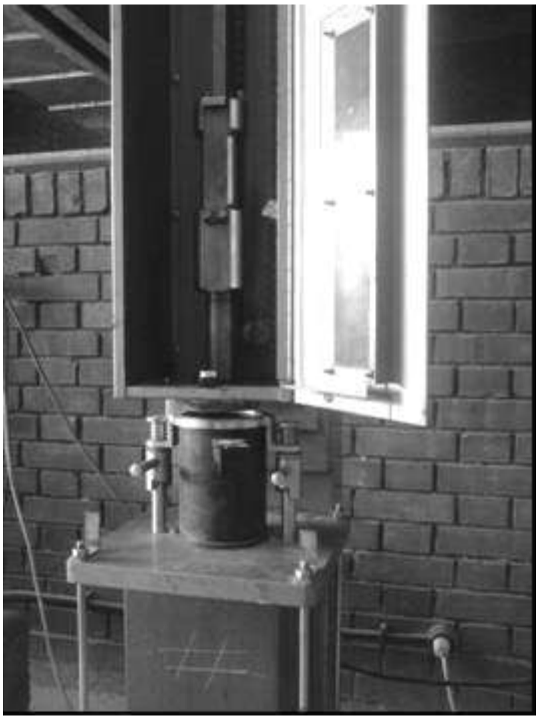

Figure 3.4 A picture of a standard Marshall compactor

3.2.4 Compaction

The standard Marshall compactor was modified to build the modified Marshall compactor by removing the pedestal of the compactor and replacing it with a shorter pedestal. The difference in the height of old and new pedestal was actually equal to the difference in the height of the standard and the modified Marshall moulds so that after compaction the desired height can be achieved for the modified Marshall specimens. Other parameters such as the hammer weight, the mould diameter, the height of drop were kept constant. The difference in heights between the standard and the modified Marshall moulds is shown in Figure 3.5.

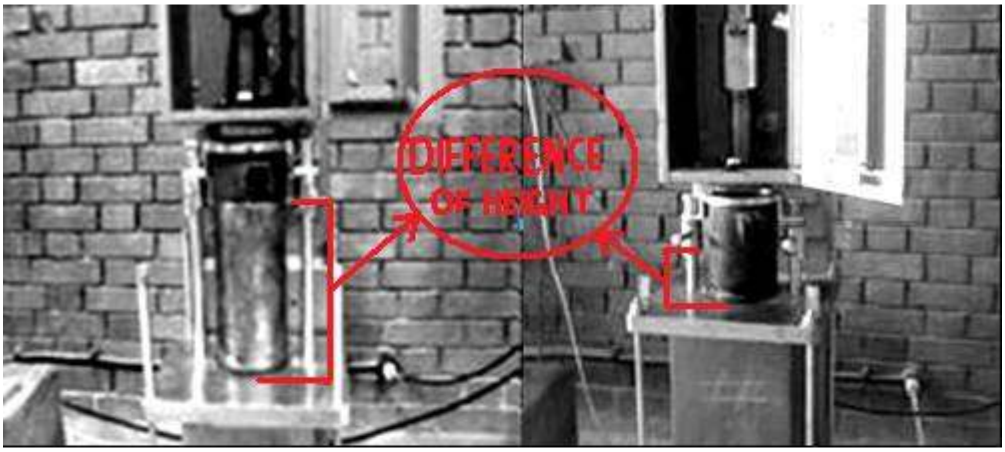

Figure 3.5 Difference in standard and modified Marshall specimens

As shown in Figure 3.6, the modified Marshall specimens were prepared in the same manner as those of standard Marshall specimens, the only difference is the increase in the height of modified Marshall specimens.

Figure 3.6 Typical steps in preparation of a modified Marshall specimen

Few plates of different thicknesses were manufactured to use in the compaction so that a constant height of compaction for modified Marshall specimens can be achieved. The plates are shown in Figure 3.7.

Figure 3.7 Plates used to maintain constant height in modified Marshall specimens

A fixed number of blows of 50 were used for the standard Marshall specimens while a series of different number of blows were used for the modified Marshall specimens.

After compaction, the standard Marshall specimens were extracted by using a hydraulic jack as shown in Figure 3.6 and left at room temperature for 24 hours to cool down. To prevent specimen breakage, specimens should be allowed to cool down inside the mould until a sufficient cohesion is developed to maintain a cylindrical shape after the extraction process. After compaction, the height of standard specimens should be within 62.2 to 64.8 mm and modified specimens 165 to 175 mm. If the desired specimens heights are not achieved then an adjustment is made to the mix components for the next compaction.

3.2.5 Rice Tests and Bulk Specific Gravity Measurements

Specimen bulk specific gravities were measured in accordance with AASHTO T 166, Method A. After extraction from the mould, the specimens were left at room temperature to cool down. The measured dry mass of the specimens was marked as A. The specimens were placed in water at 25ºC for approximately 3 to 5 minutes and weight in water as C. The specimens were taken out from water and surface dried by blotting with a wet cloth and their surface-dry masses (B) were recorded. The bulk specific gravities were then calculated by using the following formula:

$$Bulk\ Specific\ Gravity = \frac{A}{(B-C)} \tag{3.1}$$

The maximum specific gravity of the samples were measured in accordance with AASHTO T 209 by using the flask method. The test procedures for the flask method in Figure 3.8 were carried out in the following manner: After the sample was thoroughly mixed in the mixer, it was spread over a paper sheet with the help of a spatula by separating the aggregate particles from each other without fracturing them. The sample was left to cool down at room temperature. The mass of the dry flask was measured as X. The sample was placed in the flask. The sum of the masses of the flask, the cover, and the sample was recorded as Y. The mass of the sample was calculated by subtracting the mass X from the mass Y and recorded as A. Sufficient water was then added to the flask at nearly 25±1ºC to cover the sample about 2.54 cm. The lid was then placed on the flask and the system was connected to the vacuum line. The flask was subjected to a partial vacuum of 27.5±2.5 mm Hg residual pressure at about 15±2 minutes.

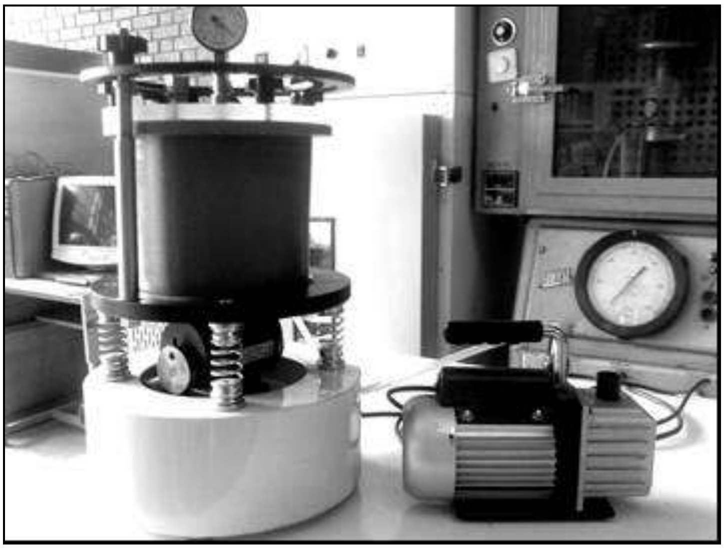

Figure 3.8 A picture of Rice test measurement system

The flask and the whole assembly were agitated through a mechanical device in order to remove the entrapped air from the mix sample. After the agitation time was over, the release valve was opened slowly followed by the removal of vacuum pump and the lid. The flask was filled with water and then placed in a 25±1ºC water bath where it was allowed to stand for 10±1 minutes. The flask was filled with water, and the cover was placed on the flask after eliminating all the air from it. The excess water was wiped off from the flask and the cover. The mass of the flask, the cover, the desired water, and the sample was recorded to the nearest 0.1 g. This mass was designated as E. The mass of the flask and water was determined as D during the calibration of the flask procedure. The maximum theoretical specific gravity (G_{mm}) was calculated to three decimal places as follows:

$$G_{mm} = \frac{A}{(A+D-E)} R \tag{3.2}$$

Where R is the correction factor for the density of water and used only if the test temperature is different from 25ºC. The procedure of flask method can be seen in Figure 3.9.

Figure 3.9 Process of Rice test for asphalt mix samples

3.3 Repeated Creep Testing of Specimens with Recycled Cement Concrete

The objective of this section is to investigate the rutting potential of HMA prepared with different percentages of RCC. The approach to determine the permanent deformation parameters of mix specimens is to use repeated dynamic load test and record the cumulative permanent deformation over the test duration. Typically, the test is carried out at a temperature of 50°C under unconfined stress conditions ranging from 10, 20, and 35 psi. A haversine pulse load of 0.1 seconds and 0.9 second rest time are applied for the test duration. All the test specimens were subjected to 10,000 cycles repeated loading which takes nearly 3 hours. However, the entire test duration may be smaller if the specimen fails or goes into the tertiary flow stage earlier (Witczak et al., 1999).

Initially, the mix design was performed by preparing modified Marshall specimens with different percentages of RCC to determine the Optimum Asphalt Content (OAC). After finding out the OAC, another set of modified Marshall specimens were prepared at OAC to use for the rutting tests. The specimens were then cut using a jaw crusher from both sides in order to achieve a smooth end surfaces and a desired height of 150 mm. After cutting the samples, a greased membrane was placed on both sides of the samples and then placed in the heating chamber for performing the repeated creep tests. After completing the creep test, the required readings were recorded.

The recycled materials selected for this study was obtained from the discarded cement concrete specimens dumped in the materials laboratory of the Civil Engineering, Department of Middle East Technical University, Ankara.

As mentioned earlier, the specimen preparation procedure for modified Marshall mix design is not different from the standard Marshall specimen procedure. The only difference between the two was in their respective heights. The rest of all the parameters were kept constant. For the rutting tests, RCC materials were added to the modified Marshall specimens with percentages of 0%, 25%, 50%, and 75%. The OAC was measured at 4% as Voids in Total Mix (VTM). The OAC was used in the preparation of new set of specimens. The bulk specific gravity and Rice tests were performed on these specimens with air voids level at 4±0.5%.

3.3.1 Materials Selected for Repeated Creep Test

The RCC obtained from the discarded cement concrete specimens and the crushed limestone aggregates were used. This selection was made because RCC aggregates are angular in shape and have a rough surface texture by their mortar adhesion. These qualities give the coarse RCC favorable mechanical properties as good abrasion resistance and bearing strength, which allows them to effectively resist weathering and erosion (FHWA, 1993). The flexural strength of RCC is quite high due to stronger physical and chemical bonding between the aggregates and the cement binder. In addition, the long-term, continuous, and increasing demand of aggregates in urban and developing areas (Robinson and Brown, 2002) creates a market for increased RCC aggregate supply from demolished old buildings. Thus, RCC can be considered as the most abundant and available substitutes for natural aggregate in urban areas.

After obtaining the discarded specimens, they were broken into small pieces with the help of a hammer and then further broken into even smaller pieces by putting them in a jaw crusher as shown in Figure 3.10. The gap between the jaws of the crusher was fixed based on the size to which the aggregate was desired to be broken. The recycled aggregates were then placed in an oven at a temperature of 105 to 110°C. This was done to completely dry the aggregates, as there was a possibility of presence of moisture in the aggregates particles that might have been added after rain when they were placed in the dumps.

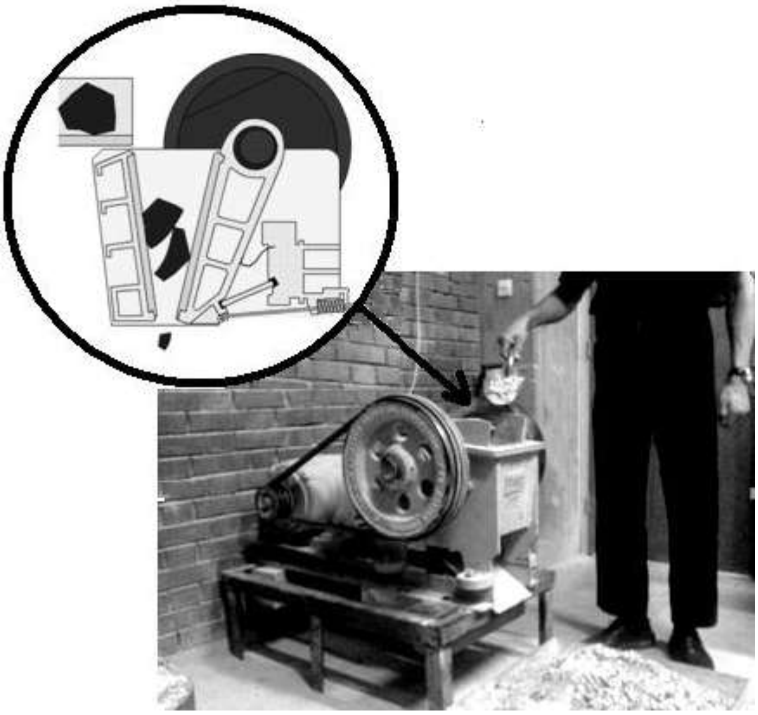

Figure 3.10 The crushing of recycled cement concrete aggregates with a jaw crusher

After the aggregates were dried, they were further broken into fine particles with a jaw crusher and sieved to achieve the desired gradations. The recycled aggregates for each combination were then blended with the crushed limestone aggregates in the desired percentages. These specimens were then mixed after addition of asphalt and compacted with the desired number of blows.

3.3.2 Experimental Design for Repeated Creep Rutting Test

In this section, the experimental design shown in Table 3.5 for the repeated creep tests is discussed.

Table 3.5 Experimental design for repeated creep tests

<table>
<tr><th colspan="2">Design Variable</th><th colspan="4">Variable Level</th></tr>
<tr><td colspan="2">Gradation</td><td colspan="2">Coarse</td><td colspan="2">Fine</td></tr>
<tr><td colspan="2">Number of Compacted Samples</td><td colspan="2">24</td><td colspan="2">24</td></tr>
<tr><td colspan="2">Number of Loose Samples</td><td colspan="2">24</td><td colspan="2">24</td></tr>
<tr><td rowspan="10">Recycled Content added to HMA</td><td rowspan="5">Trial Asphalt Contents Selected</td><td>RCC</td><td>AC (%)</td><td>RCC</td><td>AC (%)</td></tr>
<tr><td>0%</td><td>4, 5, 6</td><td>0%</td><td>4, 5, 6</td></tr>
<tr><td>25%</td><td>4, 5, 6</td><td>25%</td><td>4, 5, 6</td></tr>
<tr><td>50%</td><td>4, 5, 6</td><td>50%</td><td>5, 6, 7</td></tr>
<tr><td>75%</td><td>4, 5, 6</td><td>75%</td><td>6, 7, 8</td></tr>
<tr><td rowspan="5">Optimum Asphalt Contents Computed</td><td>RCC</td><td>AC (%)</td><td>RCC</td><td>AC (%)</td></tr>
<tr><td>0%</td><td>4.9</td><td>0%</td><td>4.4</td></tr>
<tr><td>25%</td><td>5.2</td><td>25%</td><td>4.8</td></tr>
<tr><td>50%</td><td>5.5</td><td>50%</td><td>5.8</td></tr>
<tr><td>75%</td><td>6.0</td><td>75%</td><td>6.8</td></tr>
</table>

The aggregate types of crushed limestone and RCC aggregates were used, but this time a different gradation as shown in Table 3.7 was selected. The specimens were prepared by modified Marshall mix design procedure with a diameter of 100 mm and a height of 152 mm. A fixed number of blows of 140 were used for all the specimens. The weight of the compaction hammer was 4.5 kg with a 45 mm height of drop. Each specimen weighed approximately 3200g, and 50-70 grade bitumen was used. The three trial asphalt contents (AC %) were selected.

3.3.3 Definition of Test Parameters

The five parameters that were selected to be measured as the outcome of the repeated creep tests are shown in Table 3.6.

Table 3.6 Parameters used for repeated creep test

Serial Number	Parameters
1	Flow number
2	Permanent strain
3	Slope
4	Intercept
5	$\varepsilon_p/\varepsilon_r$ @ 1000 pulse

The resulting values of each parameter are compared with RCC% and their relationship is shown through a graph in Chapter 5.

The slope represents the rate of change in the permanent strain as a function of the change in loading cycles (N). Figure 3.11 shows the graphical relation between the permanent strain and number of cycles of fine graded specimen with 75% RCC content. A log-log scale has been used for this graph. The curve is divided into three stages, the primary, secondary, and tertiary stages. A straight line is drawn through the secondary portion of the curve where the R^2 value is the highest. The slope of this straight line is the required parameter ‘b’ whose value is calculated by the formula:

$$m = \frac{\Delta y}{\Delta x} \qquad (3.3)$$

A high slope value of any mix means that the mix has high potential for rutting and vice versa.

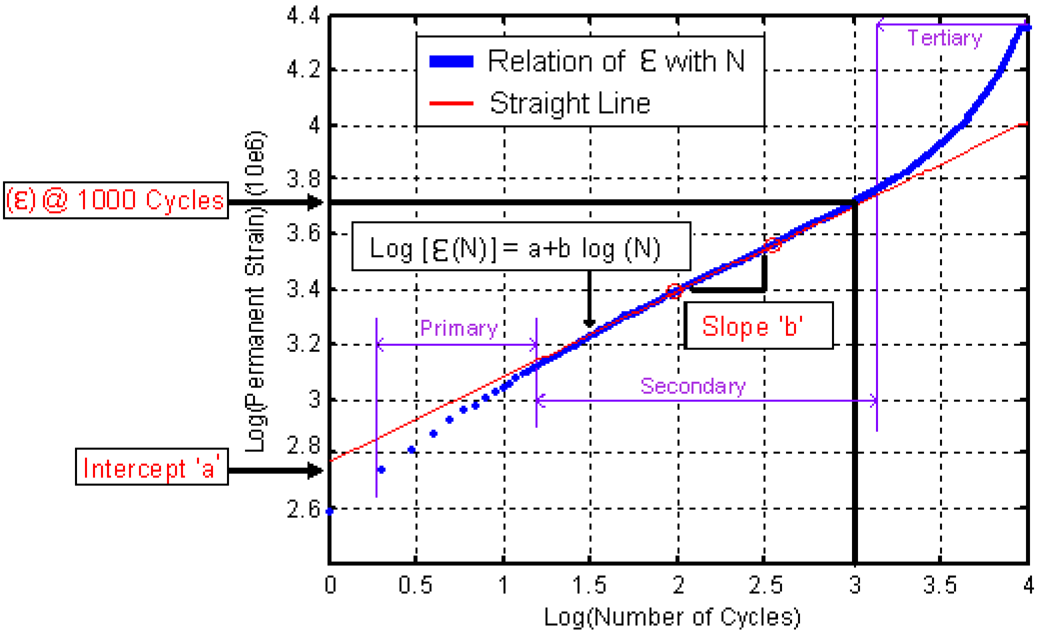

Figure 3.11 Calculation of slope, intercept and (ε) @ 1000 cycles for fine graded tall specimen with 75 % RCC content

The point where the straight line touches the y-axis is the intercept 'a'. The intercept represents the permanent strain at N=1 where N is the number of load cycles. The higher the value of intercept, the larger the strain and hence the larger the potential for permanent deformation, as mentioned in the Superpave study carried out by Witczak et al. (1999).

In the log-log scale, the value 3 along the x-axis represents 1000 number of cycles (N). The number 1000 cycles is selected for the calculation of the permanent strain for all the specimens in order to have a constant value for comparing the results of different specimens. The parameter ε_p / ε_r @1000 pulses is actually the ratio of ε_p (plastic strain) and ε_r (resilient strain) at a load cycle (N) of 1000. This ratio can be used as guidance for mixture performance evaluation and is usually a function of the test temperature, the stress level applied and the mix parameters. The higher this ratio, the more susceptible the mix is to rutting. If the permanent strain value of any asphalt mix is high than that mix has less resistance to rutting.

The starting point at which the tertiary flow associated with pure plastic shear deformation occurs is referred to as the “flow number” for this test. In order to calculate the flow number, a derivative of permanent strain is carried out with respect to the number of cycles, d (ε) / d (N), and is plotted against the number of cycles. The minimum value of d (ε) / d (N) is considered the flow number of the specimen. Figure 3.12 shows calculation of flow number of fine graded specimen with 75 % RCC content. The flow number in this case was calculated to be 1896.

