THE EFFECTS OF GEOSYNTHETICS ON MITIGATION OF RUTTING IN FLEXIBLE PAVEMENTS

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ABSTRACT

THE EFFECTS OF GEOSYNTHETICS ON MITIGATION OF RUTTING IN FLEXIBLE PAVEMENTS

One of the more important problems associated with asphalt concrete pavements is rutting. This phenomenon is mostly illustrated as a surface depression in the wheel paths due to load-induced conditions as well as improper mix design or insufficient compaction of hot-mix asphalt (HMA). Some of the latest techniques for mitigating the severity and/or decreasing of permanent deformations on asphalt concrete pavements, in other words, preventing the occurrences of wheel path ruts include incorporating geosynthetic products into pavement structure. The purpose of this thesis is to study the benefits of applying geosynthetic reinforcement for rutting mitigation in asphalt concrete pavements. Geosynthetics are referred to herein as geogrids, fabrics, or composites. The procedure suggests installing the geosynthetic material in an existing, in-service flexible pavement with an asphalt tack coat and then overlaying with a specified thickness of HMA.

Rut depth measurements will be taken from a “Wheel Tracking” test using an available Hamburg Wheel Tracking Device (HWTD). The HWTD measures the combined effects of rutting and moisture damage by rolling a rubber coated wheel across the surface of an asphalt concrete specimen that is immersed in heated water.

As a conclusion, Rutting is one of the main distress types to lead to pavement failure and is difficult to track and simulate with deformation/strain measurements in majority of materials of asphalt concrete. The purpose of this research study is to investigate the effectiveness of geosynthetics in mitigation of rutting in asphalt concrete pavements. Moreover, making a cost effective analysis of geosynthetic reinforced flexible or composite pavements with comparisons either included unreinforced pavements.

The expected contribution of this study to the state of the art is to present a new laboratory study and its findings to help better understand rutting occurrence in asphalt

concrete layer and its mitigation with the use of geosynthetic reinforcement. The aim of this work is to study the effects of applying geosynthetics as reinforcement in improving the rutting resistance of the asphalt pavement.

ÖZET

ESNEK ÜSTYAPILARDA GEOSENTETİKLERİN KULLANIMININ TEKERLEK İZİ OLUŞUMUNUN AZALTIMINA ETKİSİ

Asfalt beton yol üstyapılarıyla bağdaştırılmış önemli problemlerden biri tekerlek izinde oturmadır. Bu olgu genellikle yüke bağlı sebeplerden, uygun olmayan karışım dizaynından ve sıcak asfaltın yetersiz bir şekilde kompaksionundan kaynaklanan tekerlek izlerindeki yüzey çökmesidir. Bozulmaları engellemek ve asfalt yolların kaplama tabakasındaki kalıcı deformasyonları azaltmak için, son yıllarda kullanılan teknikleren biri; geosentetik ürünlerin yol kaplaması içine yerleştirilerek tekerlek izi oluşumunun engellenmesidir. Bu tezin amacı, asphalt beton üstyapılarda oluşan tekerlek izlerinin azaltılması için geosentetik ile güçlendirme uygulamalarının faydalarını araştırmaktır. Bu çalışmada geogridler, tekstiller veya kompozitler geosentetik anlamında kullanılmaktadır. Geosentetik malzemenin varolan, kullanımda olan bir yol üstyapısında (esnek, rijit ya da kompozit) bir asfalt astarın (yapıştırıcı) kullanımı ve ardından tanımlanmış bir kalınlıkta Sıcak Karışım Asfalt'ın serilmesi tavsiye edilen uygulama prosedürüdür.

Tekerlek izi derinlik ölçümleri uygun bir Hamburg tipi Tekerlek izi test cihazı ile yapılacaktır. Hamburg tekerlek izi test cihazı, tekerlek izi oluşumu ile nem tahribatını kombine etkilerini, ısıtılmış suya konmuş asfalt betonu numunesinin üzerinde lastik kaplamalı tekerlek geçirerek ölçmektedir.

Sonuç olarak, tekerlek izinde oturma, yol kaplamasının bozulmasına yol açan ana sorunlardan biri olup, asfaltın muhtevasındaki malzemelerin çoğunda oluşan kalıcı bozulma şekil değiştirme ölçümleri ile simüle edilmesi ve ölçülmesi zor olan bir problemdir. Bu çalışmanın amacı asfalt beton yollardaki tekerlek izlerinin azaltımasında geosentetik malzemenin etkinliğini araştırmaktır.

Bu çalışmanın güncel bilgiye beklenen katkısı, yeni labaratuvar çalışmalarını yapıp sunmak, sonuçların asfalt beton tabakalarında tekerlek izi oluşumunun daha iyi

anlaşılmasınını sağlanması ve geosentetik güçlendirmesi ile bu etkinin azaltılmasıdır. Bu çalışmanın amacı, asfalt beton yollarda tekerlek izi oluşumunda dayanımındaki artışta, güçlendirme olarak geosentetik uygulamasının etkileri üzerinde çalışmaktır.

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LIST OF SYMBOLS/ ABBREVIATIONS

AASHTO	American Association of State Highway Transportation Officials
AC	Asphalt Concrete
ACP	Asphalt Concrete Pavement
BCR	Base Course reduction Ratio
BST	Bituminous Surface Treatments
CBR	California Bearing Ratio
CRC	Continuously Reinforced Concrete Pavement
CST-M&P	Construction Division, Materials & Pavements Section
DGAC	Dense Graded Asphalt Concrete
EN	European Norms
FDR	Full-Depth Reclamation/recycling
FHWA	Federal Highway Administration
GGAC	Gap Graded Asphalt Concrete
GMA	Geosynthetics Materials Association
HMA	Hot Mix Asphalt
HMAC	Hot Mix Asphalt Concrete
HWTD	Hamburg Wheel Tracking Device
JPCP	Jointed Plain (Non-Reinforced) Concrete Pavement
LCCA	Life Cycle Cost Analysis
M-E	Mechanistic-Empirical
MLE	Multi-layered Elastic
OECD	Organization for Economic Cooperation and Development
OGAC	Open Graded Asphalt Concrete
OGFC	Open-graded Friction Course
PCC	Portland Cement Concrete
PDE	Pavement Design Engineer
PI	Plasticity Index
PID-controller	Proportional–Integral–Derivative controller
PMS	Pavement Management System
PA	Polyamide
PE	Polyethylene

PET	Polyethylene
PP	Polypropylene
PVC	Polyvinyl Chloride
RAP	Reclaimed Asphalt Pavement
SHRP	Strategic Highway Research Program
SMA	Stone Mastic (Matrix) Asphalt
TBR	Traffic Benefit Ratio
TS-EN	European Norms to Turkish Standards
UTA	Ultra Thin Asphalt
UTAC	Ultra Thin Asphalt Concrete

1. INTRODUCTION

1.1. General and Background Information

Flexible pavements under the application of freight traffic are exposed to high magnitudes of stress and strain conditions. So many pavement distresses occur due to the fact of phenomena, which changes in stress and strain conditions. Rutting is the permanent deformation in the wheel path occurring as a result of accumulated permanent strains, which are often difficult to measure magnitudes due to the complex heterogeneous nature of asphalt concrete materials (Uzan, 2004). Therefore, owing to its complicated occurrence mechanism, rutting prediction becomes more difficult under repeated axial loading. Besides, asphalt concrete under the influence of heavy loading and high temperature is a viscoelastic material due to the properties of asphalt binder. Due to the temperature and humidity of the material which differ in repeated load cycles; the mechanisms of rut formation is complex and highly dependent on the types of materials used, applied traffic data and the climatic effects (Laurinavičius and Oginskas, 2006).