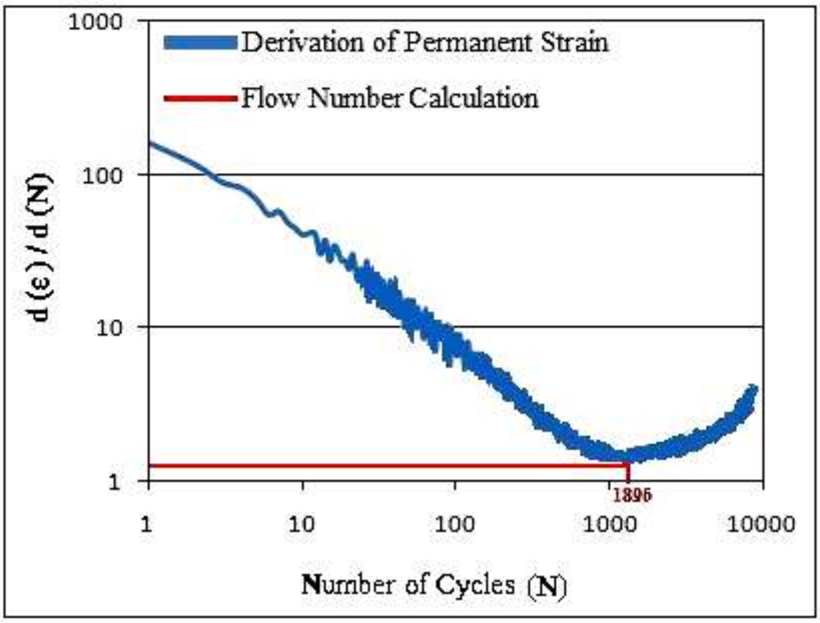

Figure 3.12 Calculation of flow number for fine graded specimen with 75 % RCC content

High flow number of a mix indicates that better resistance of that mix against rutting and vice versa.

3.3.4 Specimen Preparation

The gradation selected for the specimens of the repeated creep tests was taken on the basis of gradation commonly used by the General Directorate of Highways (2006) in Ankara, Turkey. A dense graded aggregate blend was chosen from the General Directorate of Highways (2006) which is shown in Figure 3.13 and Table 3.7.

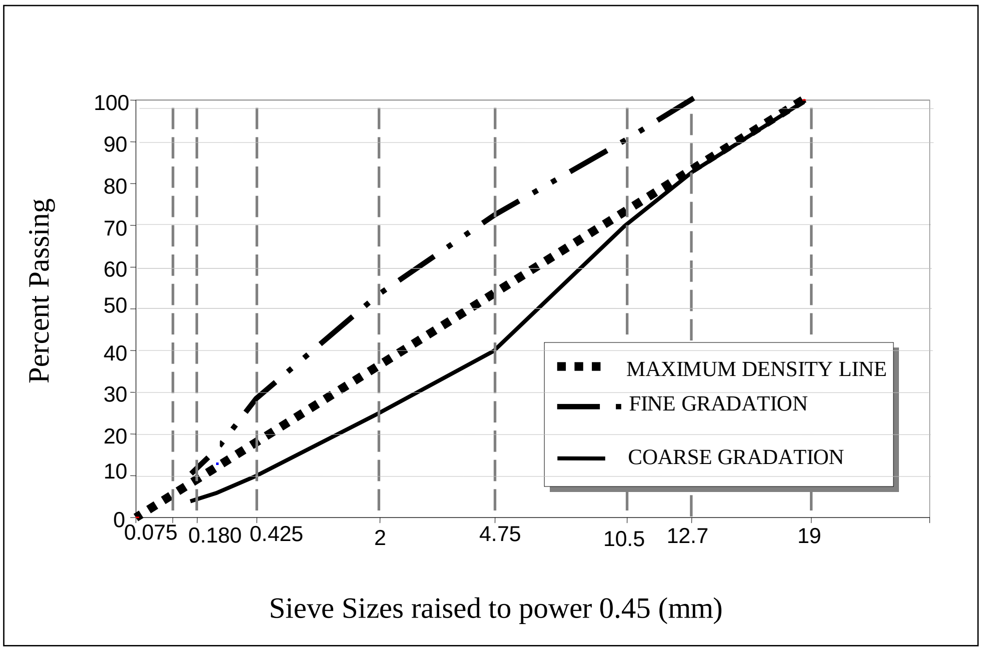

Figure 3.13 Gradation limits according to the General Directorate of Highways (2006)

Table 3.7 Gradation for repeated creep test

Sieve Size (mm)	19	12.5	9.5	4.75	2	0.425	0.180	0.075
Minimum % Passing (Coarse Gradation)	100	83	70	40	25	10	6	4
Maximum % Passing (Fine Gradation)	100	100	90	72	53	28	16	10

The experimental design is carried out in two steps: In the first step, there were a total of 2 gradations (coarse and fine), 4 combinations of crushed and recycled material, i.e. 0:100 (recycled: crushed), 25:75, 50:50 and 75:25 respectively, 3 assumed (trial) asphalt contents (to investigate optimum asphalt content) so the total numbers of samples became 24. Also, 24 loose samples were prepared for finding G_{mm} of the corresponding combinations through Rice tests. Thus, the total number of samples for step 1 became 48. The results of three replicates for each combination helped in investigating the correct optimum asphalt content (OAC) for the corresponding combination. The detailed results are given in Chapter 4. In this chapter, only elaboration of the procedure for finding out optimum asphalt content, and their application to the specimens prepared for rutting test is given.

3.3.5 Procedure to Determine Optimum Asphalt Content

The selection of OAC is also a very critical factor in this study. Generally, the amount of asphalts content used in a highway pavement varies from 4 % to 7 % by weight of the mix. This amount depends upon the type of gradation, position of the layer in the pavement (asphalt wearing course or asphalt base course) and the amount of traffic. It is also a well known that the fine graded specimens generally requires more asphalt content then the coarse graded specimens, as the absorption of fine graded specimens is higher than that of coarse graded specimens. After going through conventional methods of mix design in order to arrive at OAC, a narrower range of optimum asphalt content was investigated based on the past tests performed using the same material. Accordingly, it was decided that for coarse gradation the trial AC % should be 4, 5 and 6 % and for fine gradation an AC % ranging from 4 to 6 and 8 % respectively, depending upon the type of mix.

After performing bulk specific gravity and Rice test of the specimens, the trial asphalt contents for each combination were plotted against their corresponding air voids, the example of which is shown in Figure 3.14.

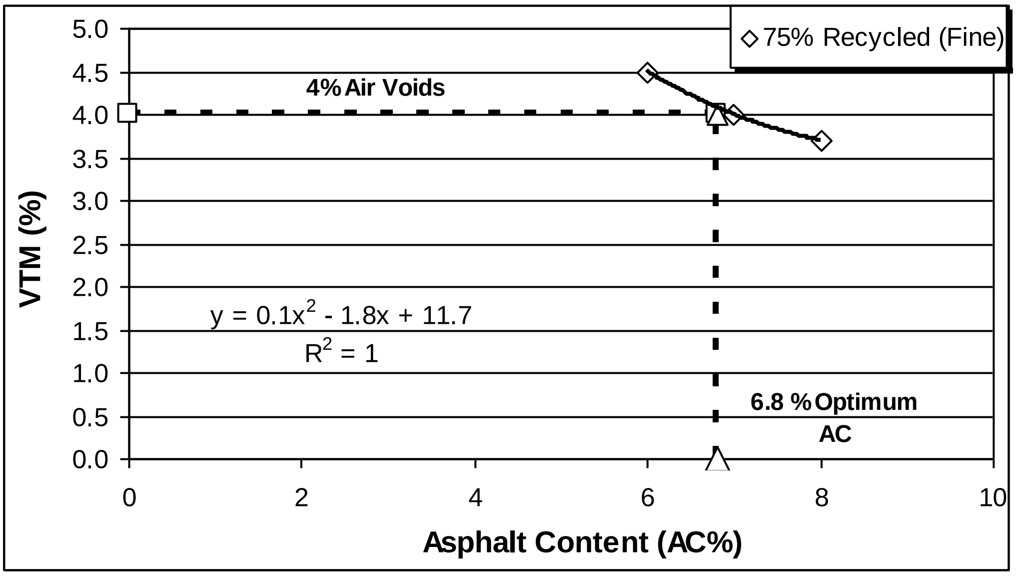

Figure 3.14 Optimum AC% calculation of a specimen with 75 % RCC

As an example to the procedure, for fine graded specimen with addition of 75 % RCC as shown in Table 3.8, the three trial ACs (%) were selected as 6%, 7% and 8%. Three samples were prepared using these AC% contents, respectively, of which the bulk specific gravity and the maximum specific gravity values were measured in order to calculate the VTM of the corresponding samples. The results obtained were plotted in the form of a graph, as shown in Figure 3.14. The equation of the curve was used to find the optimum AC% corresponding to 4 % air voids. In this case, the OAC% value obtained was 6.8%. The graphs of the rest of the samples are shown in the Appendix D.

In the second step, the measured OAC was used in preparing a new set of specimens, where the following variable parameters were used: Two gradations (Coarse and fine) and 4 different recycled cement concrete content aggregate at 0:100 (recycled: crushed), 25:75, 50:50 and 75:25. A total of three replicates were used for each combination. In total, there were 24 compacted specimens along with 24 loose samples.

Thus, 24 samples from step 2 were compacted and tested through UMMATA (Universal Materials Testing Apparatus) and the rest of 24 loose samples were prepared for maximum specific gravity test. The tabular format of all the samples was same. The details of the modified Marshall fine graded specimens are shown in Table 3.8 and that of modified Marshall coarse graded specimens in Table 3.9 The details of the rest of the modified Marshall specimens and their corresponding samples for maximum specific gravity test are given in Appendix C.

Table 3.8 Fine graded specimens for repeated creep test

Sieve (mm)	Cumulative Retained (%)	Weight in Grams for Modified Marshall Test Specimens (FINE)							
		Combination 1		Combination 2		Combination 3		Combination 4	
		Natural 100 %	Recycled 0%	Natural 75 %	Recycled 25 %	Natural 50 %	Recycled 50 %	Natural 25 %	Recycled 75 %
9.500	0	0	0	0	0	0	0	0	0
4.750	30	857	0	640	213	422	422	626	209
2.000	15	581	0	434	145	286	286	425	142
0.425	15	765	0	571	190	377	377	559	186
0.180	4	367	0	274	91	181	181	268	89
0.075	2	184	0	137	46	90	90	134	45
Pan	4	306	0	228	76	151	151	224	75
Total Weight (gms)		3,059	0	2,284	761	1,507	1,507	2,237	746
AC (%)		4.4		4.8		5.8		6.8	

Table 3.9 Coarse graded specimens for repeated creep test

Sieve No.	Cumulative Retained (%)	Weight in Grams for Modified Marshall Test Specimens							
		Combination 1		Combination 2		Combination 3		Combination 4	
		Natural 100 %	Recycled 0%	Natural 75 %	Recycled 25 %	Natural 50 %	Recycled 50 %	Natural 25 %	Recycled 75 %
19.000	0	0	0	0	0	0	0	0	0
12.500	17	517	0	387	129	257	257	128	384
9.500	13	396	0	296	99	197	197	98	293
4.750	30	913	0	683	228	454	454	226	677
2.000	15	456	0	341	114	227	227	113	338
0.425	15	456	0	341	114	227	227	113	338
0.180	4	122	0	91	30	60	60	30	90
0.075	2	61	0	46	15	30	30	15	45
Pan	4	122	0	91	30	60	60	30	90
Total Weight (gms)		3,043	0	2,276	759	1,512	1,512	752	2,256
AC (%)		4.9		5.2		5.5		6.1	

The mixing and compaction of the specimens were carried out in the same way as mentioned before. The fixed number of blows, which was calculated earlier (140), was used for all the specimens. The Rice test and bulk specific gravity measurements were carried out in the same manner as was done in the previous section. The specimens were then cut from both sides in order to achieve the required height of 15 cm. As shown in Figure 3.15, the surface of the specimen before cutting was rough but after cutting a smooth surface with relatively less friction was obtained.

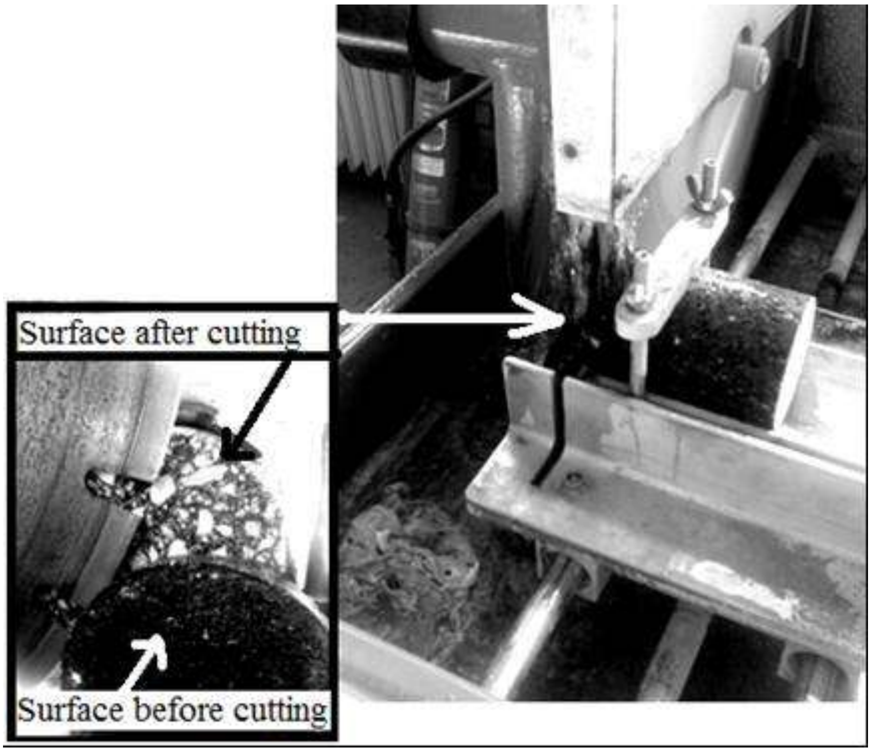

Figure 3.15 Specimen cutting process

3.3.6 Repeated Creep Tests for Rutting Behavior

This section covers preparation, testing, and measurement of permanent deformation of asphalt concrete specimens prepared with varying ratios of RCC and crushed aggregates. The modified Marshall asphalt concrete specimens were subjected to repeated load unconfined uniaxial tests in order to investigate the effects of RCC on the rutting resistance asphalt concrete. A continuous repeated compressive loading is applied at a loading cycle of 1 second in duration, 0.1 sec haversine load with 0.9 sec rest time. During the process of repeated loading, the corresponding permanent strain occurring in the specimen at every instant is continuously recorded. A constant temperature of 50ºC is set for both the core and skin of the specimen.

The purpose of performing repeated load uniaxial test was to determine the rutting performance of the specimens, prepared with different combinations, and to investigate the effects of increase in RCC% on the rutting potential of asphalt concrete mixtures. In this study, Universal Materials and Testing Apparatus (UMATTA) were used to test the modified Marshall test specimens under uniaxial repeated loading.

3.3.6.1 Test Equipment

The introduction of UMATTA System is very important for this study. This system has capability to check, save and display stress, strain, and stiffness data from the specimen under test for extended time and loading cycles. UMATTA comprises a Control and Data Acquisition System (CDAS), an IBM compatible personal computer (PC) and an integrated software package. The CDAS captures and digitizes analogue signals from a range of transducers then transfers this data to the PC for further processing. The mechanical hardware details include the following:

The loading frame has a flat, heavy base plate supported on four leveling screws. Two threaded rods support the crosshead beam and provide height adjustment. The frame is of heavy construction to limit deflection and vibrations that could influence the accuracy of measurements during the test. Loading force is applied through the shaft of a pneumatic actuator mounted in the centre of the cross head, which is closely coupled to an electrical solenoid-operated air valve. A strain gauged force transducer, mounted in line with the loading shaft measures the force applied to the specimen. Low friction displacement transducers with enough sensitivity were used to measure the deflection of the specimen during loading.

The pneumatic system is also a vital part of the test equipment. Vertical force pneumatics is used for the asphalt tests. The system requires clean air supply for operation. A filter regulator assembly is provided to further filter the air to 5 microns and allow manual adjustment of pressure to 750 kPa. Electrical signals applied to this transducer from the CDAS enables computer control adjustment of the air pressure over the range of 0 to 700 kPa.

Two asphalt creep jigs are provided to center the specimens. Each jig comprises a base plate, a top plate and two rods that are screwed into the base plate to mount two transducers. A temperature chamber is used in asphalt testing. The temperature range of the standard unit ranges from 0 to 60ºC.

The UMATTA Control and Data Acquisition System (CDAS) is a compact, self-contained unit that provides all critical control, timing and data acquisition functions for testing frame and transducers. The CDAS is linked to a personal computer. Supervised by a PC, the UMATTA CDAS automatically controls the operation of the loading frame. The CDAS applies a voltage via the Digital to Analogue Converters (D/As) to adjust the air to the required levels. At the time the specimen is subjected to loading forces, the CDAS takes the data from the transducers and shifts it to the PC for processing, display, and storage. The repeated creep test setup can be seen in Figure 3.16.

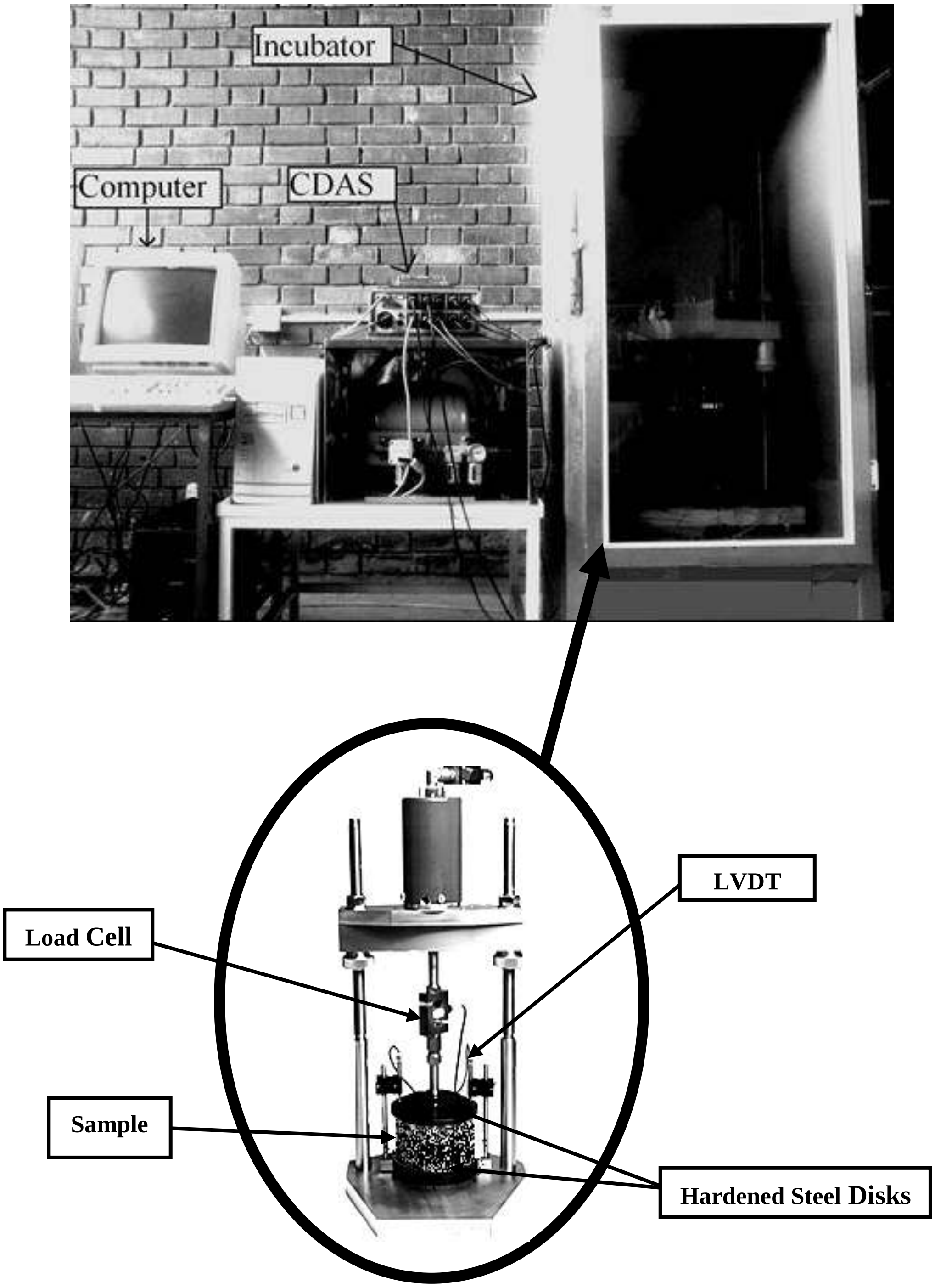

Figure 3.16 Repeated creep test setup

The repeated load uniaxial asphalt creep test is carried out in four steps:

- A time duration of static conditioning load
- A period of pre-load rest without applied stress
- A period of pulse loading
- A period of post-load recovery time without applied stress

3.3.6.2 Selected Parameters for Rutting Test

As shown in Table 3.10, the value of the skin and core temperature of the specimens was kept constant at 50ºC, as it better simulates the pavement temperature in the field. The maximum pulse count limit was kept constant to 10,000 pulses. Mixture properties were varied through the addition of different percentages of RCC to coarse and fine crushed stone aggregate gradations. A number of different values were tried as the applied stress for different specimens.

Table 3.10 Parameters chosen for rutting test

<table>
<tr><th>Parameter</th><th colspan="2">Selected Value</th></tr>
<tr><td>Temperature</td><td colspan="2">50°C</td></tr>
<tr><td>Maximum Pulse Count</td><td colspan="2">10,000</td></tr>
<tr><td>Applied Stress</td><td colspan="2">350 kPa.</td></tr>
<tr><td>Conditioning Stress</td><td colspan="2">35 kPa.</td></tr>
<tr><td>Conditioning Time</td><td colspan="2">5 minutes</td></tr>
<tr><td>Preload Rest Time</td><td colspan="2">1 minute</td></tr>
<tr><td>Recovery Time After Test</td><td colspan="2">15 minutes</td></tr>
<tr><td rowspan="2">Load Cycle Time</td><td rowspan="2">1 Second</td><td>0.1 sec. haversine load</td></tr>
<tr><td>0.9 sec. test period</td></tr>
</table>

Witczak et al. (1999) used a value of 210 kPa, which was also used initially in this study but no deformation occurred hence the load was increased to 350 kPa to bring the deformations to a measurable level. The repeated stress was kept constant to 350 kPa for all specimens. A conditioning stress of 35 kPa was selected because it is usually 1/10 of the full test stress. Conditioning stress was applied for 5 minutes and this assisted with bedding in the platens with the specimen ends. Preload rest time was 1 minute and the recovery time was of 15 minutes. Furthermore, after the test the effects of these factors over the parameters like flow number, slope, intercept, $(\varepsilon_p / \varepsilon_r)_{@\,1000}$ pulses and permanent strain were studied.

In the rutting test, a fixed frequency was used for load application. The procedure calls for a loading cycle of 1.0 second in duration consisting of application of 0.1 second haversine load followed by 0.9 second test period. The test automatically terminates after the application of 10,000 pulse counts if the sample does not fail before the completion of 10,000 load cycles.

The measurement of the permanent and resilient strain of the test specimen corresponding to the number of pulses were continuously recorded and displayed in a graphical form. This strain data is plotted in the logarithmic scale. During the experiments, the parameters that are measured and updated continuously are: the permanent strain, the resilient strain, the resilient modulus and the skin temperature. Tabulated data displayed on the screen include: Test time (hours, minutes, and seconds) for each of the four test stages, loading stage pulse count, both static conditioning and peak loading stresses, specimen permanent strain during each of the four test stages, specimen resilient strain and modulus which is valid only during the pulse loading stage, core and skin temperature of a dummy specimen. The binary file is named with the name of the specimen. The data saved in the binary file can be seen at the end of test; also in case of a power failure it can be recovered. The data file was saved in the ASCII format onto a floppy disk and converted into a MS Excel file for further analysis. A setup definition file, named UMDCREEP.DEF, is used by the software to store the control parameters, options and settings for the next time the program and this test is invoked. If the file does not exist (i.e. first time operation) then default parameters are written into a newly created file in the current directory. Thus, the related detailed information of every experiment is updated with the beginning of every new test.

3.3.6.3 Specimen Setup and Test Procedure

Separate jigs are mounted vertically in the loading frame. The jigs consist of an upper and lower loading platen due to which the load is equally distributed to the ends of the specimen. LVDT displacement transducers are used to measure the vertical axial strain of the specimens. These transducers are calibrated over the range from zero to 5 mm. The transducers are mounted on support rods attached to the axial loading jig base plate, with the probe ends bearing on the upper loading platen surface.

The edit command from the test setup and control menu is used for setting different parameters of a particular specimen in accordance to its properties. The specimen should be uniquely identified by giving it a name on the basis of its properties. e.g. CH1 means a specimen with coarse gradation (C), high asphalt content (H), and is replicate number 1. The first eight characters of the field are used by the software to create a name for both the binary and ASCII files. These numeric fields specify the dimensions of the specimen which affects the calculated test results. The stress to be applied is also specified by the numeric field. Pulse width and repetition period numeric fields define the timing characteristics of the loading pulse. Conditioning, Pre-load Rest and Recovery Times specify the time in minutes for each of the test stages. Set Temperature is an arbitrary numeric field that may be used to indicate the desired specimen testing temperature. Test termination strain defines the maximum strain threshold above which the test will automatically be terminated. Test termination pulse count defines the maximum number of loading pulses to apply to the specimen before automatically terminating the test stage and progressing to the recovery stage. Binary and ASCII Filenames specify the full directory path, filename and extension of the binary and ASCII files generated by the system. The specimen name parameter will automatically alter the filename but not the path or extension. The path will be default to the current drive and directory if not supplied, the extension will default to BIN for binary files, and CSV for ASCII files.

The test setup has a run command and control menu that causes the start of the test sequence. With the appearance of the graphic screen the test starts with a zero strain level, followed by the application of the conditioning stress. The stress is removed at the end of the conditioning time and again the resulting strain is monitored and displayed. The strain resulting with the application of pulse load is checked and displayed on the screen as part of the tabulated data. The strain measured at the conclusion of the pre-load rest stage is now used as the zero reference strain during the remainder of the test sequence.

The full test loading stress is then constantly applied by pulsing the air valve and a graph of the resulting axial permanent and resilient strain of the specimen, against the number of applied loading pulses, is plotted on logarithmic scales on the screen. This phase of the test will carry on until the maximum pulse count is reached, or the operator himself stops the test by pressing F1. After the pulse count is complete or the test is stopped by the operator, the unloaded recovery phase of the test starts and the resulting specimen strain is tabulated in a numeric field on the screen. The test ends after the recovery stage is finished and than Esc key can be used to return to the setup and control menu.

As stated in the previous sections, the physical properties of specimens are recorded with great care since the length and diameter of each specimen significantly affects the calculations of the UMMATA software. The specimens without surface and shape irregularities are used for the tests. Not only the specimen’s dimensions, but also the air voids content of specimens has tremendous affects on the analysis. Figure 3.17 shows a specimen which did not fail after the rutting test, while Figure 3.18 shows a specimen which failed after rutting test.

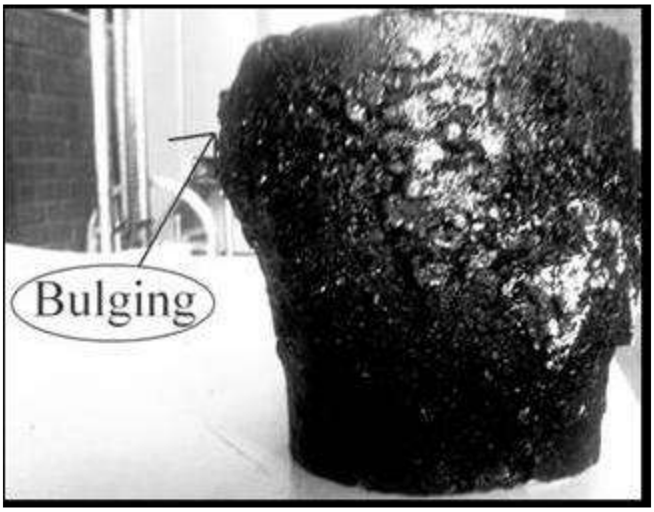

Figure 3.17 Bulging in a test specimen after rutting test

Figure 3.18 No bulging in a test specimen after rutting test (3iv is a number used to differentiate from other specimens)

CHAPTER 4

RESULTS AND DISCUSSION FOR MODIFICATION OF MARSHALL MIX DESIGN

This chapter presents the results of the tests and methods explained in Chapter 3. Initially, the calculation of air voids for the standard Marshall (short) specimens is carried out. Estimation of the design number of blows is carried out by two different methods, and the results are then compared and analyzed. The Voids in Total Mix (VTM) for modified Marshall specimens are calculated through three different methods, and a brief conclusion is drawn from the findings.

4.1 Determination of Measured Air Voids for the Standard and Modified Marshall Specimens

The measured air voids for both the standard and the modified Marshall specimens were calculated as follows; the bulk specific gravity (G_{mb}) values of the specimen were calculated, then the G_{mm} values were measured using the Rice test as explained in Chapter 3. The G_{mb} and G_{mm} values of each specimen were then used to calculate the VTM for the specimens by using the following equation:

$$VTM = 100 \text{ x} \left[1 - \left(\frac{G_{mb}}{G_{mm}}\right)\right] \tag{4.1}$$

The list of the air voids for the standard Marshall specimens is given in Table 4.1 and that of modified Marshall specimens in Table 4.2. In the abbreviation CH501 used in Table 4.1, C stands for coarse gradation, H stands for high asphalt content, 50 is the number of blows, and 1 indicates the first replicate. Similarly, in FL502, F stands for fine graded specimen, L stands for low asphalt content, 50 is the number of blows and 2 indicates the second replicate.

Table 4.1 Measured air voids calculation for standard Marshall (short) specimens

Replicate One						
Mix Type	*Dry Weight (g)*	*Submerged Weight (g)*	*Saturated Surface Dry Weight(g)*	G_{mb}	G_{mm}	*VTM (%)*
CH501	1146.5	661.7	1145.1	2.37	2.47	4.0
CL501	1138.1	656.9	1142.5	2.34	2.49	5.7
FH501	1144.5	667.3	1146.3	2.39	2.45	2.5
FL501	1132.4	655.7	1136.7	2.35	2.45	3.8
Replicate Two						
Mix Type	*Dry Weight (g)*	*Submerged Weight (g)*	*Saturated Surface Dry Weight(g)*	G_{mb}	G_{mm}	*VTM (%)*
CH502	1149.7	660.6	1145.1	2.37	2.47	4.0
CL502	1151.7	665.2	1156.3	2.35	2.49	5.7
FH502	1141.9	667.9	1145.5	2.39	2.45	2.5
FL502	1124.2	643.4	1120.4	2.36	2.45	3.7

Table 4.2 Measured values of air voids for modified Marshall (tall) specimens

Replicate 1					***Replicate 2***				
Mix Type	**Ht. (cm)**	$\mathbf{G_{mb}}$	$\mathbf{G_{mm}}$	**VTM (%)**	**Mix Type**	**Ht. (cm)**	$\mathbf{G_{mb}}$	$\mathbf{G_{mm}}$	**VTM (%)**
CH751	17.3	2.29	2.47	7.3	CH752	17.2	2.30	2.47	7.0
CH1001	17.1	2.30	2.47	6.8	CH1002	17.3	2.31	2.47	6.6
CH1501	16.7	2.35	2.47	4.8	CH1502	16.7	2.36	2.47	4.6
CL751	17.5	2.26	2.49	8.9	CL752	17.4	2.27	2.49	8.7
CL1001	17.3	2.29	2.49	8.0	CL1002	17.1	2.29	2.49	7.8
CL1501	17.1	2.34	2.49	6.0	CL1502	16.7	2.35	2.49	5.5
CL1751	17.4	2.37	2.49	4.7	CL1752	17.3	2.39	2.49	4.0
FH501	17.5	2.27	2.45	7.5	FH502	17.4	2.27	2.45	7.6
FH751	17.6	2.30	2.45	6.0	FH752	17.2	2.31	2.45	6.0
FH1001	16.3	2.35	2.45	4.3	FH1002	16.7	2.34	2.45	4.4
FH1501	16.6	2.41	2.45	1.6	FH1502	16.1	2.41	2.45	1.6
FL751	16.8	2.29	2.45	6.3	FL752	17.2	2.30	2.45	6.1
FL1001	17.2	2.31	2.45	5.7	FL1002	16.8	2.31	2.45	5.4
FL1501	16.6	2.38	2.45	2.9	FL1502	16.9	2.37	2.45	3.0

4.2 Estimation of Design Number of Blows for Modified Marshall Specimens

For the compaction of modified Marshall specimens, a fixed number of blows was required in order to simulate the field conditions where a constant load is applied on the hot mix asphalt pavement during the compaction process. As in the standard Marshall specimen, 4 % air voids was achieved by the application of 50 number of blows, the aim of preparing modified Marshall specimens was to achieve 4% air voids by applying the same energy per unit volume, as was applied in the standard Marshall specimens. Hence, the air voids and energy values of standard Marshall specimens were used for the calculation of number of blows for the tall specimens based on two different methods. In one method, the total energy values were used while in the second method, VTM values of the standard Marshall specimens were utilized to calculate the final number of blows for the modified Marshall specimens.

4.2.1 Estimation of Number of Blows Based on Energy per Unit Volume

In this case, a fixed number of blows for the modified Marshall specimens were calculated on the basis of the estimation that the energy per unit volume of the standard and the modified Marshall specimens were the same. This estimation was made in order to find out the number of blows that were required for the tall specimens in order to achieve the same energy per unit volume as was achieved in the short specimens after the application of 50 blows. The energy values of the standard Marshall specimens, as listed in Table 4.3, were calculated by using the following formula:

$$Energy = \frac{Number\ of\ blows\ x\ weight\ of\ rammer\ x\ height\ of\ drop\ x\ 2\ x\ 9.81}{Volume\ of\ the\ specimen} \qquad (4.2)$$

Where the value 9.81 is the force of gravity used to convert mass into weight. The energy value is multiplied by 2 as the compaction is done on both sides of the specimen. The weight of the rammer and the height of its drop are shown in Table 4.3. The energy values obtained by Equation (4.2) were then used in the equation derived from the graphical relationship between energy and number of blows of tall specimens. This was done in order to calculate the design number of blows for the tall specimens as shown in Figure 4.1. All the energy values for the short specimen as shown in Table 4.3 were used in the corresponding equations obtained from the graph between energy and number of blows of tall specimens, and the resulting number of blows were obtained for tall specimens. The procedure is described only for specimen FH1 and the number of blows for the rest of the specimens can be seen in Appendix B.

Table 4.3 Energy calculation for standard Marshall specimens

Mix Type	Height (cm)	Blows	Volume (cm^3)	Rammer Weight (Kg)	Rammer Drop (meters)	Energy (Joules / cm^3)
CL501	6.19	50	486.41	4.54	0.457	4.18
CL502	6.29		494.52			4.11
CH501	6.19		486.41			4.18
CH502	6.29		494.52			4.11
FL501	6.19	50	486.41	4.54	0.457	4.18
FL502	6.19		486.41			4.18
FH501	6.19		486.41			4.18
FH502	6.19		486.41			4.18

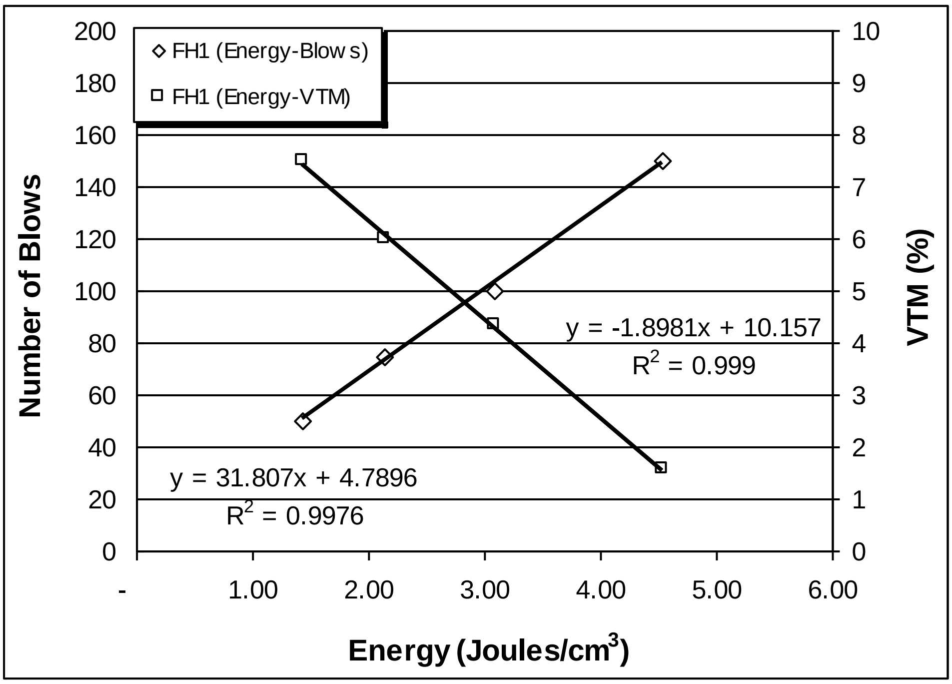

Figure 4.1 Graph for fine graded modified Marshall specimen with high asphalt content, replicate 1

4.2.2 Estimation of Number of Blows Based on Air Voids

In this case, the number of blows for the tall specimens is calculated on the basis of the air voids of the corresponding short specimens as shown in Table 4.1. The air voids of the tall and short specimens are assumed equal and the corresponding number of blows is calculated. This assumption is made in order to find out the number of blows in the tall specimens that will achieve the same VTM as was achieved in the short specimens after the application of 50 blows. Initially, the air voids value of the standard Marshall specimen were used to calculate their energy by using the relationship between the air voids and energy of tall specimens as shown in Figure 4.2. This energy value of the short specimen is then used to calculate the desired number of blows for the corresponding tall specimens by using the graphical relationship between the energy and the number of blows of the tall specimens, as shown in Figure 4.1. The graphical relationship between the number of blows, energy and VTM for the rest of the tall specimens are shown in Appendix A.

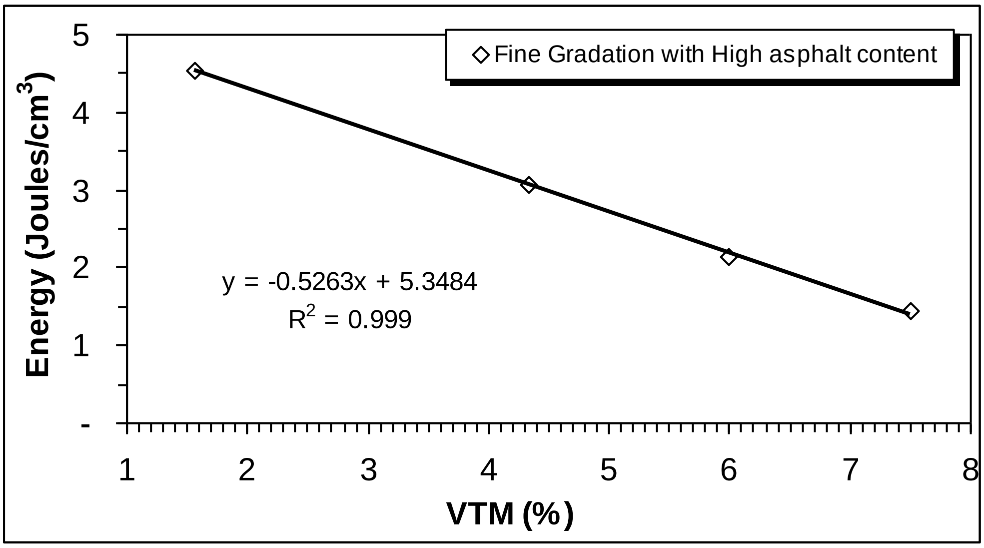

Figure 4.2 Graph for fine graded tall specimen with high asphalt content, replicate 1

As shown in Figure 4.2, an equation was derived from a graphical relation between the air voids and the energy values of the tall specimen FH1. In this equation, the air voids of the short specimen FH501, obtained from Table 4.1, is used and the corresponding energy for the tall specimen is calculated, which would later be used in the equation of the graph between energy and number of blows of the tall specimens, as shown in Figure 4.1. This is done in order to obtain the corresponding number of blows for the tall specimens. The complete list of the comparison between the results obtained from the two methods is shown in Table 4.4.