Rutting is also a serious safety issue for road users. Wheel path ruts are treated as dangerous defects since they might cause danger for traffic, especially when the surface is wet. By considering its effects on driving safety and driver comfort, many countries define different allowable rut depths according to highway road failure criteria in their specifications and standards. The problem of rutting can be effectively addressed by decreasing vertical stress due to applied wheel load. To minimize vertical stress/strain on top of each pavement layer, the solution is to endeavor reinforcement in the base course to increase its elastic modulus (Archilla and Madanat, 2000).

Geosynthetics have been used for many purposes in asphalt concrete pavements such as for the reduction of reflective cracks in HMAs. Such an application can prevent moisture intrusion into the underlying pavement structure. The geosynthetic material is being used to prevent reflective cracking by acting as an interlayer between the old pavement and the overlay (Ling and Liu, 2001). Geosynthetics also are widely used in new road constructions in order to extend life cycle of pavements by inhibiting of distresses such as cracking. Besides, geosynthetics can be used in various ways to mitigate rutting of Asphalt Concrete (AC) pavements

Reinforcing pavements is not an absolutely new phenomenon; it has been sufficiently well investigated. However, the majority of investigations have been concentrated on reinforcing the road base and the embankment by geosynthetics, Research into reinforcing asphalt concrete generally has been concerned with the prevention of reflection cracking. Some researchers has been studying on preventing of other pavement distresses such as rutting, water bleeding or pumping of ground water out of surface by using geosynthetics. Moreover, very little research has been conducted into the impact of reinforced asphalt concrete on the formation of plastic and shear strains in asphalt concrete.

1.2. Objective

In order to develop a better understanding of geosynthetic–reinforced flexible pavements, a number of objectives were developed and executed.

i). Convey Extensive Literature Review: A review of literature concerning to all aspects of the current work will be performed to establish the current state of practice in each area.

ii). Evaluate Geosynthetic Performance Mechanism in Flexible Pavements: Geogrids and geotextiles are utilized to improve the rutting resistance performance of flexible pavement structures under repeated loads

iii). Perform Hamburg Wheel Tracking Device (HWTD): HWT Device will be tested under different test conditions such as different slab thicknesses and different test conditions.

1.3. Scope of the Study

This thesis study is organized as follows, and a brief summary of each section has been included for reference.

Part 2 describes Pavement topic with its basic outlines. This part includes pavement types, design concept of pavements, pavement materials and extensive explaining of concerned type of pavement, flexible pavements. What is more, this part provides a detailed description of pavement management with its all outlines. Pavement preservation concept and pavement evaluation criteria related our thesis concept is investigated in this part.

Part 3 examines flexible pavement distresses. One of the important flexible pavement distresses, rutting is focused in its all details.

Part 4 outlines the extensive details about geosynthetics and their usage in pavement systems. Geosynthetic reinforcement is largely discussed in this part.

Part 5 explains the main concept and aim of laboratory studies of current issue. Key questions and scope of tests are summarized with general statements.

Part 6 provides detailed description of rutting test for specially prepared testing slabs. All facets of test section are imposed in this part. Moreover, includes analyzing of test result and basic comments for each test results.

Part 7 states conclusion, short summary of study and recommendations for future studies. Moreover, this part explains the contribution of this study to the state of the art of pavement engineering.

2. PAVEMENTS

2.1. Introduction

Pavement is an engineered structure in contact with the earth for the transport of people and goods such as pedestrians, personal vehicles (buses, cars, and bicycles), freight vehicles (trucks), and warehouse vehicles (lift trucks). Otherwise, rail structures (railroads, streetcars), aircrafts and spacecraft are other users of pavements as a transportation vehicle.

All engineering structures have some design principles and end product features. Pavement design also aims to consider some principles which are generally valid for other engineering structures. These principles are listed as following:

- Safe
- Cost Effective
- Constructability
- Low maintenance
- Long Lasting
- Durability

The requirements of a pavement are the reflection of design principles into end product. Pavements should meet some requirements such as; load support, which means to provide sufficient thickness to distribute the wheel load stresses to a safe value on the sub-grade soil. Other is adequate coefficient of friction to prevent skidding of vehicles. What is more, smooth surface to provide comfort to road users even at high speed is another important requirement. Noise control is to provide to produce least noise from moving vehicles, thanks to having of drainage feature to protect sub-grade soil and cost efficiency for long design life with low maintenance cost.

2.2. Design of Asphalt Pavements

Effective pavement design is one of the more important principles of project design. The pavement is the portion of the highway which is most obvious to the drivers. The condition and adequacy of the highway is often judged by the smoothness or roughness of the pavement. Bad pavement conditions can result in increased user costs and travel delays, braking and fuel consumption, vehicle maintenance repairs and probability of increased crashes. The pavement life is substantially affected by the number of heavy load repetitions applied, such as single, tandem, tridem and quad axle trucks, buses, tractor trailers and equipment. A properly designed pavement structure will take into account the applied loading. (Mass. DOT, 2006)

To select the suitable pavement type and properly design a pavement structure, the Designer must obtain information and input from the Pavement Management System (PMS), the Pavement Design Engineer (PDE), Research and Materials. The Designer must also apply sound engineering judgment.

An important advancement in highway engineering is the realization and presentation that structural design of asphalt pavements familiar to the matter of designing any complex engineering structure. When asphalt pavement first introduced, determining the suitable thickness was a matter empiricism and opinion based on experience.

There is no standard thickness for a pavement. Needed total thickness is determined by engineering design procedures. Factors considered in the design procedures are the following:

i). Traffic to be served initially over the design service life of the pavement.
ii). Strength and other properties of subgrade.
iii). Strength and other effective characteristics of the materials available or chosen for the layers or courses in the total asphalt pavement structure.
iv). Any special factor concerned to the road being designed, such as environmental factors.

2.2.1. Structural Design Concepts

There are a lot of structural design concepts for flexible pavements. These concepts are substantially condensed to reach a model which provides long lasting feature to pavement under traffic loads and other related conditions such environmental. The chronologies for structural design concepts have advanced positively and the recent studies have generated more comprehensive concepts for structural design of pavements.

2.2.2. Empirical Design Method

An empirical approach is based on the results of experiments or experience. Generally, it needs a number of observations to be made in order to understand the relationships between input variables and outcomes. It is not necessary to sticky establish the scientific basis for the relationships between variables and outcomes as long as the limitations with such an approach are recognized. Specifically, it is not prudent to use empirically derived relationships to describe phenomena that occur outside the range of the original data used to develop the relationship. In some cases, it is much more expedient to rely on experience than to quantify the exact cause and effect of certain phenomena. Many pavement design procedures use an empirical approach. This means that the relationship between design inputs (e.g., loads, materials, layer configurations and environment) and pavement failure were arrived at through experience, experimentation or a combination of both.

Empirical design methods can range from extremely simple to quite complex. The simplest approaches specify pavement structural designs based on what has worked in the past. For example, local governments in US often specify city streets to be designed using a given cross section (e.g., 100 mm (4 inches) of HMA over 150 mm (6 inches) of crushed stone) because they have found that this cross section has produced adequate pavements in the past. More complex approaches are usually based on empirical equations derived from experimentation.

2.2.3. Mechanistic-Empirical (M-E) Design Method

Mechanics is the science of motion and the action of forces on bodies. Thus, a mechanistic approach wants to explain the thing only by using physical causes. In pavement design, the phenomena are the stresses, strains and deflections within a pavement structure and the physical causes are the loads and material properties of the pavement structure. The relationship between these phenomena and their physical causes is typically described using a mathematical model. Various mathematical models can be (and are) used; the most common is a layered elastic model. (Perkins, 2001)

Apart from this mechanistic approach, empirical elements are used when defining what value of the stresses, strains and deflections result in pavement failure. The relationship between physical event and pavement failure is described by empirically derived equations that compute the number of loading cycles to failure.

The basic advantages of a mechanistic-empirical pavement design method over a purely empirical one are: M-E pavement design can be used for both existing pavement rehabilitation and new pavement construction and accommodates changing load types. Moreover, it can better characterize materials allowing for better utilization of available materials and accommodation of new materials. Besides, M-E pavement design approach provides an improved definition of existing layer properties. Therefore, it uses material properties that relate better to actual pavement performance. M-E pavement design approach provides more reliable performance predictions. What is more, it better defines the role of construction M-E design concept accommodates environmental and aging effects on materials.

The advantage of a mechanistic-empirical approach is to provide a requirement to accurately characterize in situ material considering its structural condition (including subgrade and existing pavement structures). This is typically done by using a portable device (like a Falling Weight Deflectometer (FWD)) to make some field deflection measurements on a pavement structure to be overlaid. These measurements can then be input into equations to determine existing pavement structural support (often called "backcalculation") and the approximate remaining pavement life. This allows for a more realistic design for the given conditions.

Mechanistic-Empirical (M-E) analysis and design procedures for rutting assessment in pavements provide a wide range of evaluations. Factors that affecting rutting such as traffic, climate, materials, and pavement layered structure are the principal inputs for M-E pavement design process. The pavement failure criteria such as rutting, fatigue, roughness and thermal cracking are obtained from relating critical pavement responses to failure conditions through transfer functions. Figure 2.1 represents design process of M-E Pavement Design concept in a systematic demonstration.

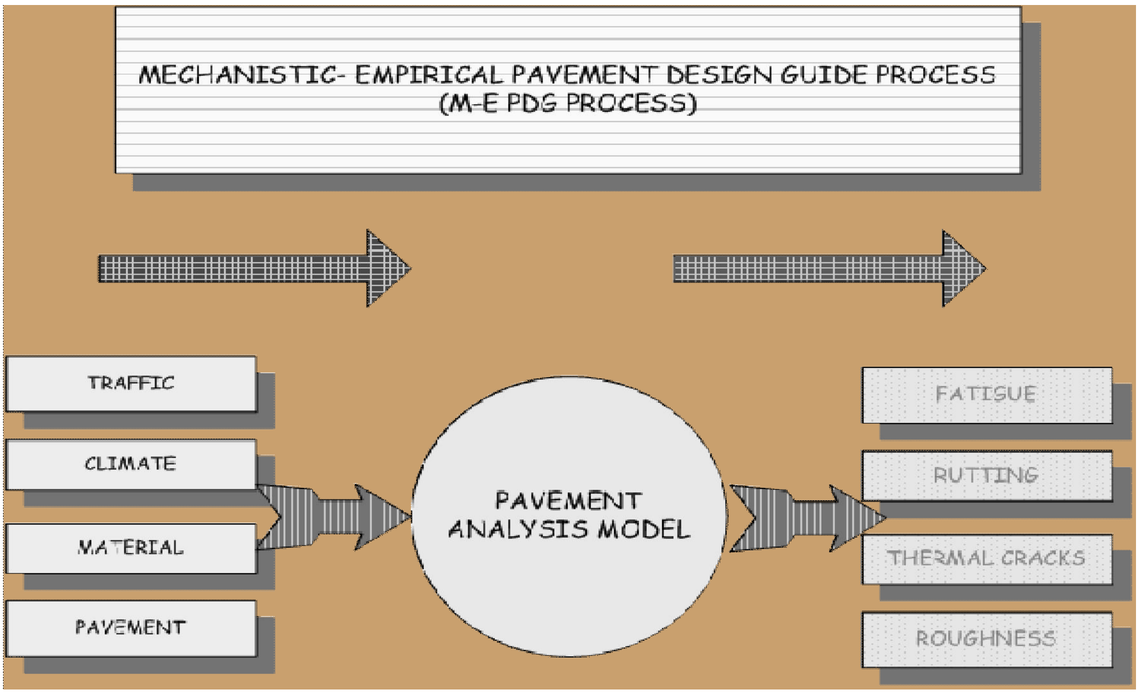

Figure 2.1. Representation of M-E Pavement Design Guide Process

2.3. Pavement Types

Basically, all hard surfaced pavement types can be categorized into two groups, flexible and rigid pavements. The FHWA (Federal Highway Administration) in the United States also identifies a third type of pavement, called a composite pavement.

3.3.1. Flexible Pavements

Flexible pavements are those which are surfaced with bituminous (or asphalt) materials. These types of pavements are called "flexible" since the total pavement structure "bends" or "deflects" due to traffic loads. A flexible pavement structure is generally composed of several layers of material with better quality materials on top where the intensity of stress from traffic loads is high and lower quality materials at the bottom where the stress intensity is low. Flexible pavements can be analyzed as a multilayer system under loading. A typical flexible pavement structure covers the surface course and underlying base and subbase courses. Each one of all layers contributes to distribution of loads and drainage.

A typical structural design results in a series of layers that gradually decrease in material quality with depth. When hot mix asphalt (HMA) is used as the surface course, it is the stiffest (as measured by resilient modulus) and may contribute the most (depending upon thickness) to pavement strength. The underlying layers are less stiff but they are still important to pavement strength as well as drainage and frost protection.