Table 4.4 Comparison of number of blows calculated through different procedures for modified Marshall specimens

Mix Type	Number of Blows Estimated Based on Energy per Unit Volume	Number of Blows Estimated Based on Air Voids
FH1	137.81	132.52
FL2	140.59	136.01
FH2	135.43	133.91
FL1	139.11	133.68
CL2	139.89	142.23
CL1	143.82	152.41
CH1	139.51	172.17
CH2	137.74	169.22
Average = 139.23		Average = 146.51
Standard Deviation= 2.45		Standard Deviation= 16.28
Coefficient of Variation = 1.76		Coefficient of Variation= 11.11

There are two reasons due to which the average number of blows obtained from energy per unit volume criteria was given preference over the results based on air voids criteria, in order to decide a single design number of blows for the modified Marshall specimens. Firstly, as shown in Table 4.4, the number of blows calculated from the energy per unit volume criteria has a smaller standard deviation and coefficient of variation as compared to the values obtained for the number of blows calculated from air voids criteria. This means that the results of the “energy per unit volume criteria” are more consistent and vary less as compared to the results obtained from the “air voids criteria”. Secondly, as mentioned in the literature review section, Kandhal, (1989) also determined the design number of blows for larger specimens based on the unit energy values of the short specimens rather than based on the air voids. Hence, it can be concluded that the number of blows obtained from the energy per unit volume criteria, i.e. approximately 140 blows, should be taken as the design number of blows for the modified Marshall specimens. The comparison between the number of blows calculated through two different methods can be seen in Figure 4.3.

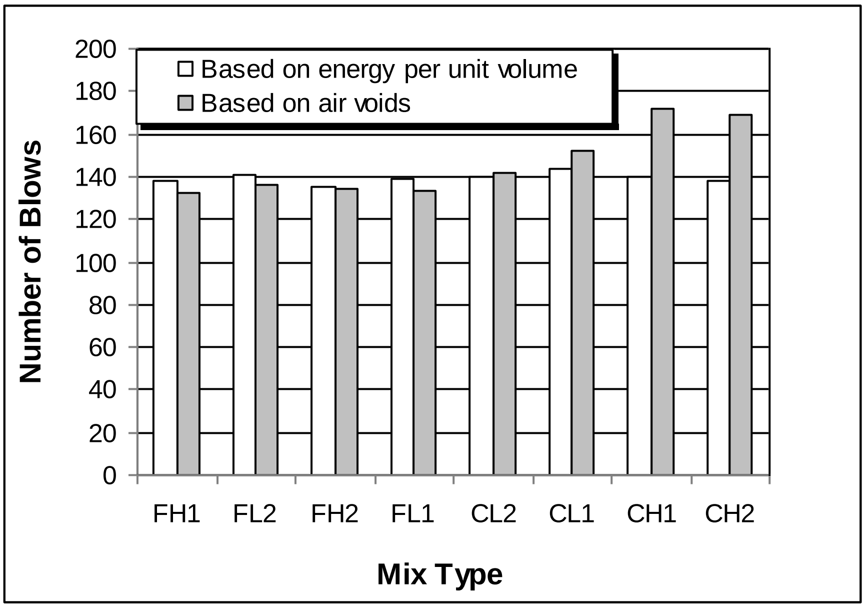

Figure 4.3 Number of blows calculated through two different methods

4.3 Comparison of Air Voids of Modified Marshall Specimens with Standard Marshall Specimens

The calculated air voids of the modified Marshall specimens can be determined as follows. The energy values for short specimens were calculated as shown in Table 4.1. These energy values were then used to calculate the air voids of the tall specimens using the equations formed from the graphs between energy and the air voids of the tall specimens as shown in Figure 4.1. These calculated air voids of tall specimens were then compared with the measured air voids of the short specimens as shown in Table 4.5.

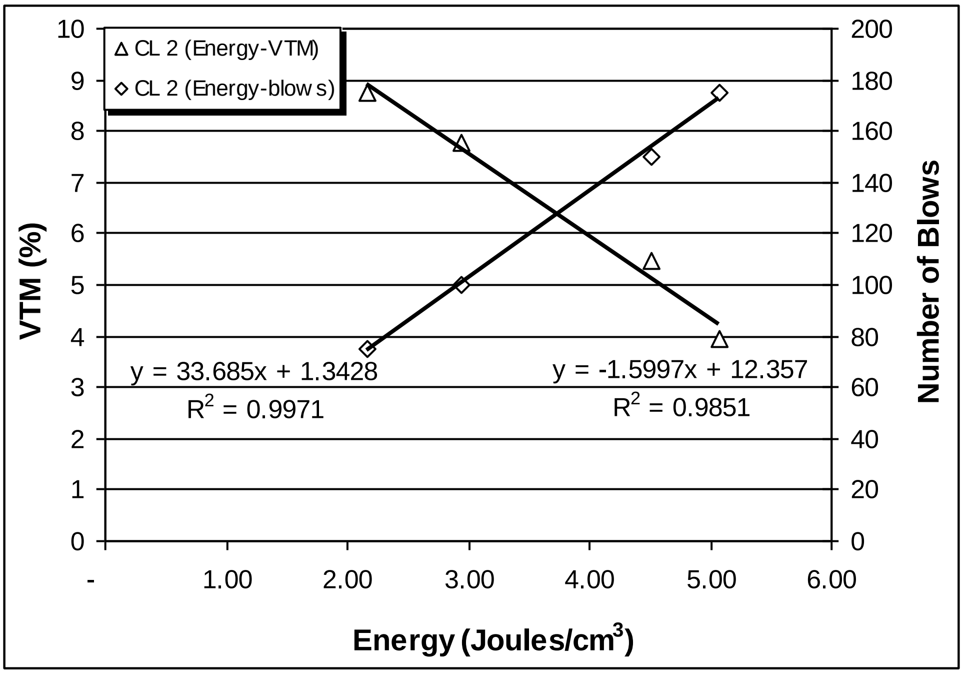

Figure 4.4 Graph for coarse graded tall specimen with low asphalt content, replicate 2

For specimen CL2, the graphical relationship between the energy and the air voids is shown in Figure 4.4. The procedure can be summarized based on specimen CL502 as follows: the energy value of the short sample CL502 is used in the equation of the trend line in Figure 4.4, and the required VTM value is obtained. The air voids of all the modified specimens are calculated in the same manner using the energy values of the short samples, which are listed in Table 4.5.

Table 4.5 Comparison of air voids calculations for tall and short specimens

Mix Type	VTM for Tall Specimens-V_m (%)	VTM for Short Specimens-V_s (%)	ΔVTM(%) $=V_m - V_s$
FH1	2.21	2.53	-0.32
FL2	2.38	2.46	-0.08
FH2	3.51	3.76	-0.25
FL1	3.47	3.66	-0.19
CL2	6.10	5.71	0.39
CL1	5.80	5.66	0.14
CH1	5.21	4.03	1.18
CH2	5.06	3.98	1.08
Mean of ΔVTM (%) = 0.24			
Standard Deviation of ΔVTM (%) = 0.59			

The ΔVTM values show the difference of the air voids of tall and short specimens. It can be noted from Table 4.5 that the ΔVTM of all the fine graded specimens are negative and the ΔVTM values of all the coarse graded specimens are positive. Hence, it can be concluded that for fine graded specimens the air voids in the tall specimens are less than the air voids in the short specimens. However, in the case of coarse graded specimens, the air voids in the tall specimens are generally higher than the air voids in the short specimens. In addition, the fine graded specimens with high asphalt content have lower air voids than the fine graded specimens with low asphalt content. However, in the case of coarse gradation, the specimens with high asphalt content have higher air voids than the specimens with low asphalt content.

In Figure 4.5, it can be seen that in the case of fine graded specimens, the air voids of both tall and short specimens are very low while for the coarse graded specimens, the air voids are relatively higher. This phenomenon can be because there are more air voids in the coarse graded specimens as compared to the fine graded specimens.

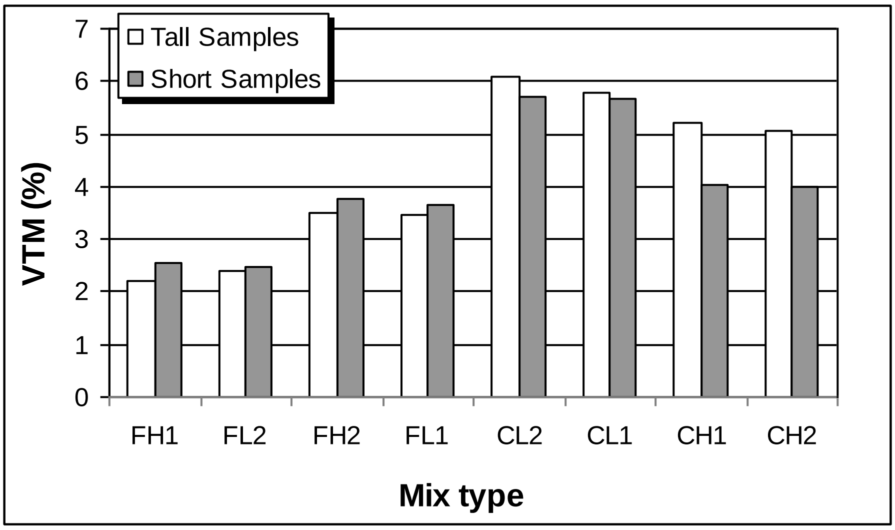

Figure 4.5 Relationship between air voids of tall and short specimens

Figure 4.6 shows the relationship between the air voids of the tall and short specimens, the values of which are shown in Table 4.5. It can be seen that the air voids of the tall specimens compare well with the air voids of the short specimens with a coefficient of determination of 0.87.

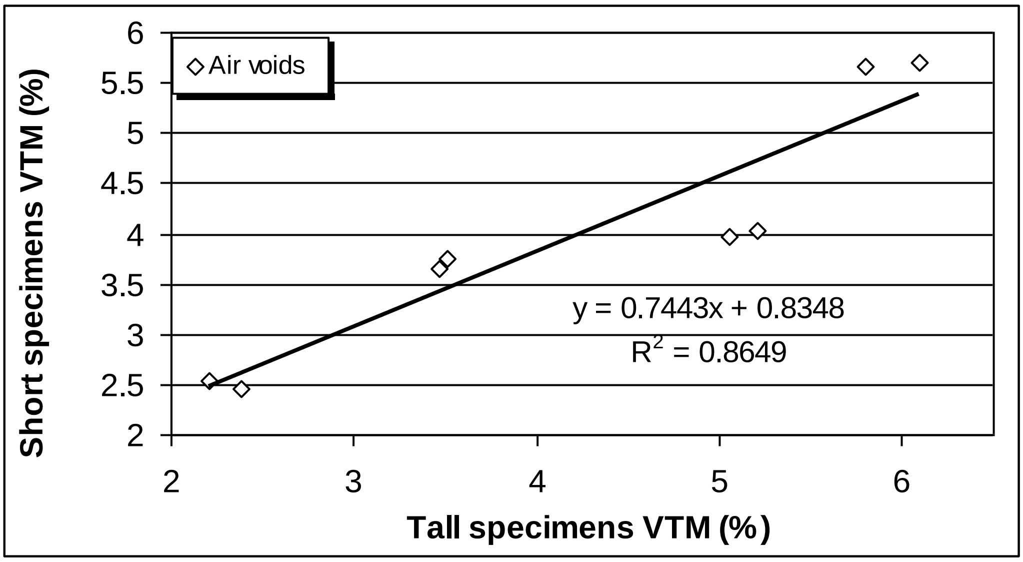

Figure 4.6 Comparison of air voids for tall (modified) and short (standard) specimens

It can be observed in Table 4.6 and Table 4.7 that the mean and the standard deviation of the ΔVTM values for the fine graded specimens are lower than those for the coarse graded specimens. Hence, this indicates that the change in the gradation hence the asphalt content for the fine graded specimens has less effect on the air voids as compared to the change for the coarse graded specimens, in which the increase in asphalt content has a prominent effect on the air voids of the total mix.

Table 4.6 Analysis of air voids calculations for fine graded tall and short specimens

Mix Type	VTM for Tall Specimens-V_m (%)	VTM for Short Specimens-V_s (%)	ΔVTM (%) $=V_m - V_s$
FH1	2.21	2.53	-0.32
FL2	2.38	2.46	-0.08
FH2	3.51	3.76	-0.25
FL1	3.47	3.66	-0.19
Mean of ΔVTM (%) = -0.21			
Standard Deviation of ΔVTM (%) = 0.10			

Table 4.7 Analysis of air voids calculations for coarse graded tall and short specimens

Mix Type	VTM for Tall Specimens-V_m (%)	VTM for Short Specimens-V_s (%)	ΔVTM (%) $=V_m - V_s$
CL2	6.10	5.71	0.39
CL1	5.80	5.66	0.14
CH1	5.21	4.03	1.18
CH2	5.06	3.98	1.08
Mean of ΔVTM (%) =0.69			
Standard Deviation of ΔVTM (%) = 0.51			

Figure 4.7 shows ΔVTM (%) values of different combinations of mix types. It is clear from Figure 4.7 that difference between the air voids of the tall and short specimens is prominently higher in case of coarse graded specimens with high asphalt content as compared to the other combinations used.

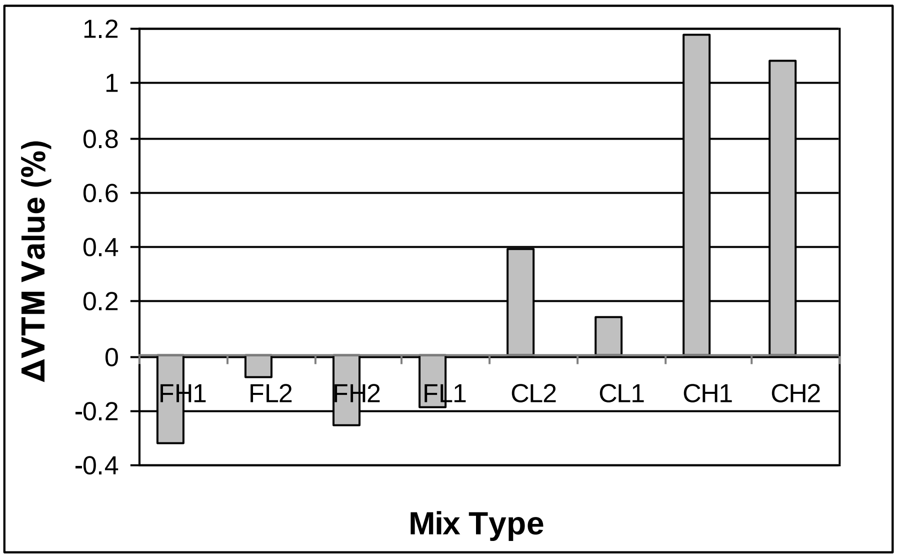

Figure 4.7 Values of the difference between air voids of tall and short specimens

CHAPTER 5

RESULTS AND DISCUSSIONS FOR RUTTING TEST

In this chapter, rutting test parameters are defined and their relations to the rutting behavior of asphalt concrete specimens are depicted. The repeated creep test results for each test parameter are listed and the effects of Recycled Cement Concrete content (RCC %) is discussed in terms of rutting behavior. The relation of each test parameter with the test results was considered and the goodness-of-fit statistical parameters were used in rating the effect of increase in the RCC content on each test parameter.

5.1 Rutting Behavior of Modified Marshall Specimens with Recycled Concrete Aggregate

As mentioned in Chapter 3, the five parameters selected for rutting tests were flow number, slope, intercept, ε_p / ε_r @1000 pulse count and permanent strain @ 1000 cycles. Table 5.1 shows the definition of each parameter and describes how their high and low values impact the rutting behavior of the asphalt mix specimens.

Table 5.1 Selected rutting parameters and their descriptions

Parameters	*Definition*	*Effect on Rutting Behavior of Asphalt Mix*	
		High Value	***Low value***
Flow Number	The starting point or cycle number at which the tertiary flow (associated with pure plastic shear deformation) occurs	Low rutting	High rutting
Slope	The rate of change in the permanent strain as a function of the change in loading cycles	High rutting	Low rutting
Intercept	The permanent strain at number of cycles (N) equal to 1	High rutting	Low rutting
Permanent Strain @1000 cycles	Axial deformation measured with a linear variable displacement transducer (LVDT) at 1000 load cycles	High rutting	Low rutting
ε_p / ε_r @ 1000 cycles	Ratio of the permanent strain to the resilient strain measured at 1000 cycles	High rutting	Low rutting

The results of these rutting parameters for all the specimens are shown in Table 5.2. The coarse and fine graded modified Marshall specimens are listed separately in Table 5.2. The symbols assigned to each combination has been given on the basis of C for coarse and F for fine graded specimens. The second letter in the symbols indicates the RCC% and the third letter indicates the replicate number. The values of ε_p / ε_r @ 1000 cycles and permanent strain @ 1000 cycles for the specimen C03 are not mentioned due to the fact that this specimen failed before reaching 1000 pulse counts.

Table 5.2 Final results for repeated creep tests

Mix Type	VTM (%)	Height (cm)	Dia. (cm)	Flow Number	Slope	Intercept	ε_p / ε_r @ 1000 Cycles	Permanent Strain @ 1000 cycles
Fine Gradation								
F01	3.8	16.7	100	9960	0.28	2.94	11.19	6190
F02	4.3	16.6		9960	0.26	2.10	4.12	741
F03	3.6	16.8		8253	0.20	2.85	8.96	2768
F251	4.4	16.7	100	5198	0.25	2.86	7.24	4201
F252	3.9	16.9		9400	0.21	2.65	7.74	1873
F253	4.5	16.9		8245	0.25	2.92	8.36	4625
F501	3.6	16.7	100	9960	0.20	2.86	5.26	2831
F502	4.0	16.6		8442	0.26	2.84	6.39	4204
F503	4.3	16.5		2699	0.26	2.85	6.73	4422
F751	3.6	16.6	100	6255	0.31	2.90	8.41	6691
F752	4.1	16.9		1896	0.31	2.77	6.31	5181
F753	4.3	16.7		2162	0.29	2.84	6.23	5200
Coarse Gradation								
Mix Type	**VTM (%)**	**Height (cm)**	**Dia. (cm)**	**Flow Number**	**Slope**	**Intercept**	**ε_p / ε_r @ 1000 Cycles**	**Permanent Strain @ 1000 cycles**
C01	4.5	17.1	100	1613	0.31	3.17	22.03	14119
C02	4.5	16.8		637	0.36	2.98	18.42	13573
C03	3.7	16.9		399	0.39	3.22	N/A	N/A
C251	3.7	17.2	100	1092	0.34	2.84	12.42	7760
C252	3.8	17.1		9960	0.25	2.87	13.50	4240
C253	3.8	16.9		4052	0.33	2.89	13.78	8019
C501	3.5	16.7	100	5230	0.32	2.81	9.84	5981
C502	3.8	17.2		1448	0.32	2.86	9.85	6873
C503	4.3	16.8		1493	0.33	2.82	10.85	7039
C751	3.5	17.0	100	2592	0.32	2.70	7.33	4799
C752	3.7	16.8		7788	0.28	2.65	6.77	3085
C753	3.7	16.7		3899	0.26	2.83	8.76	4142

5.2 Effect of Recycled Cement Concrete Content on Repeated Creep Test Parameters

5.2.1 Flow Number

The starting point or the cycle number at which the tertiary flow associated with pure plastic shear deformation occurs is referred to as the "Flow number". Higher the flow number for HMA indicates better the resistance against rutting, which means that an asphalt mix with low flow number is most likely subjected to rutting.

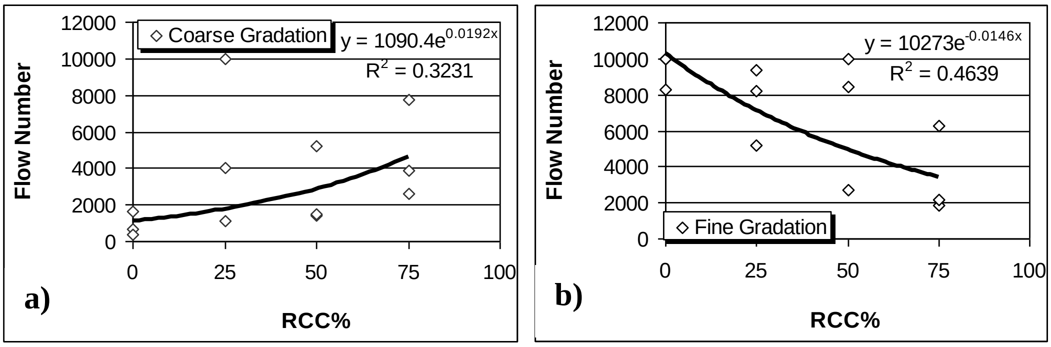

Figure 5.1 Flow number vs. RCC content a) for coarse graded specimens, b) for fine graded specimens

As shown in Figure 5.1, the repeated creep test results show that the flow number for the coarse graded specimens increases gradually with an increase in the recycled cement concrete percent, which indicates that the addition of RCC to the coarse graded specimens increases the resistance against rutting. On the other hand, there is a relatively sharper decrease in the flow number for the fine graded specimens with an increase in the recycled material percentage. As low flow number of any mix indicates lower resistance against rutting, it can be concluded that even a little addition of RCC to the fine graded specimens reduces its resistance against rutting.

5.2.2 Slope Constant

The slope constant represents the rate of change in the permanent strain as a function of the change in loading cycles (N) in the log-log scale when the rutting behavior is modeled using a Power model. High slope constant for a mix indicates an increase in the material deformation rate hence less resistance against rutting. A mix with a low slope constant is preferable as it prevents the occurrence of the rutting distress mechanism at a slower rate.

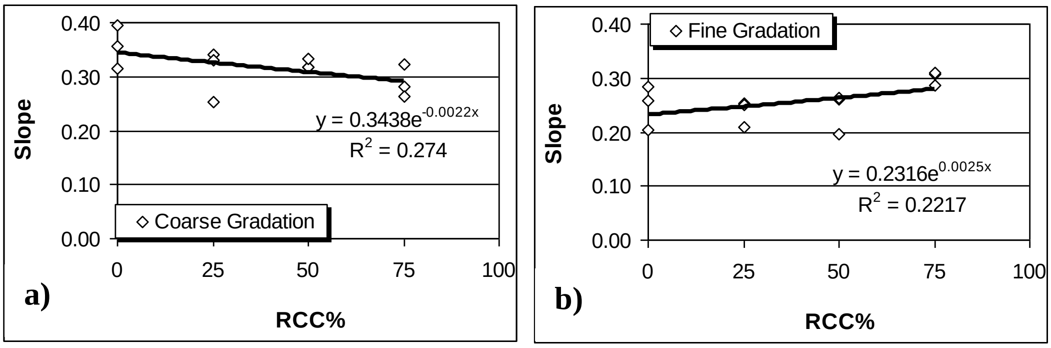

Figure 5.2 Slope constant vs. RCC content a) for coarse graded specimens, b) for fine graded specimens

As shown in Figure 5.2, there is a minor decrease in the slope of coarse graded specimen with an increase in the RCC percent. A lower slope means that there is a less material deformation and hence more resistance against rutting, therefore, it can be concluded that the addition of RCC in the coarse graded mix improves the resistance of the mix against rutting. On the other hand, in the case of fine graded specimens, the slope increases with an increase in the RCC content, which means that the addition of RCC in the fine graded mix causes decrease in the rutting resistance of the specimens.

5.2.3 Intercept Constant

The intercept represents the permanent strain at N=1, where N is the number of the load cycles. The higher the value of intercept, the larger the strain and hence the larger the potential for permanent deformation as mentioned in the Superpave study carried out by Witczak et al. (1999).

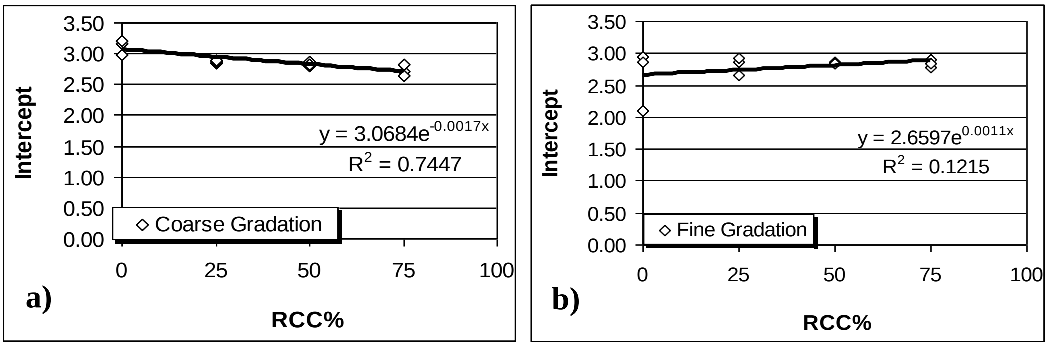

Figure 5.3 Intercept constant vs. RCC content a) for coarse graded specimens, b) for fine graded specimens

As shown in Figure 5.3, there is a minor decrease in the intercept value for the coarse graded specimens with an increase in the RCC content, which means that the addition of RCC to the coarse graded specimens improves their resistance against rutting. There is a minor increase in the intercept constant for the fine graded specimens with an increase in the RCC, which also indicates the fact that addition of RCC to the fine graded specimens reduces the rutting resistant. The parameters “intercept” and “slope” contents are assumed the indicators for the rutting potential of asphalt concrete. The larger the “intercept” value, the larger the strain and hence larger the potential for permanent deformation. In addition, the larger the “slope” value, larger the potential for permanent deformation.

5.2.4 Permanent Strain

One objective of using the permanent strain parameter in this research is to compare the combined influence of the intercept and the slope parameters with the rut depth measurements. The selected number of cycles at which these values are compared are arbitrary. These were selected to keep a common reference among all the test specimens. In this study, the permanent strain was measured at 1000 cycles. The increase in permanent strain leads to higher rutting in a flexible pavement, which is an undesirable characteristic of HMA against rutting.

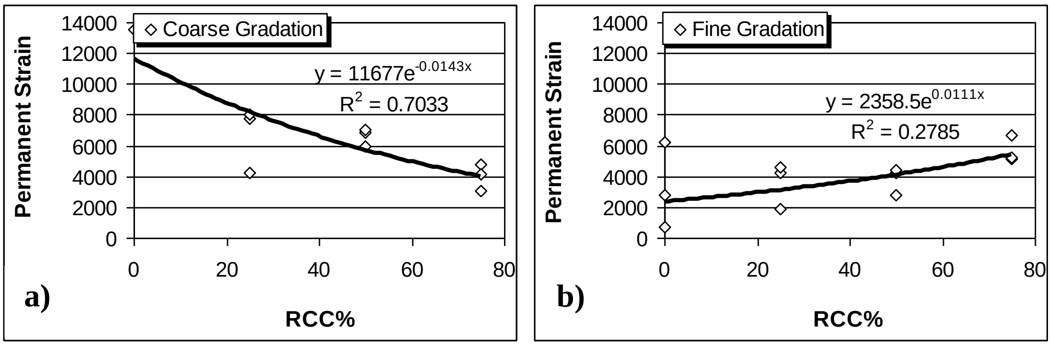

Figure 5.4 Permanent strain @ 1000 cycles vs. RCC content a) for coarse graded specimens, b) for fine graded specimens

As shown in Figure 5.4, the permanent strain @ 1000 cycles decreases rapidly for the coarse gradation with an increase in the RCC content. As the decrease in permanent strain indicates higher resistance against rutting, therefore it can be concluded that the increase in RCC% in the coarse graded specimens adds to the overall rutting resistance of the HMA. On the other hand, permanent strain @ 1000 cycles increases in fine gradation with the increase in RCC%, which means that addition of RCC in the fine graded specimens decreases its resistance against rutting.

5.2.5 ε_p / ε_r @ 1000 Cycles

The parameter ε_p / ε_r @ 1000 cycles is actually the ratio of ε_p (plastic strain) to ε_r (resilient strain) at a load cycle of 1000 which can be used as one of the standards for the assessment of mixture performance. Its value is dependent on mix parameters, test temperature and stress level. The bigger the value of this ratio, the more vulnerable the mix becomes to rutting.

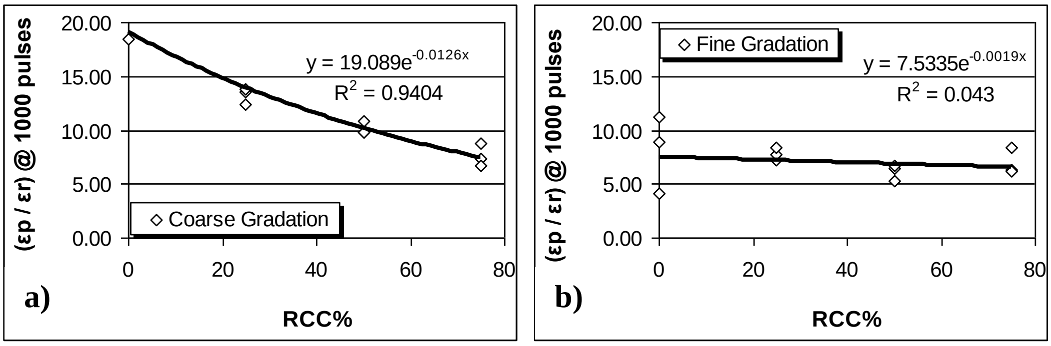

Figure 5.5 $\varepsilon_p/\varepsilon_r$ @ 1000 cycles vs. RCC content a) for coarse graded specimens, b) for fine graded specimens

As shown in Figure 5.5, the ε_p / ε_r ratio @1000 cycles for coarse graded specimens decreases sharply with the addition of RCC to the mix, which indicates that even less a amount of RCC in the coarse graded specimens greatly improves their resistance against rutting. In the case of fine graded specimens, an increase in the RCC content results in gradual decrease in the ε_p / ε_r ratio, which means that higher content of RCC in the fine graded specimens results in smaller improvement in the rutting resistance of the mixes as compared to the coarse graded specimens.

All the rutting parameters support the fact that the addition of RCC in coarse graded specimens improves its rutting resistant properties while the rutting resistance of fine mixes reduces with an increase in the RCC content. The visual condition of the specimen after testing and the amount of deformation can also be an indication of the mixture strength against rutting. Poor performing mixtures generally deform most while good performing mixtures deform to a smaller extent.

5.3 Goodness-of-Fit statistics for Test Parameters

A number of different statistical models were investigated to achieve the best correlation between RCC content and rutting parameters. As compared to hyperbolic and power models, the exponential model gives the best correlation and the largest R^2 for the different parameters considered in this study. The R^2 values obtained for the selected parameters are shown together with the corresponding relationships in the above sections. Table 5.3 of Witczak et al., (1999) shows the criteria that can be used to rank the correlations.

Table 5.3 Subjective classification of the goodness-of-fit statistical parameters (Witczak et al., 1999)

CRITERIA	R^2
Excellent	> 0.90
Good	0.70 - 0.89
Fair	0.40 - 0.69
Poor	0.20 – 0.39
Very Poor	< 0.19

In this study, the R^2 values obtained for different parameters after rutting test and their rating are shown in Table 5.4.

Table 5.4 Rating for parameters used in this study

Parameter	Gradation	R^2 Value	Rating
Flow number	Coarse	0.32	(0.20 – 0.39) Poor
	Fine	0.46	(0.40 - 0.69) Fair
Slope	Coarse	0.27	(0.20 – 0.39) Poor
	Fine	0.22	(0.20 – 0.39) Poor
Intercept	Coarse	0.74	(0.70 - 0.89) Good
	Fine	0.12	(< 0.19) Very Poor
ε_p / ε_r @ 1000	Coarse	0.94	(> 0.90) Excellent
	Fine	0.04	(< 0.19) Very Poor
Permanent strain @ 1000 cycles	Coarse	0.70	(0.70 - 0.89) Good
	Fine	0.28	(0.20 – 0.39) Poor

As shown in Table 5.4, the flow number for the coarse graded specimens has a poor correlation with RCC percent while the flow numbers for the fine graded specimens maintain a fair correlation with the RCC percent according to the criteria suggested by Witczak et al. (1999). Similarly, the slopes of both coarse and fine graded specimens have a poor correlation with the RCC%. The intercept of the coarse graded specimens has a good correlation with the RCC%, but the flow numbers for the fine graded specimens have a very poor correlation with the RCC%. While the ε_p / ε_r ratio @ 1000 cycles for the coarse graded specimens has an excellent correlation with the RCC%, the flow numbers for the fine graded specimens have a very poor correlation with the RCC%. The permanent strain @ 1000 cycles for coarse graded specimens has a good correlation with the RCC%, but the flow numbers for the fine graded specimens have a poor correlation with the RCC%. Thus, we can say that coarse graded specimens have generally yield better correlations with the RCC% as compared to the fine graded specimens.

The results obtained by Witczak et al. (1999) are shown in Table 5.5. It is clear from this table that unconfined test gives better correlation between different parameters of rutting test; also, it simulates the field conditions better as compared to confined tests. There is a difference in the values obtained in this study and that determined by Witczak et al. (1999) because they used only crushed natural aggregates, while in this study crushed natural aggregates were used with addition of RCC%.

Table 5.5 Summary of the goodness-of-fit statistics and rationality of the trends for each test parameter (Witczak et al., 1999)

TEST	TEST PARAMETER	CONFINEMENT	MODEL TYPE	STATISTICAL MEASURE					
				TEST TEMP 100° F			TEST TEMP 130° F		
				R^3	Rational	Rating	R^3	Rational	Rating
REPEATED LOAD PERMANENT DEFORMATION	Intercept (a)	Unconfined	Linear	0.30	Yes	Poor	0.01	-	Very Poor
		Confined	Linear	0.07	No	Very Poor	0.13	No	Very Poor
	Slope (b)	Unconfined	Linear	0.59	Yes	Fair	0.95	Yes	Excellent
		Confined	Linear	0.89	Yes	Good	0.62	Yes	Fair
	Flow Number (FN)	Unconfined	Power	0.86	Yes	Excellent	0.84	Yes	Good
			Exponential	0.99	Yes	Excellent	0.99	Yes	Excellent
			Hyperbolic	0.96	Yes	Good	0.97	Yes	Excellent
	Permanent Strain at N=100	Unconfined	Linear	0.97	Yes	Excellent	0.83	Yes	Good
		Confined	Linear	0.37	-	Poor	0.17	-	Very Poor
	Permanent Strain at N=1000	Unconfined	Linear	0.95	Yes	Excellent	0.95	Yes	Excellent
		Confined	Linear	0.71	Yes	Good	0.49	Yes	Fair
	Permanent Strain at N=10000	Unconfined	-	-	-	-	-	-	-
		Confined	Linear	0.86	Yes	Good	0.71	Yes	Good
	$\varepsilon_P/\varepsilon_T$ Ratio	Unconfined	Linear	0.83	Yes	Good	0.003	-	Very Poor

5.4 ANOVA for the Repeated Creep Test Parameters

Two-way ANOVA was carried out for all the test parameters, and the results of each parameter is discussed as follows;

As shown in Table 5.6, the results of the two-way ANOVA for flow number suggests that gradation and the interaction between gradation and recycled content have a significant effect on flow number for the mix as both have P values < 0.05, while recycled content only has an insignificant effect on flow number due to the fact that its P value is equal to 0.571 > 0.05.

Table 5.6 ANOVA for flow number

Source	Sum Sq	d.f.	Mean Sq.	F	Prob>F
Gradation	7.429e+007	1	7.429e+007	10.05	0.005
Recycled Content	1.531e+007	3	5.101e+006	0.69	0.571
Gradation*Recycled Content	1.475e+007	3	2.491e+007	3.37	0.044
Error	1.182e+008	16	7.392e+006		
Total	2.826e+008	23			

Table 5.7 ANOVA for slope

Source	Sum Sq	d.f.	Mean Sq.	F	Prob>F
Gradation	0.022	1	0.227	20.6	0.000
Recycled Content	0.003	3	0.010	0.93	0.447
Gradation*Recycled Content	0.011	3	0.003	3.62	0.036
Error	0.017	16	0.001		
Total	0.055	23			

As shown in Table 5.7, the results of the two-way ANOVA suggest that gradation and the interaction between gradation and recycled content have a significant effect on slope constant as both have P values < 0.05, while recycled content only has an insignificant effect on slope due to the fact that the P value is equal to 0.4476 >0.05.

Table 5.8 ANOVA for intercept

Source	Sum Sq	d.f.	Mean Sq.	F	Prob>F
Gradation	0.066	1	0.066	1.98	0.178
Recycled Content	0.027	3	0.009	0.27	0.843
Gradation*Recycled Content	0.317	3	0.105	3.17	0.053
Error	0.534	16	0.033		
Total	0.944	23			

As shown in Table 5.8, the results of the two-way ANOVA suggest that gradation, recycled content, and the interaction between gradation and recycled content have no significant influence on intercept for the mix as all the P values are > 0.05.

Table 5.9 ANOVA for permanent strain

Source	Sum Sq	d.f.	Mean Sq.	F	Prob.>F
Gradation	3.927e+007	1	3.927e+007	3.89	0.066
Recycled Content	6.578e+006	3	2.192e+006	0.22	0.883
Gradation*Recycled Content	4.526e+007	3	1.508e+007	1.49	0.254
Error	1.615e+008	16	1.009e+007		
Total	2.526e+008	23			

As shown in Table 5.9, the results of the two-way ANOVA suggest that gradation, recycled content, and interaction between gradation and recycled content have no significant influence on permanent strain as all of the P values are > 0.05.

Table 5.10 ANOVA for $\varepsilon_p/\varepsilon_r$ @ 1000

Source	Sum Sq	d.f.	Mean Sq.	F	Prob.>F
Gradation	90.415	1	90.415	4.61	0.047
Recycled Content	53.593	3	17.864	0.91	0.457
Gradation*Recycled Content	22.97	3	7.6568	0.39	0.761
Error	313.951	16	19.621		
Total	480.929	23			

As shown in Table 5.10, the results of the two-way ANOVA suggest that the gradation has a significant effect on $\varepsilon_p/\varepsilon_r$ @ 1000 with a P value < 0.05. However, none of the other factors that is, recycled content and the interaction between gradation and recycled content have a significant influence on $\varepsilon_p/\varepsilon_r$ @ 1000 with a P value > 0.05.

5.4.1 Regression Analysis for the Repeated Creep Test Parameters

A regression analysis was carried out for the 5 parameters of the repeated creep test in which a linear model was used based on the results of the two way ANOVA obtained in the previous section. Each test parameter was taken as the dependent variable while the gradation and the RCC% were designated as the independent variables. The gradation was termed as X_1 and the RCC% was considered X_2. For the fine graded specimens the value 1 was used while the number 2 was used for the coarse graded specimens. The four values used for the RCC% were 0, 25, 50 and 75. The following regression equation was used:

$$y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 \tag{5.1}$$

Where β_0 is a constant, β_1 is non-standardized coefficient for X_1 and β_2 is non-standardized coefficient for X_2

For each of the test parameters used, regression analysis was first carried out with both independent variables. The regression analysis was carried out second time with only the independent variable whose P value is less than 0.05 in the first regression. The R^2 values helped in finding out which parameter best characterizes the rutting behavior of the test specimens.

In Table 5.11, the first row shows the details of the regression model with dependent variable y (flow number) and two independent variables X_1 (Gradation) and X_2 (RCC%). β_0, β_1 and β_2 are the non-standardized coefficients for the constant, X_1 and X_2 respectively. The t values and the significance of both β_1 and β_2 are shown in the relevant columns. The R^2 value shows how good the correlation exists between the test parameter and the independent variables. S is used for the standard error of the estimate. The F value and the significance shown in the last column is used for the Regression model.

The second row in Table 5.11 shows the second regression that was carried out with only one independent variable, i.e. excluding the independent variable with P value > 0.05. As evident from Table 11, the significance of the model has improved after the removal of the unacceptable independent variable.

The Regression analysis for the rest of the parameters was carried out in the same manner, the results of which can be seen in Tables as follows.