In flexible pavements material layers are usually aligned in order of decreasing load bearing capacity with the highest load bearing capacity material on the top and the lowest load bearing capacity material on the bottom. The typical flexible pavement structure consisting of:

i). Surface course: Surface course is the top layer of flexible pavement and the layer that comes in contact with traffic. It may be combination of one or several different HMA sublayers. These sublayers are overlaid according to required thickness to meet traffic loads.

ii). Base course: This is the layer directly below the surface course and generally consists of aggregate (either stabilized or unstabilized) or HMA. Base course provides structural support and distribute coming loads to the subbase course or subgrade whether subbase is not exist.

iii). Subbase course: This is the layer (or layers) under the base layer. A subbase is not always needed.

The material quality with depth gradually decreases in a series of layers for a typical flexible pavement design.

Figure 2.2. Typical section for a flexible pavement.

3.3.2. Rigid pavements

This type of pavement consists of a Portland Cement Concrete (PCC) surface course. Such pavements are principally “stiffer" than flexible pavements due to the high modulus of elasticity of the PCC material. Each of these pavement types distributes load over the subgrade in a different fashion.

Rigid pavements, because of PCC's high elastic modulus (stiffness), are prone to distribute the load over a relatively wide area of subgrade. The concrete slab itself supplies most of a rigid pavement's structural capacity.

Flexible pavement uses less rigid surface course and distributes loads over a smaller area. It relies on a combination of layers for transmitting load to the subgrade. Rigid pavement structure distributes loads over a wide area with only one, or at most two, structural layers.

The fundamental differences between a flexible and rigid pavement are the load distributions over the sub-grade.

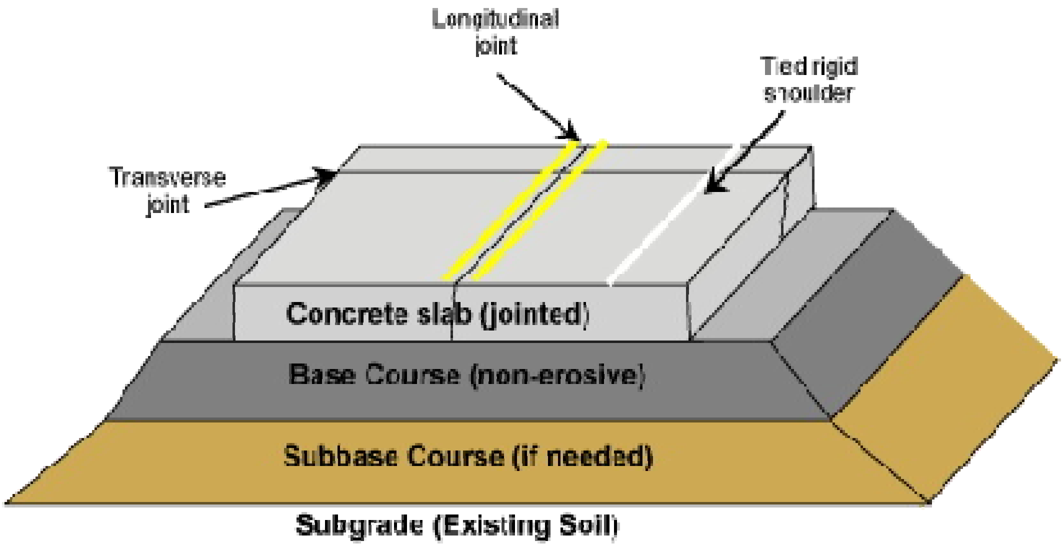

Figure 2.3. Typical section for a rigid pavement.

There are two types of concrete pavements that are commonly used: Continuously Reinforced Concrete Pavement (CRCP) and Jointed Plain (Non-Reinforced) Concrete Pavement (JPCP).

CRCP includes both longitudinal and transverse steels. CRCP does not contain transverse joints except at construction joints. The function of the longitudinal steel is not to strengthen the pavement, but to force the pavement to crack within certain desirable crack spacing and to keep those cracks tightly closed. The function of the transverse steel is to keep longitudinal joints and cracks closed. If the steel serves its proper function and keeps cracks from widening, aggregate interlock is preserved and concrete stresses at the cracks due to traffic loading are reduced.

JPC Pavements do not have reinforcing steel and has transverse joints spaced at regular intervals. The transverse joints are used to control temperature induced stresses in the concrete. Longitudinal joints are used to enable construction and control cracking. Pavements of this type will have smooth dowels at the joints for load transfer. The joint spacing is kept at a constant 15 feet (~ 4.57 m) for CPCD standards

Flexible Pavement Section Rigid Pavement Section

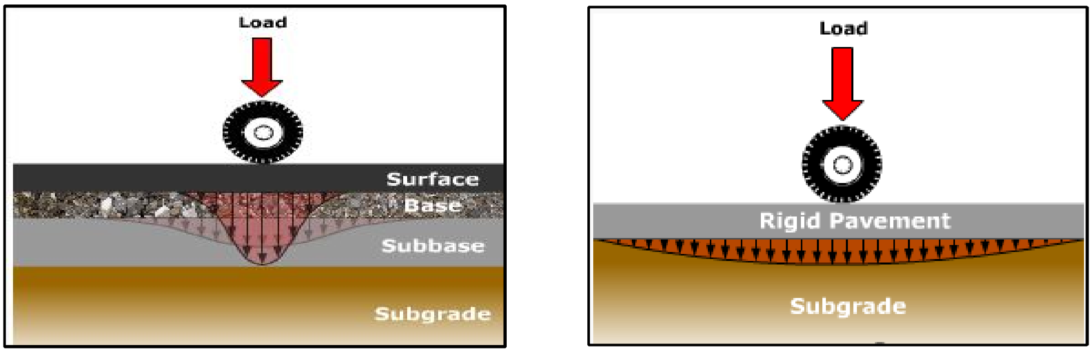

Figure 2.4. Typical stress distribution under a rigid and a flexible pavement.

3.3.3. Composite pavements

Composite pavements are the combination of HMA and Portland cement concrete (PCC) pavements. Occasionally, they are initially constructed as composite pavements. More frequently, they are the result of pavement rehabilitation e.g., HMA overlay of PCC pavement). Composite pavement behavior under traffic loading is essentially the same as that of a rigid pavement.

Finally, it may be rather confusing as to why one pavement is used versus another. Basically, highway agencies generally select pavement type either by policy, economics or both. Flexible pavements generally need some type of maintenance or rehabilitation every 10 to 15 years. Rigid pavements, on the other hand, can often provide sufficient service 20 to 40 years with little or no maintenance or rehabilitation. Thus, rigid pavements are often used in urban, high traffic areas. But, naturally, there are trade-offs. For example, when a flexible pavement requires major rehabilitation, the options are generally less expensive and quicker to perform than for rigid pavements.

In Turkey, almost entire employment and structural investments related pavement construction have condensed in flexible pavement area. Due to the fact of insufficient and incomplete infrastructure conditions, pavements always require maintenance and reconstruction. Considering easy maintenance advantage of flexible pavements, highway agencies in Turkey rightfully ought to be chose flexible pavements.