Table 5.11 Regression analysis for flow number

Model	β_0	β_1	t (Sig.)	β_2	t (Sig.)	R^2	Adjusted R^2	S	F (Sig.)
$y = \beta_0 + \beta_1 X_1 + \beta_2 X_2$	11072.2	-3518.9	-2.78 (0.01)	-18.24	-0.805 (0.430)	0.285	0.217	3102.2	4.18 (0.03)
$y = \beta_0 + \beta_1 X_1$	10388.1	-3518.9	-2.801 (0.01)	N/A	N/A	0.263	0.229	3077.3	7.85 (0.01)

S: Standard error of estimate

Table 5.12 Regression analysis for Slope

Model	β_0	β_1	t (Sig.)	β_2	t (Sig.)	R^2	Adjusted R^2	S	F (Sig.)
$y = \beta_0 + \beta_1 X_1 + \beta_2 X_2$	0.197	0.062	3.821 (0.001)	-3.8E-005	-0.131 (0.897)	0.410	0.354	0.039	7.309 (0.004)
$y = \beta_0 + \beta_1 X_1$	0.195	0.062	3.909 (0.001)	N/A	N/A	0.410	0.383	0.03855	15.284 (0.001)

S: Standard error of estimate

Table 5.13 Regression analysis for Intercept

Model	β_0	β_1	t (Sig.)	β_2	t (Sig.)	R^2	Adjusted R^2	S	F (Sig.)
$y = \beta_0 + \beta_1 X_1 + \beta_2 X_2$	2.719	0.105	1.274 (0.217)	-0.001	-0.761 (0.455)	0.095	0.009	0.201	1.101 (0.351)
$y = \beta_0 + \beta_1 X_1$	2.677	0.105	1.286 (0.212)	N/A	N/A	0.070	0.028	0.199	1.655 (0.212)

S: Standard error of estimate

Table 5.14 Regression analysis for Permanent Strain

Model	β_0	β_1	t (Sig.)	β_2	t (Sig.)	R^2	Adjusted R^2	S	F (Sig.)
$y = \beta_0 + \beta_1 X_1 + \beta_2 X_2$	2124.842	2558.583	1.989 (0.060)	-16.165	-0.702 (0.490)	0.175	0.096	3150.99	2.225 (0.133)
$y = \beta_0 + \beta_1 X_1$	1518.667	2558.583	2.012 (0.057)	N/A	N/A	0.155	0.117	3114.50	4.049 (0.057)

S: Standard error of estimate

Table 5.15 Regression analysis for $\varepsilon_p/\varepsilon_r$ @ 1000

Model	**β_0**	**β_1**	**t (Sig.)**	**β_2**	**t (Sig.)**	**R^2**	**Adjusted R^2**	**S**	**F (Sig.)**
$y = \beta_0 + \beta_1 X_1 + \beta_2 X_2$	5.285	3.882	2.359 (0.028)	-0.051	-1.741 (0.096)	0.290	0.223	4.031	4.297 (0.027)
$y = \beta_0 + \beta_1 X_1$	3.363	3.882	2.257 (0.034)	N/A	N/A	0.188	0.151	4.213	5.094 (0.034)

S: Standard error of estimate

It can be concluded from the results obtained that for the database used in the circumstances provided in this study, an acceptable statistical model could not be developed, which might be due to insufficient observations for repeated creep tests. Hence, as opposed to the expectations in the beginning of the study, the levels selected for the two independent variables might not be enough for the development of a statistical model. Increase in the variety of the gradations and the RCC% might lead to better results.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Summary

In this study, effects of different percentages of Recycled Cement Concrete (RCC) on the rutting potential of Hot Mix Asphalt (HMA) specimens were investigated by carrying out rutting creep tests on the modified Marshall specimens. Initially, the specimens were prepared with the design parameters of asphalt content, aggregate gradation, and varying number of blows. Two gradations, coarse and fine, and two asphalt contents, high and low, were selected for both standard and modified Marshall specimens. The energy per unit volume of the standard Marshall (short) specimens was used to achieve the same energy per unit volume for the modified Marshall (tall) specimens. The average number of blows for the tall specimens was calculated by different methods, but the one with the least standard deviation and the coefficient of variation was selected. Three trial asphalt contents were assumed based on which the Optimum Asphalt Content (OAC) was estimated for the tall specimens prepared with different percentages of crushed aggregates and RCC. The OAC was determined for Voids in Total Mix (VTM) of 4±0.5 %, which was considered as the acceptable limit for the specimens. Three replicates were made within the required limits for each combination, which were then tested in the UMMATA to investigate their rutting performance. In the end, the relationships of RCC content in HMA with the test parameters were determined. The test parameters used were slope, intercept, flow number, permanent strain @ 1000 cycles, and ε_p / ε_r @ 1000 cycles. After analyzing the data, a number of conclusions were made based on the results obtained.

6.2 Conclusions

1. In the first phase of the study, the difference between the air void content of the tall and short specimens was prominently higher in case of coarse graded specimens with high asphalt content as compared to all the other combinations used.

2. For the calculation of number of blows for the tall specimens, the criteria of assuming energy per unit area of the short specimens equal to that of tall specimens proved to be more effective than the criteria of assuming air voids of tall and short specimen to be equal.

3. In the second phase of the study, five test parameters; flow number, slope, intercept, permanent strain and $\varepsilon_p/\varepsilon_r$ @ 1000 were analyzed. After increasing the RCC content in HMA specimens, their effects on these five test parameters were evaluated. It was evident from the results that higher flow number is achieved for coarse graded specimens with an increase in the RCC content in the HMA mixes. On the other hand, the flow number for fine graded specimen decreases with an increase in the RCC content.

4. For the coarse graded specimens, there is a decrease in the slope, intercept, permanent strain, and ε_p / ε_r @1000 cycles with an increase in the RCC content. Hence, in general, the addition of RCC in the coarse graded mix improves its resistance against rutting. However, in fine graded specimens the slope, intercept, and permanent strain increases with an increase in RCC, thus generally the addition of RCC to fine graded mixes reduces its rutting resistance.

5. The visual condition of the specimens after the rutting tests and the amount of deformation was also an indication of the mixture strength. Poor performing mixtures were distorted the most, while there was smaller deformation in good performing mixtures.

6. It was clear from the results of the Analysis of Variance (ANOVA) that the gradation and the interaction between gradation and RCC has a significant effect on the flow number, slope and $\varepsilon_p / \varepsilon_r$ @1000 cycles of the mix, as all of them have values of $P < 0.05$. However, it has an insignificant effect on the intercept and the permanent strain. If only RCC is considered, it has an insignificant effect on all of the above-mentioned parameters, as the values of P are greater than 0.05.

7. The results of regression analysis showed that the repeated creep test parameter "slope" has the highest value of R^2 and thus gives the best correlation between RCC content and gradation.

8. After performing regression analysis on the test parameters, it was concluded that RCC content does not have a significant role in predicting the rutting behavior of the HMA; hence, it cannot be forced into the statistical model. This might be because the total number of observations taken in this study might be insufficient to determine the significance of addition of RCC to HMA. In addition, a pneumatic system was used for rutting test, in which there is a lot of variability in the deviator stress, due to which there is a lot of variation in the rutting test parameters. Thus, under the circumstances in which this study was carried out, it was not possible to generate an acceptable model as is clear from the results of the regression analysis.

6.3 Suggestions for Future Studies

It seems clear from the results of this study that the addition of RCC to HMA is a promising area for the future research. In this study, the rutting potential of HMA prepared with RCC was investigated. Such specimens can also be prepared for investigating other distress mechanisms and their potential can be determined with the help of HMA specimens prepared with different recycled materials. As the observations taken for this study turned out to be insufficient for generating a suitable statistical model, more observations of different percentages of RCC, if add to a series of different gradations of HMA, might result in the generation of an acceptable statistical model for regression analysis. The test parameters and parameter levels can be varied, such as finding out the flexural strength or resilient modulus of the specimens prepared with addition of recycled material to HMA. Additional research can be initiated to provide an in-depth analysis of other available waste materials.

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APPENDIX A

GRAPHS FOR NUMBER OF BLOWS, VTM AND ENERGY OF STANDARD AND MODIFIED MARSHALL SPECIMENS

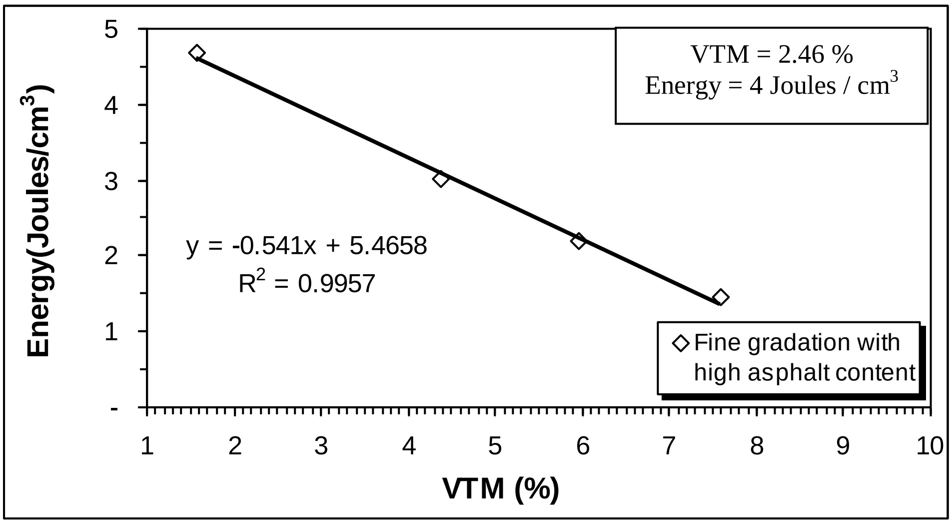

Figure A1 Graph for fine graded short specimen with high asphalt content, replicate 2

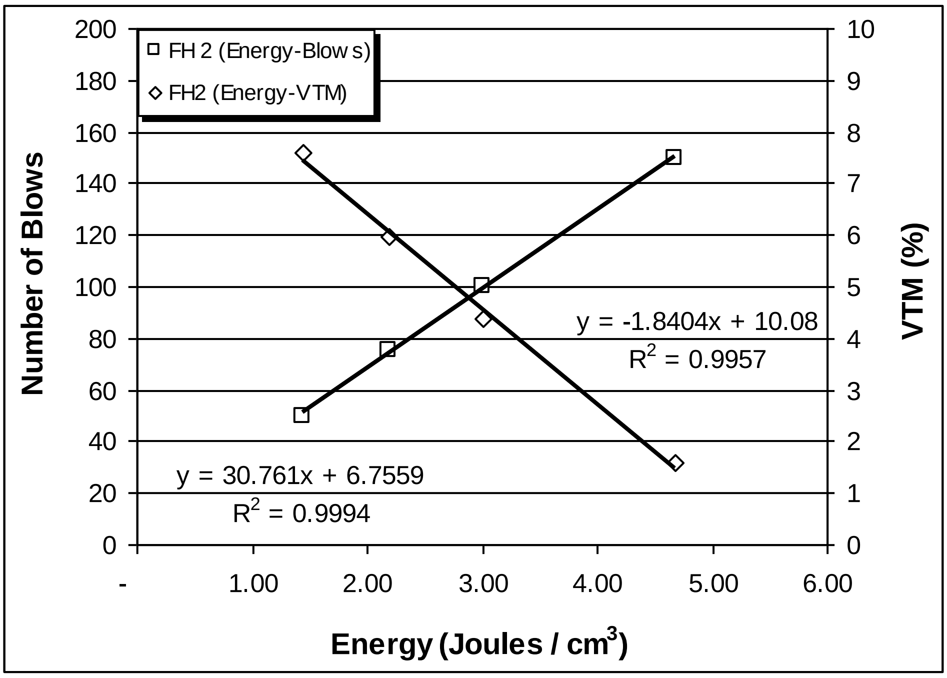

Figure A2 Graph for fine graded tall specimen with high asphalt content, replicate 2

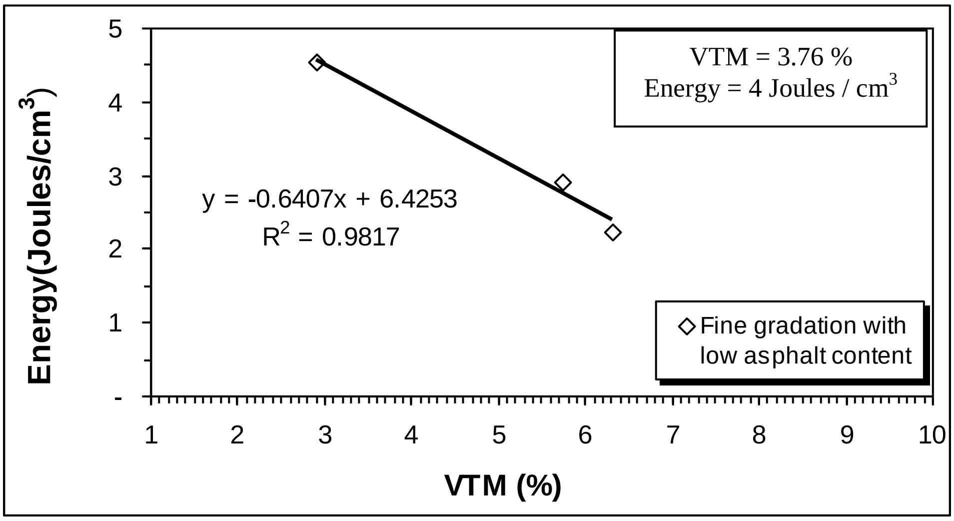

Figure A3 Graph for fine graded tall specimen with low asphalt content, replicate 1

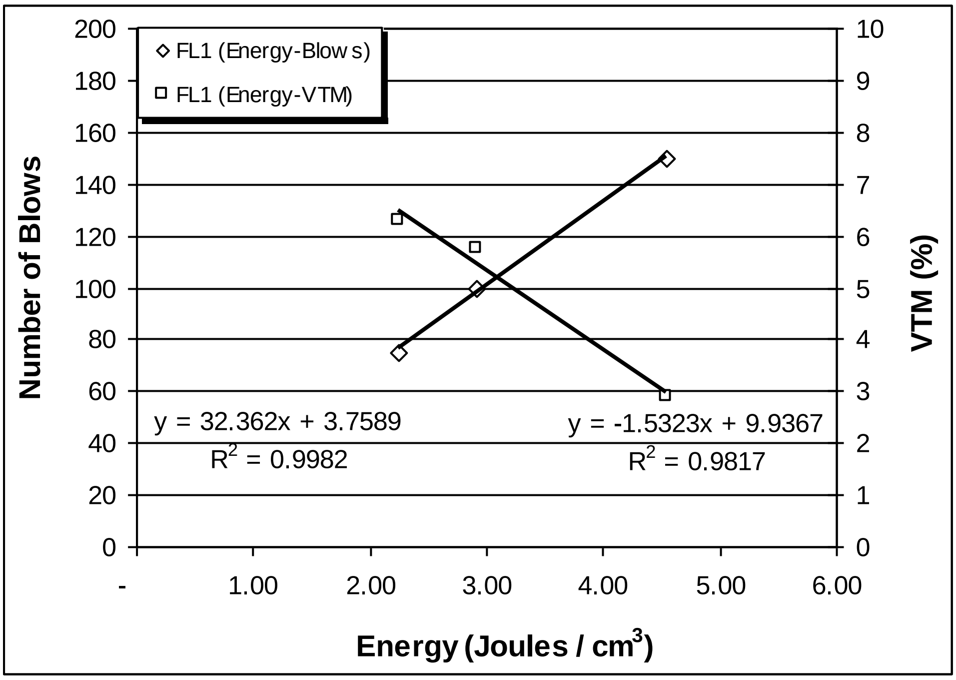

Figure A4 Graph for fine graded tall specimen with low asphalt content, replicate 1

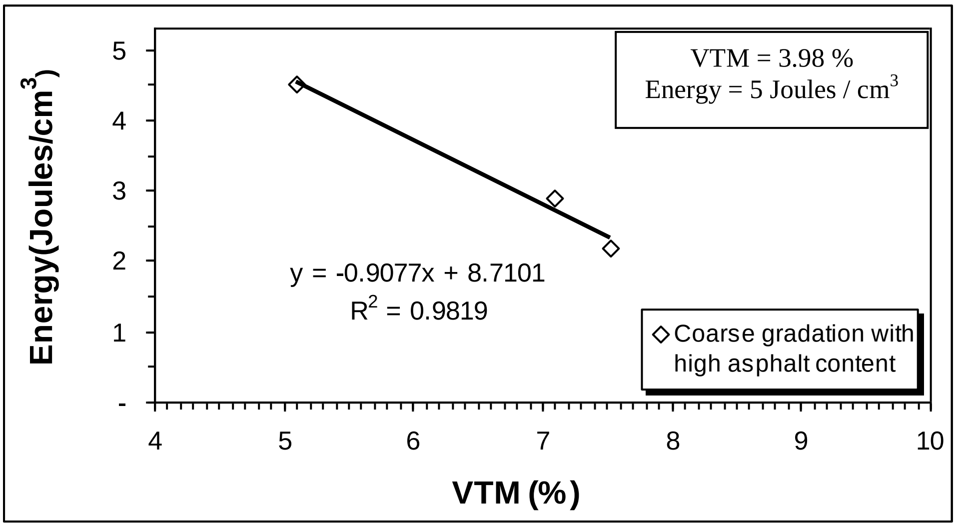

Figure A5 Graph for coarse graded tall specimen with high asphalt content, replicate 2

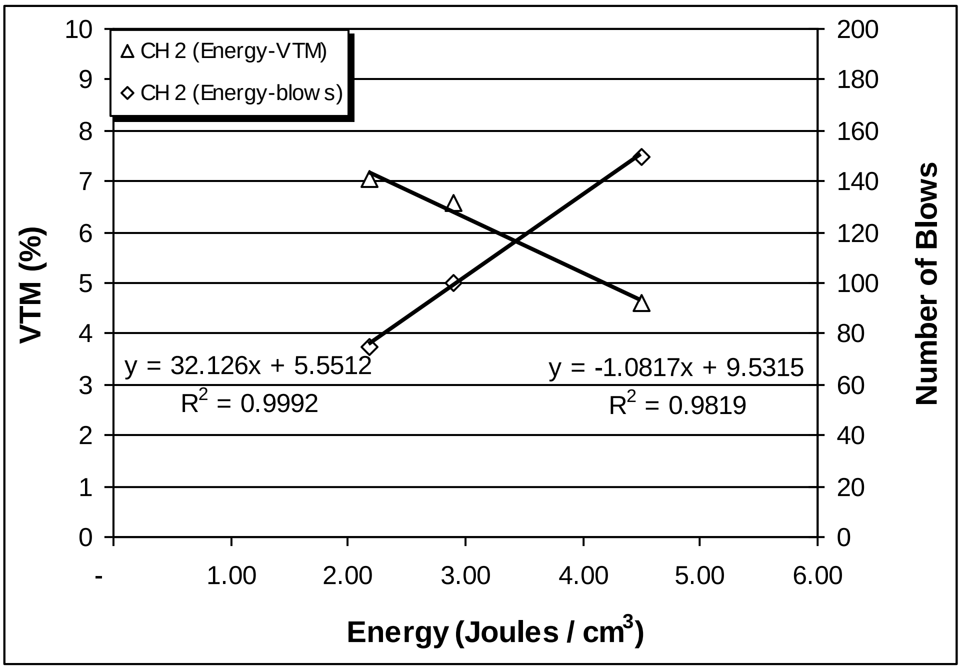

Figure A6 Graph for coarse graded tall specimen with high asphalt content, replicate 2

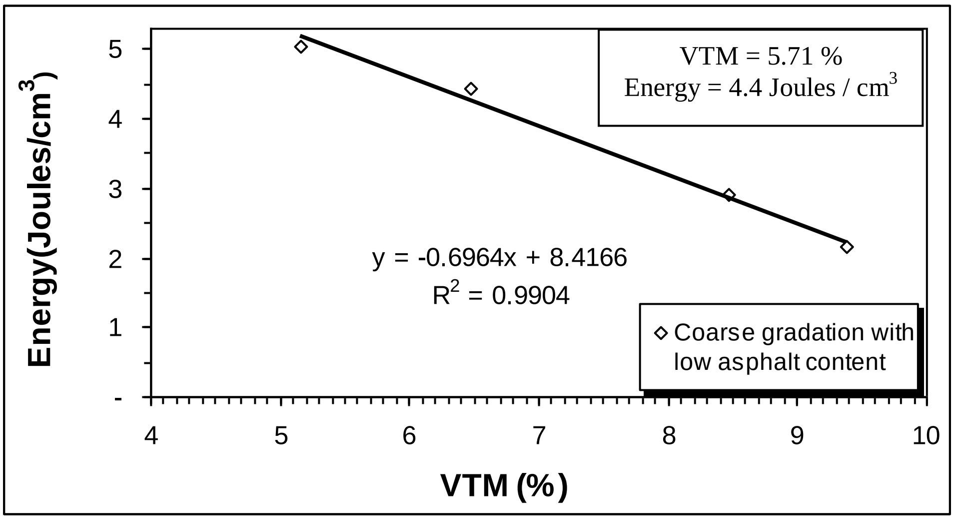

Figure A7 Graph for coarse graded tall specimen with low asphalt content, replicate 1