2.4. Flexible Pavement Materials

2.4.1. Asphalt Concrete

One of the well known type of flexible pavement surfacing in the world is hot mix asphalt (HMA). Hot mix asphalt is labeled by many different names such as hot mix, asphalt concrete (AC or ACP), asphalt, or bitumen. Hot mix asphalt (HMA) consists of a combination of uniformly graded aggregate and coated with appropriate asphalt (bitumen) cement.

Asphalt is one of the two principal input materials of Hot Mix Asphalt (HMA). Asphalt functions are generally known as waterproof, thermoplastic viscoelastic, adhesive. A dark brown to black cementitious material in which the predominating constituents are bitumen, which occur in nature or are obtained in petroleum processing. Bitumen is a class of black or dark-colored (solid, semi-solid or viscous) cementitious substances, natural or manufactured, composed principally of high molecular weight hydrocarbons, of which asphalts, tars, pitches, and asphaltenes are typical.

Aggregate is primarily responsible for the providing of load supporting capacity of pavements; accordingly, performance of a HMA mixture is heavily influenced by the aggregate. The amount of aggregate in asphalt mix is generally 90-95 percent by weight or 75-80 percent by volume.

The general terms “aggregate” refers to any hard, inert mineral material used for mixing in graduated particles or fragments.

Aggregates can either be natural or manufactured. Natural aggregates are generally extracted from larger rock formations through an open excavation (quarry). Extracted rock is typically reduced to usable sizes by mechanical crushing. Manufactured aggregate is often the byproduct of other manufacturing industries.

HMA is distinguished by its design and production methods and includes traditional dense-graded mixes as well as stone mastic asphalt (SMA) and various open-graded HMAs. Typically concerned agencies in U.S. even in Turkey, consider other types of asphalt-based pavement surfaces such as fog seals, slurry seals and Bituminous Surface Treatments (BST) to be maintenance treatments Reclaimed asphalt pavement (RAP) is generally considered a material within HMA, while forms of in-place recycling are considered separately.

Both the aggregate and the asphalt must be heated before mixing-hence the term hot mix. Regardless of the design procedure used, the design of an HMA mix consists of the following three steps:

i). Selection of the type an gradation of the mineral aggregate
ii). Selection of the type and grade (penetration based or performance based) of the asphalt binder
iii). Selection of the amount of asphalt binder to satisfy the project- specific requirements.

2.5. Classification of Hot Mixes

HMA paving mixtures may be produced from a wide range of aggregate combinations, each having its own particular characteristics suited to specific design and construction uses.

2.5.1. Dense- Graded HMA

Dense-graded mixes are generally referred to by their nominal maximum aggregate size. They can additionally be classified as either fine-graded or coarse-graded. Fine-graded mixes have more fine and sand sized particles than coarse-graded mixes.

Dense-graded mixes are appropriate for all pavement layers and also for all traffic conditions. Dense-grade mixes work well for structural, friction, leveling and patching needs.

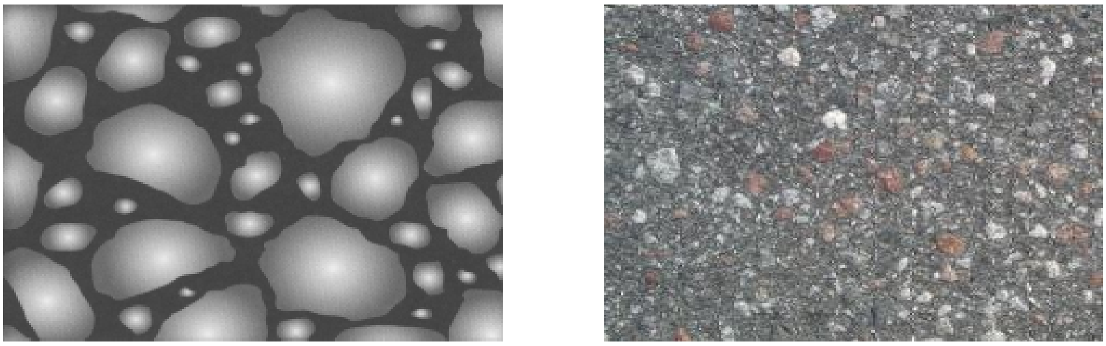

Figure 2.5. DGAC horizontal cross-section and DGAC surface texture

2.5.2. Stone Matrix Asphalt (SMA)

SMA is an HMA which have high coarse aggregate content (typically 70-80 percent), high asphalt content (typically over 6 percent) and a high filler content (approximately 10 percent by weight). The concept for SMA is to design a mixture that has excellent stone to stone contact and has better resisting ability to rutting problems.

Stone matrix asphalt (SMA) is a gap-graded HMA that is designed to provide maximum deformation, especially rutting, resistance and durability by using a structural basis of stone to stone contact. Because the aggregates are all in contact, rut resistance relies on aggregate properties rather than asphalt binder properties. Since aggregates do not deform as much as asphalt binder under load, this stone to stone contact greatly reduces rutting.

SMA is generally more expensive than a typical dense-graded HMA. Because it requires more durable and high quality aggregates, higher asphalt content and, typically, a modified asphalt binder and cellulose fibers. In the right situations it should be cost-effective because of its increased rut resistance and improved durability. SMA originally developed in Europe to resist rutting and studded tire wear, has been used in the Turkey since 2005.

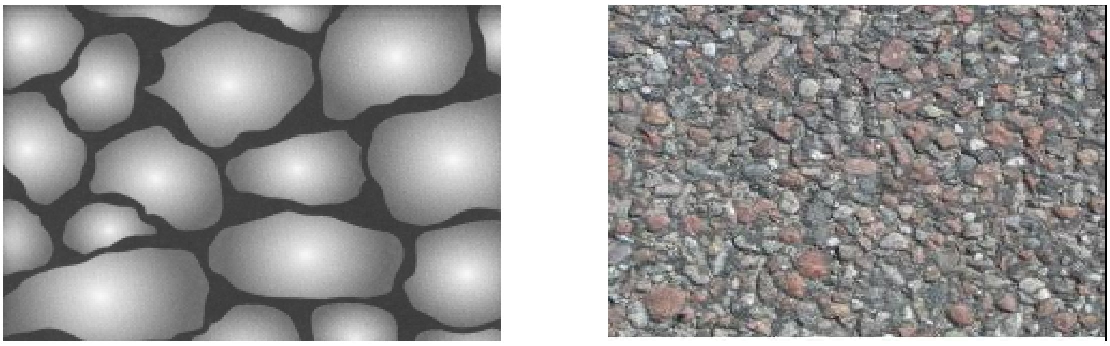

Figure 2.6. GGAC horizontal cross-section and GGAC surface texture

2.5.3. Open –Graded HMA

An open-grade layer is an HMA mixture designed to have a large volume of air-voids so that water will drain thorough the pavement layer. It is also called as Open-Graded Friction Course (OGFC) to provide a skid resistant pavement surface and as a porous base layer to provide for positive drainage under either an HMA or Portland cement concrete pavement surface.

OGFC is used as a surface course only. It reduces tire splash and spray in wet weather and typically results in smoother surfaces than dense-graded HMA. The high air voids trap road noise and thus reduce tire-road noise by up to 50-percent (10 dBA)

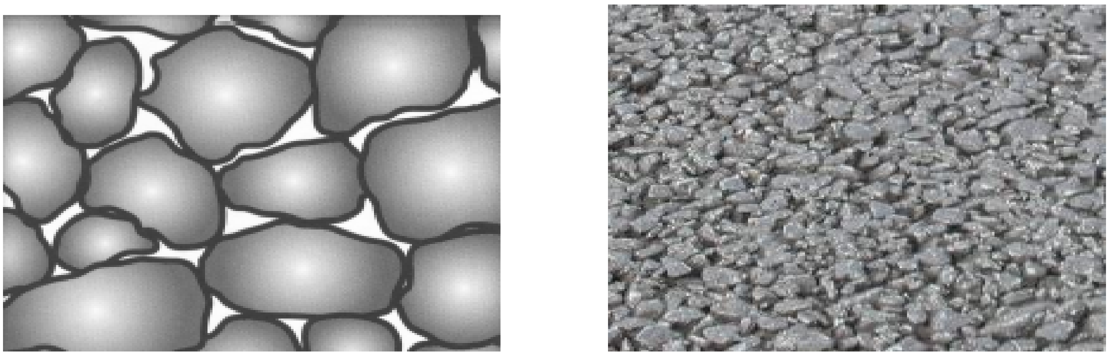

Figure 2.7. OGAC horizontal cross-section and OGAC surface texture

2.6. Design of Asphalt Pavements

The aim of mix design is to identify the suitable, optimum mixture of component materials for a pavement. This includes detailed evaluations of aggregate, asphalt and Portland cement as well as a determination of their optimum mixing ratios.

HMA is composed of two basic ingredients: aggregate and asphalt binder. HMA mix design is the process of determining what aggregate to use, what asphalt binder to use and what the optimum combination of these two ingredients ought to be. There are several different methods used to go about this process, of which the Hveem, Marshall and Superpave methods are the most common.

2.6.1. Desired Properties Considered for Mix Design

Hot mix asphalt pavements perform well if they are designed, produced, and placed to provide certain desirable properties. These include:

- Resistance to permanent deformation
- Fatigue resistance
- Low temperature cracking
- Moisture resistance
- Durability
- Skid resistance
- Workability

The final goal of mix design is to select a unique asphalt content that will achieve a balance among all the desired properties. No single asphalt content will maximize all these properties. Instead, asphalt content is selected on the basis of optimizing the properties necessary for specific conditions.

2.6.2. Evolution of HMA Design

Hot mix asphalt pavements have been built in the United States since the 1860s. The first mix design procedure was developed by Clifford Richardson. He recognized the importance of material selection, especially the characteristics of the fine aggregate, and the importance of air-voids and voids in mineral aggregate. He published his procedures in The Modern Asphalt Pavement I named book. His procedure used the Pat Test. This test procedure consisted of compacting samples of sheet asphalt against a brown manila paper and visually assessing the residual stains. A heavy stain indicated too much binder; a light stain indicated too little binder; and a medium stain indicated optimum binder content. Since Richardson's initial work, researches continued and additional more sophisticated procedures for developing a mix design have developed.

The most prominent method until World War II was the Hubbard-Field Method, which was developed in the mid-1920s. This procedure used a stability test that consisted of determining the maximum load developed on a specimen 2 inches (50 millimeters) in diameter by 1 inch (25 millimeters) in height being forced through an orifice 1.75 inches (45 millimeters) in diameter.

2.6.3. History of the Marshall Mix Design

Before the introduction of Superpave mix design procedures in the early 1990s, the majority of HMA pavements constructed in the United States were designed with earlier the Marshall or the Hveem mix design procedures. The Marshall procedure was developed by Bruce Marshall of the Mississippi Highway Department in 1939. The U.S. Army Corps of Engineers adapted the procedure during the World War II for the design of military airfields. The goal was to have a procedure that used simple potable laboratory equipment that could be used to design pavements that would support the increase in aircraft wheel loads and tire pressure continued the increase.

The Marshall Mix design procedure has been continually improved to appoint where it is now used for wheel loads of up to 60,000 pounds and tire pressure of 350 pounds per square inch. The federal Aviation Administration and the Department of Defense continue to use the Marshall procedure for airfield design.

Also, in the early 1950s state highway departments were expanding the construction of high-volume roadways. They modified the procedure developed by the Corps of Engineers to design HMA mixes for highway pavements. Prior to the introduction of the Superpave mix design procedure in the early 1990s, approximately 75 percent of state highway agencies were using the Marshall Mix Design procedure.

2.6.4. History of the Hveem Mix Design

In the 1930s, Francis Hveem, then the materials and research engineer for the California Division of Highways, developed a mix design procedure that was used expensively in the western United States. Hveem introduced the concept of the kneading compactor so that the laboratory-compacted mix would be more representative of the field-produced mixes that are being compacted by steel and pneumatic-tired rollers. In addition to developing the kneading compactor, he recognized the need to have a mechanical test that would evaluate the performance of the mix. This need led to the development of the Hveem Stabilometer, which is used to evaluate the ability of an HMA mixture to resist the shear force applied by traffic. Prior to the introduction of the Superpave mix design procedure in the early 1990s the Hveem procedure was used extensively in the western United States.

2.6.5. Adoption of Superpave Mix Design System

From 1988 to 1993, the Federal Highway Administration sponsored and the Transportation Research Board administrated the $150 million Strategic Highway Research Program (SHRP). The purpose of SHRP was to develop technology that would result in significant improvements in the way highways were designed and built to address the effect of increasing traffic on the nation's highway infrastructure. Approximately $50 million of the SHRP funding was used to support an asphalt research program. The result o this research was a new mix design system called Superpave (SUperior PERforming Asphalt PAVEments). Superpave gave the highway industry new tools for designing and constructing HMA pavements. The first projects designed with Superpave technology were built in the early 1990s and the procedure quickly became the standard for design of HMA pavement mixtures in the United States and Canada.

The airfield community has not fully implemented Superpave, but research is being conducted to adapt Superpave to the different demands of airfields.

The Marshall method is very popular because of its relatively simple, economical equipment and proven record. In Turkey, Marshall Method is widely used for mix design by governmental and municipal pavement agencies. The superpave mix design concept has not been implemented by these authorities due to the fact that of laboratory equipments is being very expensive and less know-how about SuperPave.