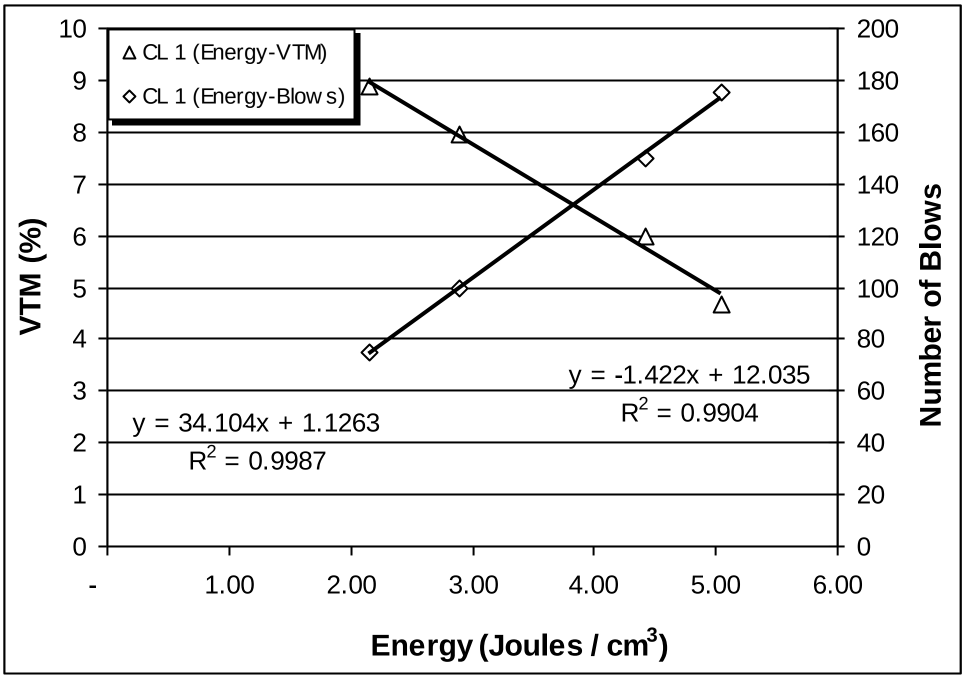

Figure A8 Graph for coarse graded tall specimen with low asphalt content, replicate 1

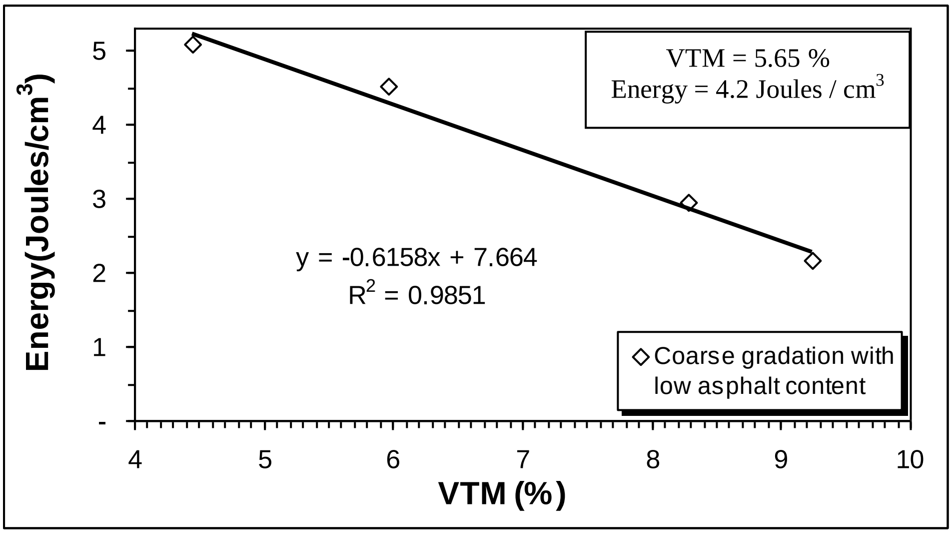

Figure A9 Graph for coarse graded tall specimen with low asphalt content, replicate 2

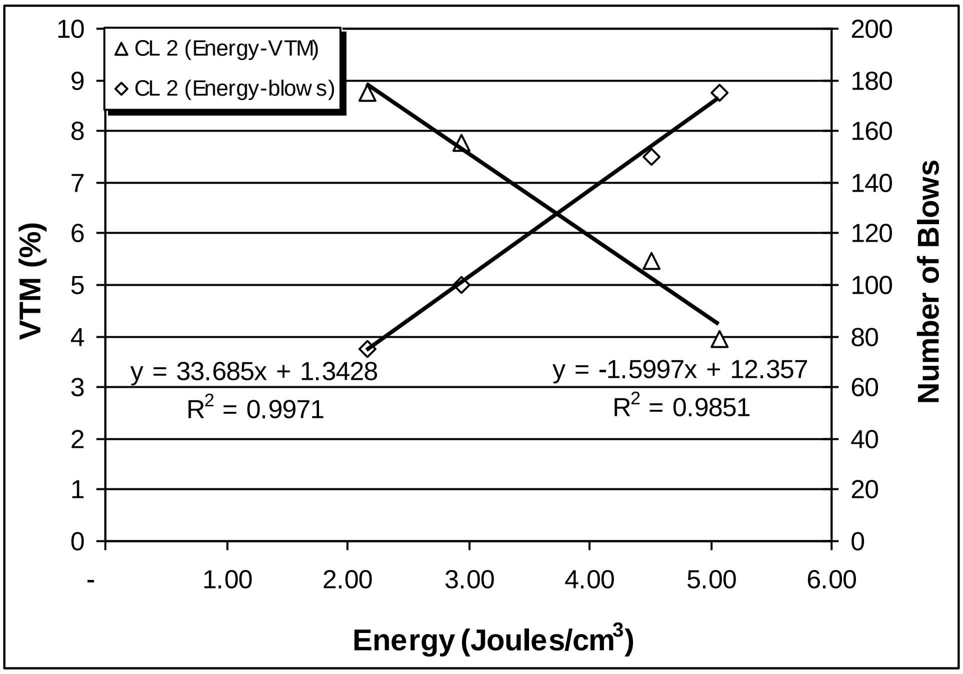

Figure A10 Graph for coarse graded tall specimen with low asphalt content, replicate 2

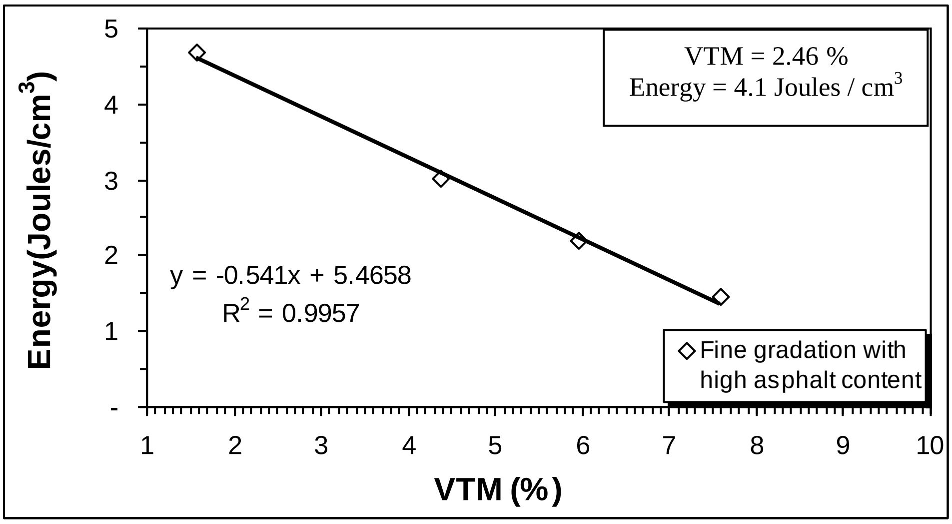

Figure A11 Graph for fine graded tall specimen with high asphalt content, replicate 2

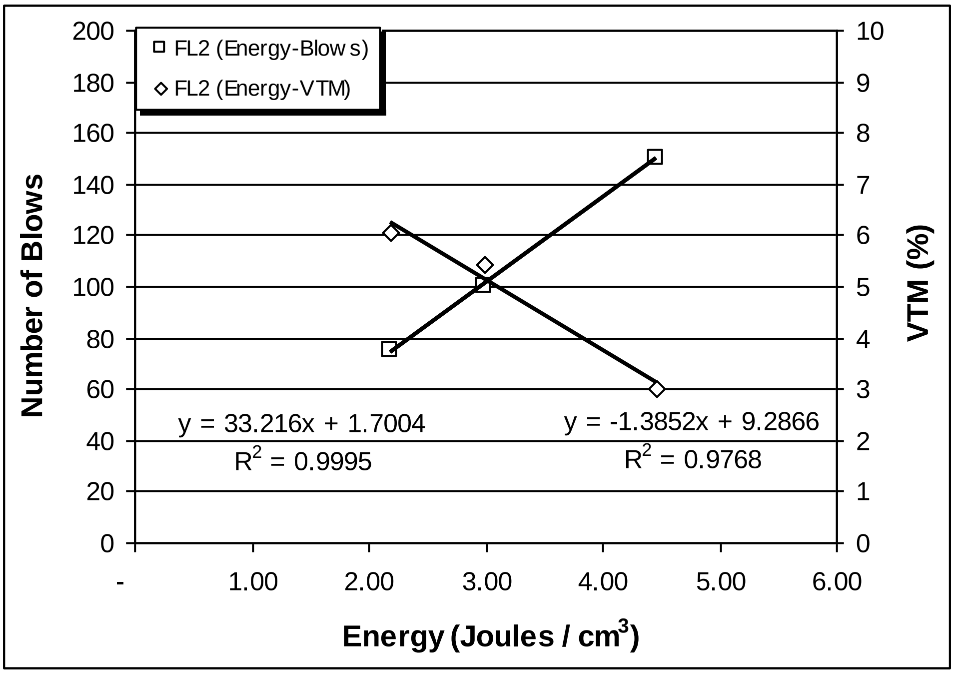

Figure A12 Graph for fine graded tall specimen with low asphalt content, replicate 2

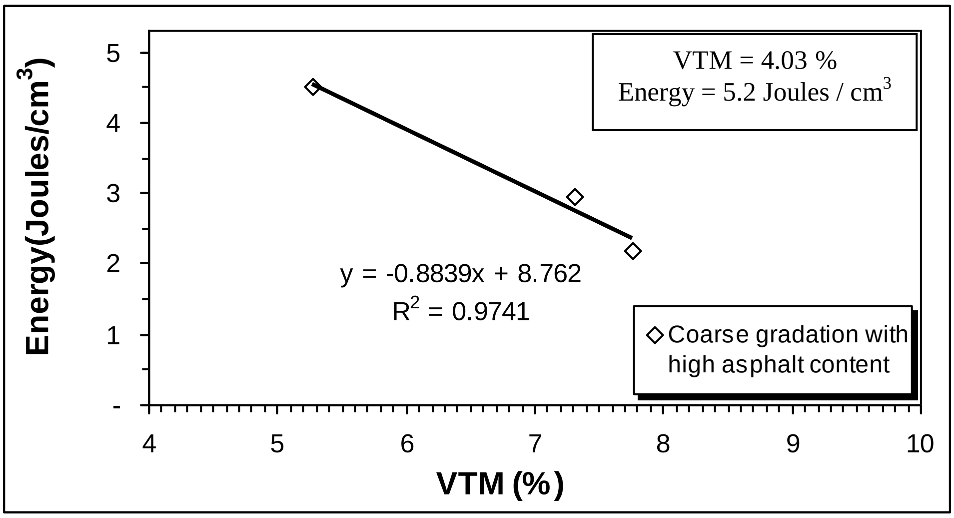

Figure A13 Graph for coarse graded tall specimen with high asphalt content, replicate 2

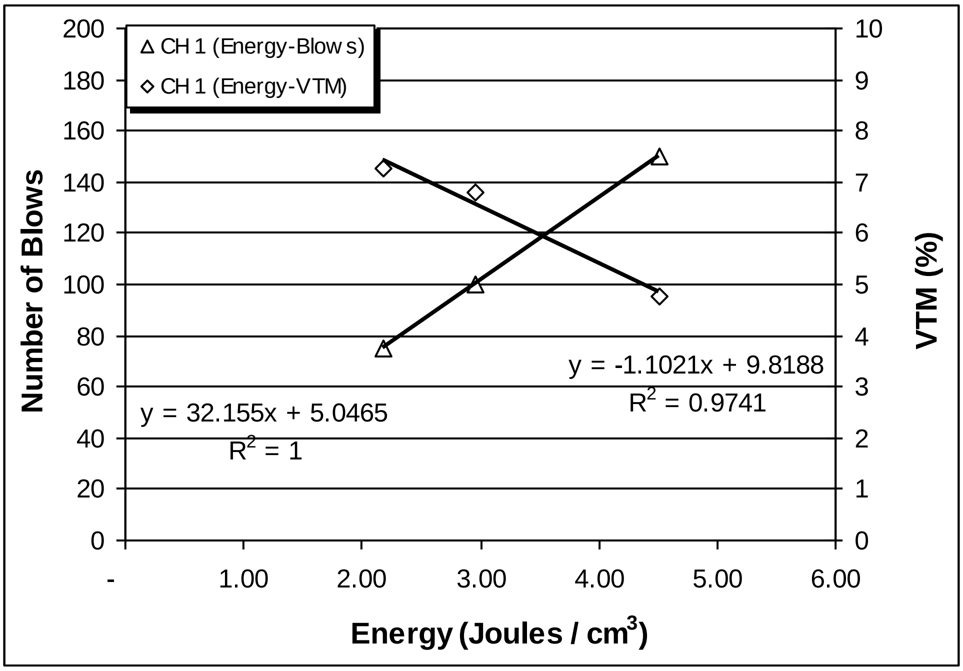

Figure A14 Graph for coarse graded tall specimen with high asphalt content, replicate 1

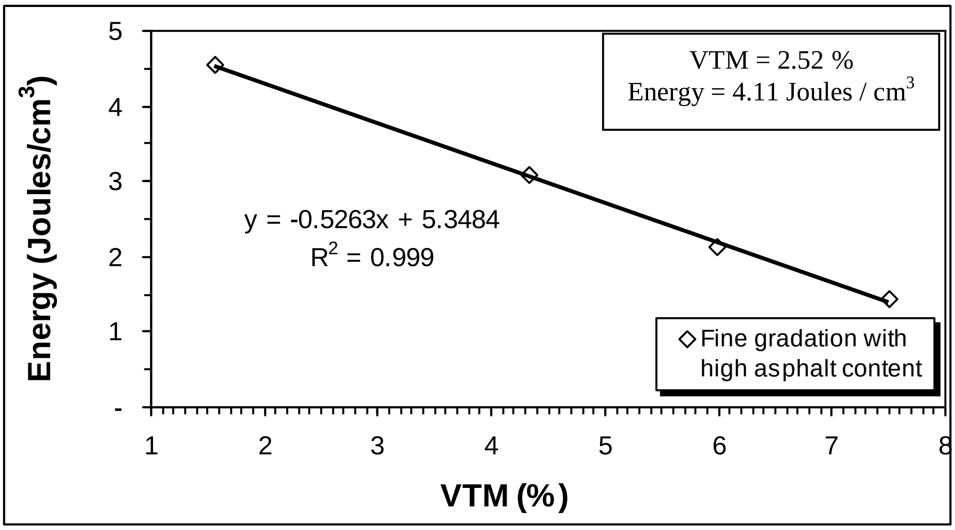

Figure A15 Graph for fine graded tall specimen with high asphalt content, replicate 1

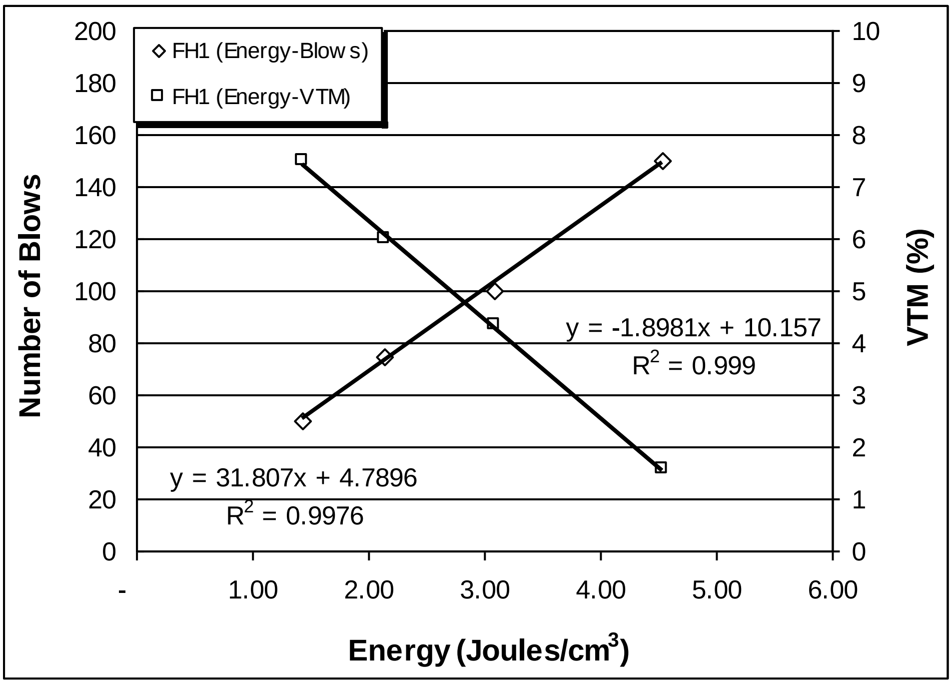

Figure A16 Graph for fine graded tall specimen with high asphalt content, replicate 1

APPENDIX B

NUMBER OF BLOWS AND VTM CALCULATION FOR MODIFIED MARSHALL SPECIMENS

Table B1 Comparison between no. of blows for modified Marshall specimens calculated through two different procedures

<table>
<tr><th>Mix Type</th><th>No. of Blows for Tall specimens calculated from Energy values of short Specimens (Y)</th><th>No. of Blows for Tall specimens calculated from VTM values of short Specimens (Y)</th></tr>
<tr><td rowspan="3">FH1</td><td>137.8</td><td>132.5</td></tr>
<tr><td colspan="2">y = 31.807x + 4.7896 (Figure 4.1)</td></tr>
<tr><td>x = 4.18 (From Table 4.3)</td><td>x=4.02 (From Figure A15)</td></tr>
<tr><td rowspan="3">FL2</td><td>140.6</td><td>136.0</td></tr>
<tr><td colspan="2">y = 33.216x + 1.7004 (Figure 4.1)</td></tr>
<tr><td>x = 4.18 (From Table 4.3)</td><td>x=4.04 (From Figure A11)</td></tr>
<tr><td rowspan="3">FH2</td><td>135.4</td><td>133.9</td></tr>
<tr><td colspan="2">y = 30.761x + 6.7559 (Figure 4.1)</td></tr>
<tr><td>x = 4.18 (From Table 4.3)</td><td>x=4.13 (From Figure A1)</td></tr>
<tr><td rowspan="3">FL1</td><td>139.1</td><td>133.7</td></tr>
<tr><td colspan="2">y = 32.362x + 3.7589 (Figure 4.1)</td></tr>
<tr><td>x = 4.18 (From Table 4.3)</td><td>x=4.02 (From Figure A3)</td></tr>
<tr><td rowspan="3">CL2</td><td>139.9</td><td>142.2</td></tr>
<tr><td colspan="2">y = 33.685x + 1.3428 (Figure 4.1)</td></tr>
<tr><td>x = 4.11 (From Table 4.3)</td><td>x=4.18 (From Figure A9)</td></tr>
<tr><td rowspan="3">CL1</td><td>143.8</td><td>152.4</td></tr>
<tr><td colspan="2">y = 34.104x + 1.1263 (Figure 4.1)</td></tr>
<tr><td>x = 4.18 (From Table 4.3)</td><td>x=4.44 (From Figure A7)</td></tr>
<tr><td rowspan="3">CH1</td><td>139.5</td><td>172.2</td></tr>
<tr><td colspan="2">y = 32.155x + 5.0465 (Figure 4.1)</td></tr>
<tr><td>x = 4.18 (From Table 4.3)</td><td>x=5.20 (From Figure A13)</td></tr>
<tr><td rowspan="3">CH2</td><td>137.7</td><td>169.2</td></tr>
<tr><td colspan="2">y = 32.126x + 5.5512 (Figure 4.1)</td></tr>
<tr><td>x = 4.11 (From Table 4.3)</td><td>x=5.09 (From Figure A5)</td></tr>
</table>

Table B2 VTM values comparison between standard and modified Marshall specimens calculated through different procedures

Mix Type	Blows	Measured VTM Values for Tall Specimens (%)	Calculated VTM Values for Tall Specimens (%)	Measured VTM Values for Short Samples (%)
FH1	50	7.510	2.213	2.526
	75	5.985	y = -1.8981x + 10.157 x = 4.18 (Table 4.3)	
	100	4.335		
	150	1.561		
FL2	75	6.074	3.465	3.658
	100	5.428	y = -1.3852x + 9.287 x = 4.18 (Table 4.3)	
	150	3.017		
FH2	50	7.597	2.379	2.462
	75	5.950	y = -1.8404x + 10.081 x = 4.18 (Table 4.3)	
	100	4.369		
	150	1.585		
FL1	75	6.308	3.509	3.762
	100	5.743	y = -1.5323x + 9.937 x = 4.18 (Table 4.3)	
	150	2.905		
CL2	75	8.750	5.796	5.656
	100	7.777	y = -1.5997x + 12.356 x = 4.11 (Table 4.3)	
	150	5.459		
	175	3.952		
CL1	75	8.887	6.092	5.715
	100	7.962	y = -1.4221x + 12.035 x = 4.18 (Table 4.3)	
	150	5.972		
	175	4.658		
CH1	75	7.259	5.211	4.031
	100	6.807	y = -1.1021x + 9.819 x = 4.18 (Table 4.3)	
	150	4.770		
CH2	75	7.028	5.061	3.983
	100	6.590	y = -1.0817x + 9.532 x = 4.11 (Table 4.3)	
	150	4.595		