Typically, the Marshall Mix design method consists of three basic steps:

i). Aggregate selection; different agencies/owners specify different methods of aggregate acceptance. Private labs may or may not run periodic aggregate physical tests on a particular aggregate source. For each mix design, gradation and size requirements are checked. Often, aggregate from more than one source is required to meet gradation requirements.

ii). Asphalt binder selection

iii). Optimum asphalt binder content determination. In the Marshall method, this step can be broken up into five substeps:

Preparing of a series of initial samples, each at different asphalt binder content; for instance, two to three samples each might be made at 4.5, 5.0, 5.5, 6.0 and 6.5 percent asphalt by dry weight for a total of 10 to 15 samples. There should be at least two samples above and two below the estimated optimum asphalt content.

Compacting of these trial mixes using the Marshall drop hammer; this hammer is specific to the Marshall Mix design method.

Testing of the samples in the Marshall testing machine for stability and flow; this testing machine is specific to the Marshall Mix design method. Passing values of stability and flow depend upon the mix class being evaluated.

Determining of density and other volumetric properties of the samples and selecting of optimum asphalt binder content; the asphalt binder content corresponding to 4 percent air voids is selected as long as this binder content passes stability and flow requirements.

2.7. Pavement Management and Evaluation

Transportation authorities have made significant investment in their pavement their pavement assets when both the cost of original construction and ongoing maintenance are considered. These assets are a vital part of the nation's economy, providing important links to markets, schools, medical facilitates, and place of employment. Demand on this road network continues to grow with time, placing more importance on maintaining suitable pavement conditions at a time when increasing loads are causing pavement to deteriorate more rapidly.

Faced with this type of challenges, managers of transportation agencies are making use of tools that provide the type of information needed to support their decision-making process. Pavement Management Systems (PMS) is an example of this type of tool. The American Association of State Highway and Transportation Officials (AASHTO) defines PMS as: "A pavement management system is a set of tools or methods that assist decision makers in finding optimum strategies for providing, evaluating, and maintaining pavements in serviceable condition over a period of time."

A PMS uses reliable pavement inventory and condition information to help to identify and prioritize pavement maintenance and rehabilitation needs within budget or other constraints that may exist. Consequently, agencies can use the information from a PMS to make more cost-effective maintenance and rehabilitation decisions, which result in more efficient use of agency resources.

2.6.6. Life-Cycle Cost Analysis

Determining the most cost-effective maintenance or rehabilitation strategy for an existing pavement is very important. However, a life cycle cost analysis (LCCA) provides a means for comparing treatments strategies over an analysis period of 20 to 40 years.

In an LCCA, all cost experienced to be incurred over the life of pavement are identified and converted to a single point in time using economic equations that represent the time value or money. Because all costs are converted to a single point in time, different treatment strategies with different performance lives can easily be compared.

LCCA allows an agency to consider both agency costs and user costs in the same analysis. Agency costs are represented by direct cost to the agency for construction, maintenance, and rehabilitation. User costs, which are not always considered in an LCCA, represent the cost born by the users under each scenario.

A study conducted by the Utah Department of Transportation (UDOT) in 1997 demonstrated the concept that it costs less to maintain roads in good condition that it does roads in bad condition. Since that time, other agencies have used the information such as that present in Figure 2.8 to illustrate the difference in the relative magnitude of cost associated with maintaining roads in good condition versus those associated with allowing pavements to deteriorate until more substantial repairs are required.

A cost-effective pavement management strategy optimizes the use of available maintenance and rehabilitation funds, budgeting some funds to address roads requiring major rehabilitation or reconstruction while also budgeting funds roar roads in good and fair condition to slow their rate of deterioration. By slowing the rate of pavement deterioration on these roads, an agency can maintain its road network at a higher overall condition level and defer the need for more costly rehabilitation treatments.

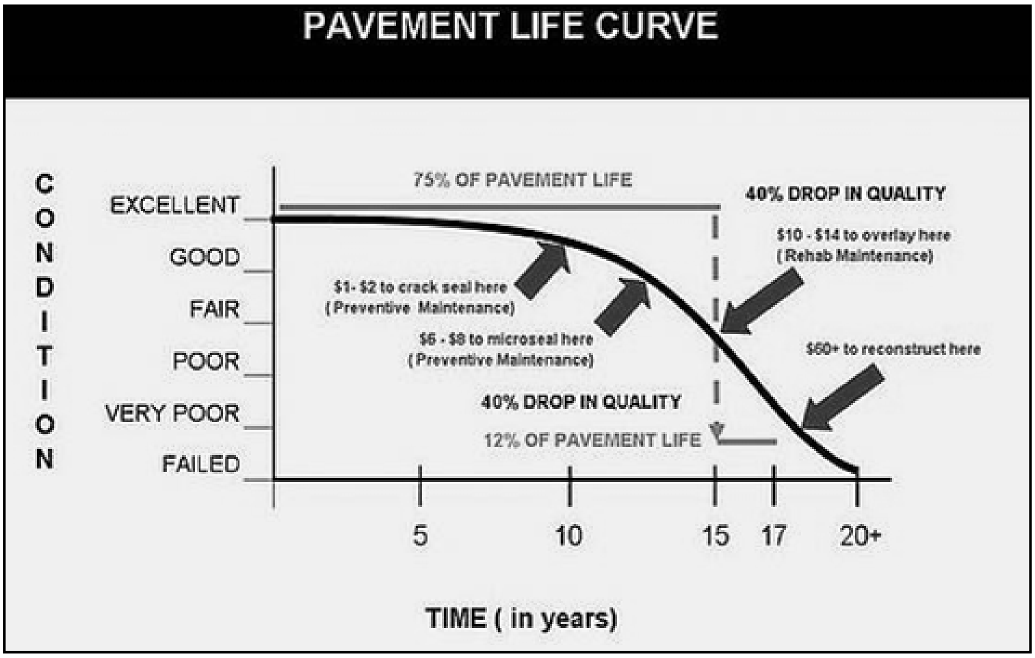

Figure 2.8. Typical Pavement Condition as a Function of Time

2.6.7. Preventive Maintenance

A pavement management system can also be used to help identify good candidates for preventive maintenance. The result of pavement condition surveys can be used to identify roads that are not severely deteriorated or where structural deterioration has not taken place. Once a candidate for preventive maintenance has been identified, the section should be investigated in more detail to determine the extent of deterioration present and to verify that a preventive maintenance treatment is appropriate to correct the distress present or to prevent the further deterioration of the pavement surface.

2.6.8. Long-Term Benefits to Pavement Preservation

Agencies that have strong pavement preservation programs in place have recognized a number of benefits, including those listed below:

- Higher consumer satisfaction
- Better informed decisions
- Improvement strategies and techniques
- Improved pavement condition

- Cost saving
- Increased safety

2.8. Pavement Evaluation

A pavement management system relies on objective, repeatable pavement condition information to determine current and future maintenance and rehabilitation needs. Therefore, conducting a meaningful pavement evaluation is one of first step in the identification of feasible maintenance and rehabilitation strategies. Two types of pavement evaluations are normally conducted as a part of an agency's pavement management practices: a functional evaluation and a structural evaluation.