APPENDIX C

GRADATIONS USED IN DIFFERENT SPECIMENS PREPARED FOR REPEATED CREEP TESTS

Table C1 Details of the gradation used for the modified Marshall compacted coarse specimens

Sieve (mm)	Cumulative Retained (%)	Weight in Grams for Modified Marshall Test Specimens (COARSE)							
		Combination 1		Combination 2		Combination 3		Combination 4	
		Natural 100 %	Recycled 0%	Natural 75 %	Recycled 25 %	Natural 50 %	Recycled 50 %	Natural 25 %	Recycled 75 %
19.000	0	0	0	0	0	0	0	0	0
12.500	17	517	0	387	129	257	257	128	384
9.500	13	396	0	296	99	197	197	98	293
4.750	30	913	0	683	228	454	454	226	677
2.000	15	456	0	341	114	227	227	113	338
0.425	15	456	0	341	114	227	227	113	338
0.180	4	122	0	91	30	60	60	30	90
0.075	2	61	0	46	15	30	30	15	45
Pan	4	122	0	91	30	60	60	30	90
Total		3,043	0	2,276	759	1,512	1,512	752	2,256
AC (%)		4.9		5.2		5.5		6.0	

Table C2 Details of the gradation used for the modified Marshall loose coarse samples

Sieve (mm)	Cumulative Retained (%)	Weight in Grams for Rice Test Specimens (COARSE)							
		Combination 1		Combination 2		Combination 3		Combination 4	
		Natural 100 %	Recycled 0%	Natural 75 %	Recycled 25 %	Natural 50 %	Recycled 50 %	Natural 25 %	Recycled 75 %
19.000	0	0	0	0	0	0	0	0	0
12.500	17	243	0	181	60	120	120	60	180
9.500	13	185	0	139	46	92	92	46	137
4.750	30	428	0	320	107	213	213	106	317
2.000	15	214	0	160	53	106	106	53	159
0.425	15	214	0	160	53	106	106	53	159
0.180	4	57	0	43	14	28	28	14	42
0.075	2	29	0	21	7	14	14	7	21
Pan	4	57	0	43	14	28	28	14	42
Total		1,427	0	1,067	356	709	709	353	1,058
AC (%)		4.9		5.2		5.5		6.0	

Table C3 Details of the gradation used for the modified Marshall compacted fine specimens

Sieve (mm)	Cumulative Retained (%)	Weight in Grams for Modified Marshall Test Specimens (FINE)							
		Combination 1		Combination 2		Combination 3		Combination 4	
		Natural 100 %	Recycled 0%	Natural 75 %	Recycled 25 %	Natural 50 %	Recycled 50 %	Natural 25 %	Recycled 75 %
9.500	0	0	0	0	0	0	0	0	0
4.750	30	857	0	640	213	422	422	626	209
2.000	15	581	0	434	145	286	286	425	142
0.425	15	765	0	571	190	377	377	559	186
0.180	4	367	0	274	91	181	181	268	89
0.075	2	184	0	137	46	90	90	134	45
Pan	4	306	0	228	76	151	151	224	75
Total		3,059	0	2,284	761	1,507	1,507	2,237	746
AC %		4.4		4.8		5.8		6.8	

Table C4 Details of the gradation used for the modified Marshall loose fine specimens

Sieve (mm)	Cumulative Retained (%)	Weight in Grams for Rice Test Specimens (FINE)							
		Combination 1		Combination 2		Combination 3		Combination 4	
		Natural 100 %	Recycled 0%	Natural 75 %	Recycled 25 %	Natural 50 %	Recycled 50 %	Natural 25 %	Recycled 75 %
9.500	0	0	0	0	0	0	0	0	0
4.750	30	402	0	300	100	198	198	98	294
2.000	15	272	0	203	68	134	134	66	199
0.425	15	359	0	268	89	177	177	87	262
0.180	4	172	0	128	43	85	85	42	126
0.075	2	86	0	64	21	42	42	21	63
Pan	4	143	0	107	36	71	71	35	105
Total		1,434	0	1,071	357	707	707	350	1049
AC %		4.4		4.8		5.8		6.8	

APPENDIX D

OPTİMUM ASPHALT CONTENT CALCULATION FOR MODIFIED MARSHALL SPECİMENS

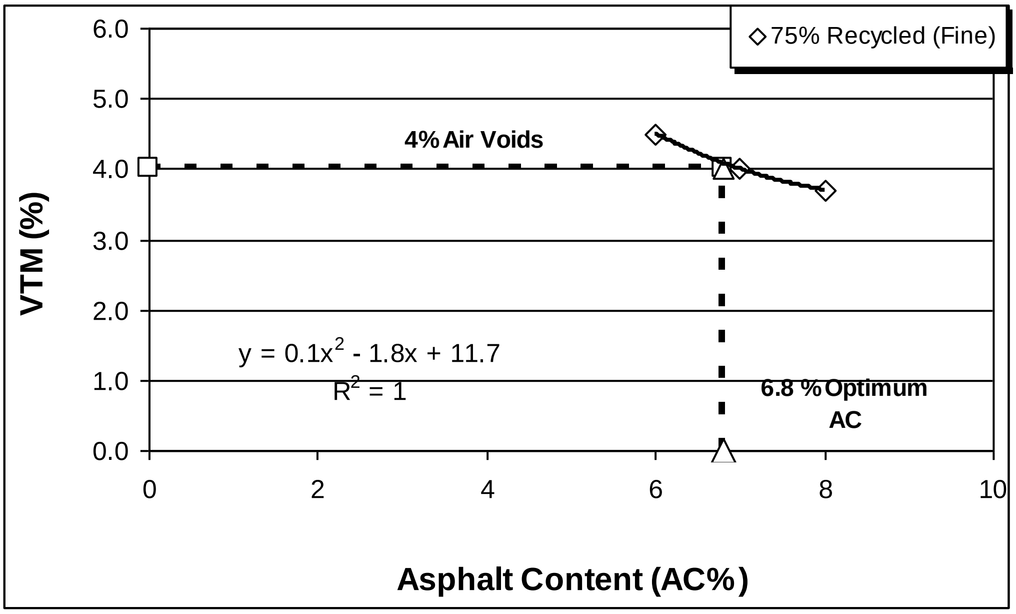

Figure D1 Optimum asphalt content calculation for fine graded tall specimen with 75 % recycled cement concrete

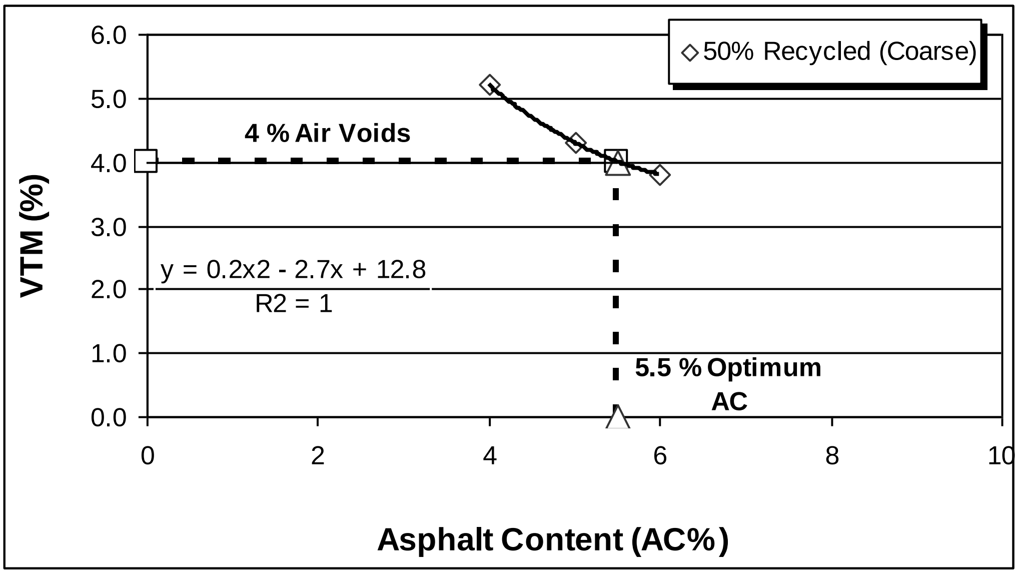

Figure D2 Optimum asphalt content calculation for coarse graded tall specimen with 50 % recycled cement concrete

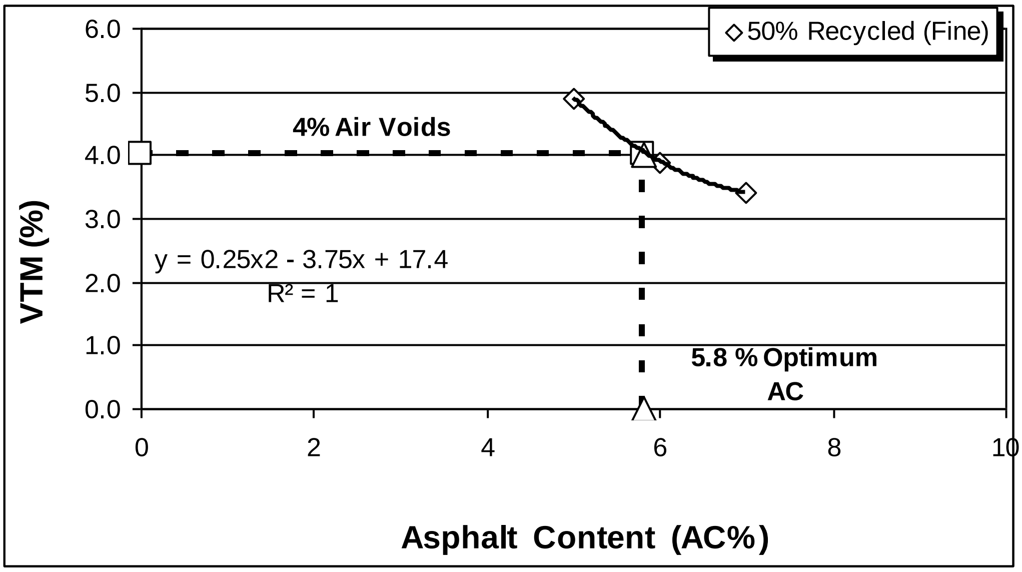

Figure D3 Optimum asphalt content calculation for fine graded tall specimen with 50 % recycled cement concrete

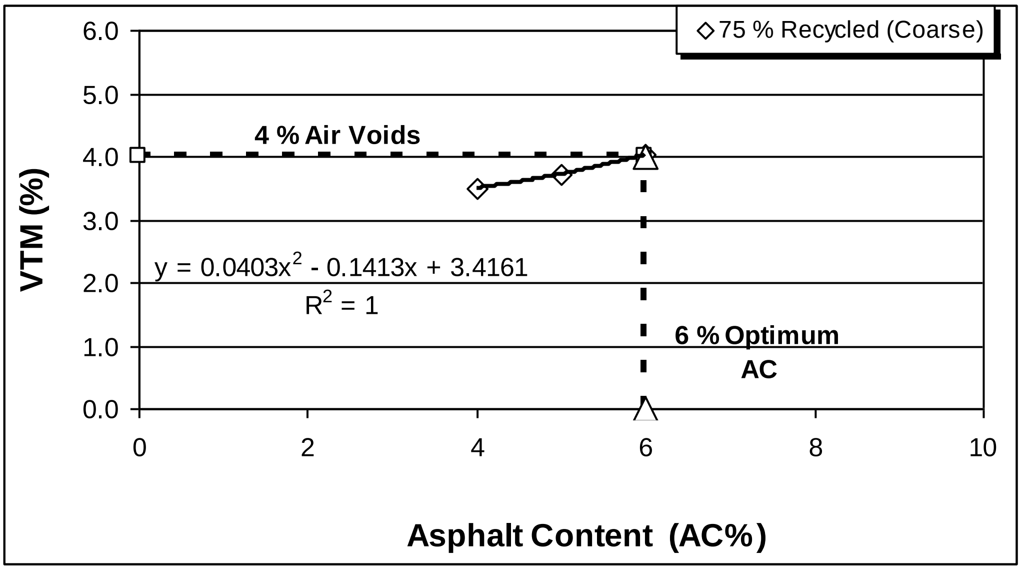

Figure D4 Optimum asphalt content calculation for coarse graded tall specimen with 75 % recycled cement concrete

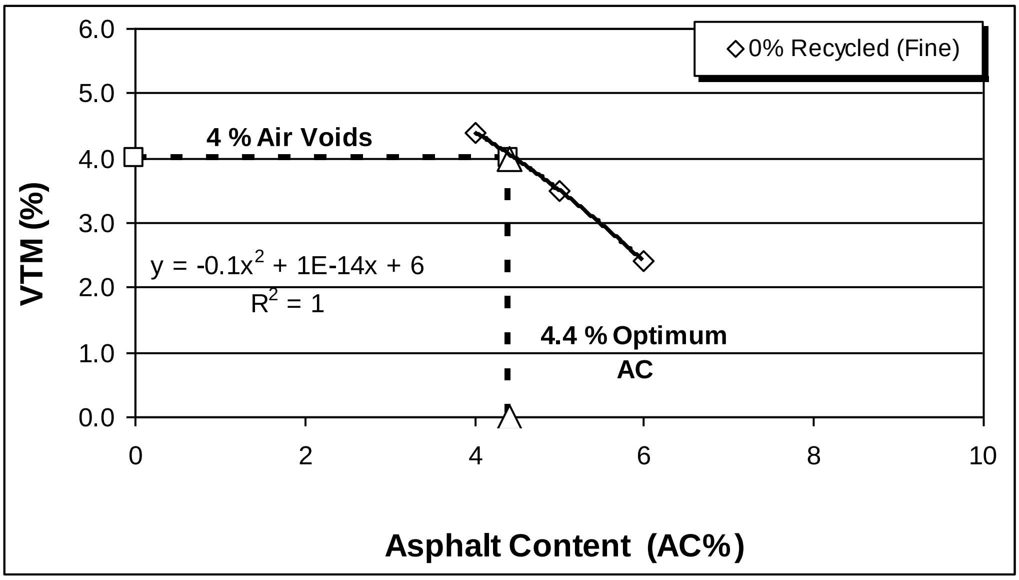

Figure D5 Optimum asphalt content calculation for fine graded tall specimen with 0 % recycled cement concrete

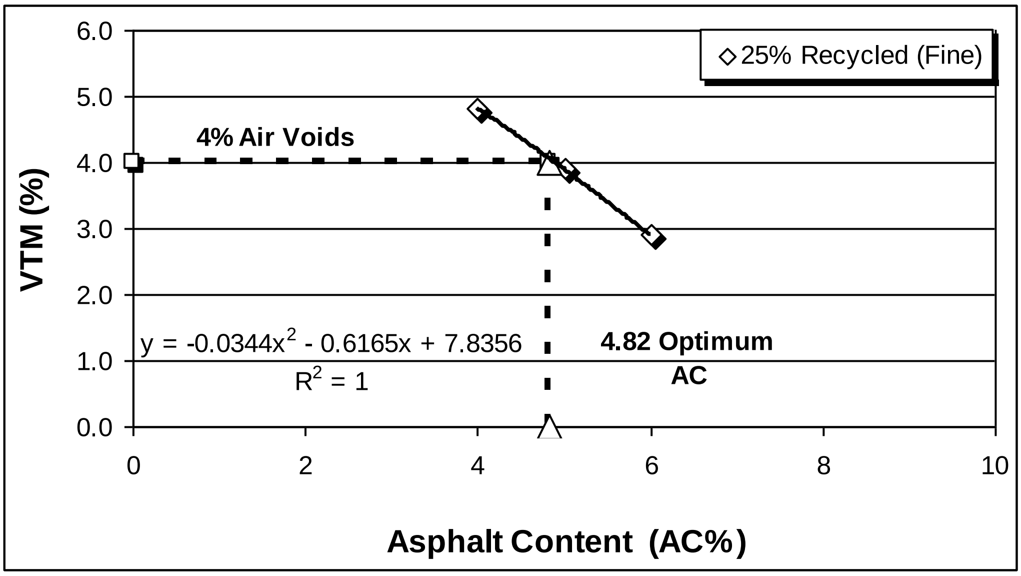

Figure D6 Optimum asphalt content calculation for fine graded tall specimen with 25 % recycled cement concrete

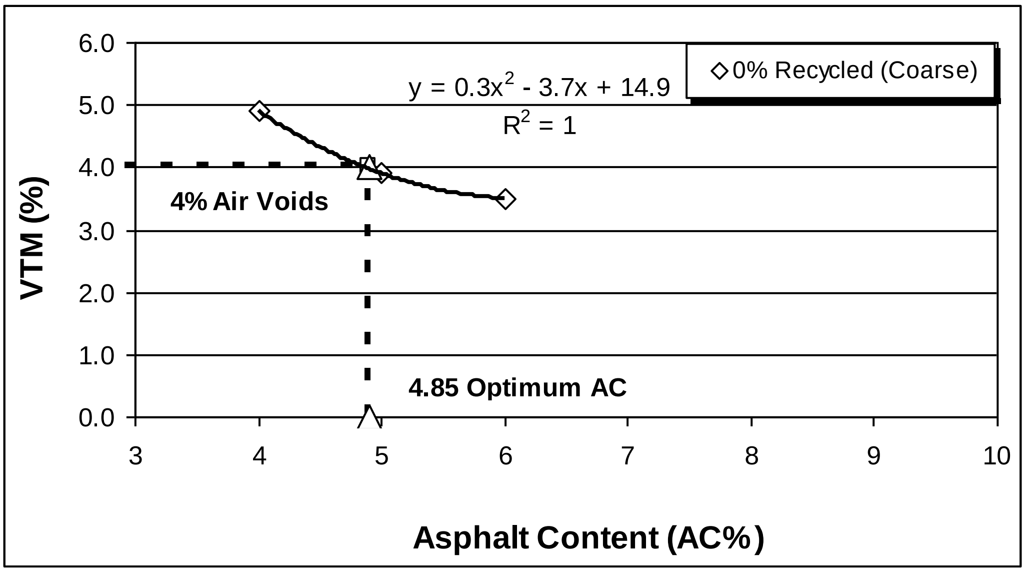

Figure D7 Optimum asphalt content calculation for coarse graded tall specimen with 0 % recycled cement concrete

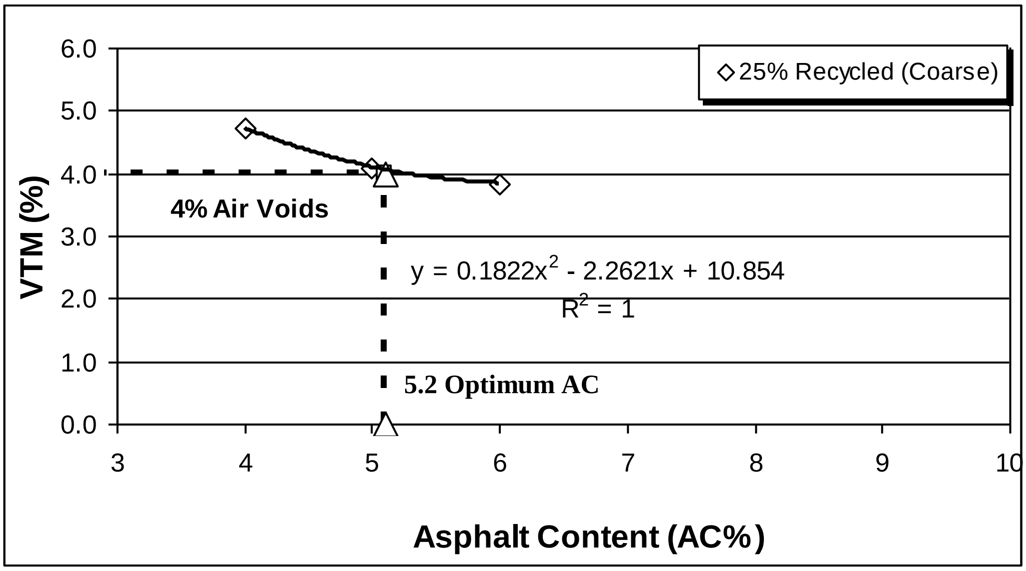

Figure D8 Optimum asphalt content calculation for coarse graded tall specimen with 25 % recycled cement concrete