A functional evaluation considers the surface characteristics of a road, including certain types of cracking, surface smoothness, noise, and surface friction characteristics.

A structural evaluation is used to determine the ability of pavement structure to carry traffic loadings. A structural evaluation typically requires detailed information about pavement layer thickness, paving layer material properties, subgrade support conditions, traffic, and the response of the pavement to loading.

2.6.9. Use of Pavement Condition Information

The result of functional pavement evaluation provides the information needed for the pavement management system to identify and prioritize maintenance and rehabilitation needs. Typically, pavement distress and roughness information are used to calculate pavement condition indexes that can be used to compare the condition of one pavement section with another and to identify the level of repair that might be needed to address the deficiencies identified. Over time, pavement condition information help establish rates of pavement deterioration that can be used to forecast future pavement conditions under different funding scenarios.

2.6.10. Pavement Condition Indexes

To facilitate the use of pavement distress information in a pavement management system, pavement condition indexes are calculated based on the type, severity, and quantity of distress present.

Pavement condition indexes typically range in value from 0 to 10 or 0 to 100, with the highest values corresponding to pavement excellent condition. Points are subtracted from a perfect score based on the distresses observed in the field to calculate the index. The complexity of pavement condition index calculation varies depending on the type of survey being conducted.

Pavement condition indexes can be calculated for each type of distress present or composite index can be used that aggregates all of the distress into one index representing the overall condition of pavement section. Some agencies use a combination these two types of indexes; the individual indexes are used to identify appropriate maintenance and rehabilitation treatments and the composite index is used to report the overall network conditions.

3. FLEXIBLE PAVEMENT DISTRESSES

3.1. Introduction

Road construction is relatively expensive investment according to other infrastructural investments. Governments are naturally tried to minimize their expenses for investments which have consistent rehabilitation and maintenance charges. Road construction investment has same characteristic features on definition above. Pavement is the most expensive part of road construction so that evaluation of pavements then rehabilitation and maintenance of them are significantly effects its management costs. Considering importance of management process, defining and explicating of pavement distresses get become important to choose right preservation application. Herein, typical flexible pavement distresses are going to be defined and explained with pictures. One of the important permanent deformations, rutting, are going to be particularly explained in subtitles. Common types of distress can be classified into the five general categories listed here:

- Cracking
- Distortion
- Disintegration
- Skid hazards
- Surface treatment distresses

3.2. Typical Causes of Deterioration

Some of the primary causes of HMA pavement deterioration are listed here:

- Traffic loading
- Environment or climate
- Drainage deficiencies
- Material Problems
- Construction deficiencies and External causes (such as utility cuts)

As pavement age, one or more of these distress mechanisms begins to take its toll. As cracking and other forms of disintegration begin to appear as the primary cause of deterioration, secondary factors often contribute additional amounts of deterioration. For instance, once cracking begins to appear in the pavement surface, moisture can intrude in the pavement structure and accelerate the deterioration caused by initial distress mechanism. The timely application of pavement maintenance techniques serves to help prevent or slow down the effects of both primary and secondary distress mechanism.

3.3. Cracking

A number of different types of cracking can occur in an HMA pavement and appropriate type of repair depends on the type of cracking present. Some cracking, such as alligator cracking, indicates load-related deterioration that requires a different maintenance strategy than block cracking, which is typically caused by climatic forces. This section provides information on some of the most common types of cracking included in a pavement evaluation.

- Alligator (Fatigue) Cracking
- Block Cracking
- Edge Cracking
- Longitudinal (Linear) and Transverse Cracking
- Reflection Cracking (PCC Joint Reflection Crack)
- Slippage Cracking

3.4. Distortion

Distortions in the pavement layer are a result of instability in the HMA layer or weaknesses in the base or subgrade layers. The distortion may appear in a number of different forms, including rutting, corrugations, depressions, or upheavals. Cracking may or may not accompany the distortion. Each of the typical types an causes of distortions are discussed further in this section.

Figure 3.1. High severity longitudinal and transverse cracks.

3.4.1. Corrugations and shoving

Corrugations and shoving, also known as washboarding, result in ripples across the asphalt pavement surface at fairly regular intervals of less than 3 meter and perpendicular to the traffic direction. Corrugation and shoving typically occur at point of severe horizontal stress in HMA layers that lack stability.

3.4.2. Rutting

A rut is a surface depression in the wheel paths that may also have transverse displacement along the side of the rut. Rutting is caused by consolidation or lateral movement of any of the pavement layers or subgrade under traffic. It may be caused by insignificant design thickness, lack of compaction, weakness in the pavement layers due to moisture infiltration, weak asphalt mixtures, or load induced stresses.

Figure.3.2 Shoving and corrugation on pavement surface

Figure 3.3 High Severity asphalt rutting on highway

3.4.3. Settlement or Grade Depressions

Depression and settlements are located pavement surface areas with elevations lower than the surrounding pavement areas. Minor depression are often not noticed until after a rain when water ponds in the depression, causing a "birdbath" that can be hazardous to motorist. In dry weather, depressions can be observed where staining is present. Depression may be caused by settlement or failure in the lower pavement layers or by improper construction techniques.

3.4.4. Upheaval or Swell

An upheaval or swell is a localized upward displacement in a pavement due to swelling of the subgrade or some portion of the pavement structure. A frost heave is an example of this type of distress.
Upheavals, or swells, are often the result of expansive soils or frost heave (in which ice lenses grow beneath the pavement, causing the pavement to crack).

3.4.5. Utility cuts and/or Patch Failure

A patch is apportioned of a pavement that has been removed and replaced or where additional material has been added. Patches are a form of pavement distress regardless of how well they are performing, but the severity of the problem increases when the patch the patch has deteriorated. The rate at which a patch deteriorates may be influenced by poor installation techniques, such as inadequate compaction, inferior materials, or failure of the surrounding or underlying pavement.

3.5. Disintegration

Disintegration includes the breakup of the pavement into smaller pieces that may become dislodged over time. It is important to address pavement disintegration early, before too much of the pavement surface has been lost. Weathering, raveling and potholes are examples of pavement disintegration.

3.6. Skid Hazards

An important component of providing a safe pavement surface is keeping the surface free of properties that might increase the likelihood of vehicles skidding on the surface, such as surface water, polished aggregates, or excess oil on the pavement surface in a way that allow water to flow off the prevent surface and maintains sufficient contact between the vehicle tires and aggregate. Bleeding, flushing and polished aggregate are the example of skid hazards

3.7. Surface Treatment Distress

Because of their unique properties, some types of distress may occur only in an asphalt surface treatment and not in other types of asphalt pavements, such as loss of cover aggregate and streaking. Other distress discussed earlier in this chapter, such as depressions, raveling, corrugations, and potholes occur in both hot-mix asphalt and bituminous surface treatments. Losses of cover aggregate and longitudinal/transverse streaking are the example of surface treatment distress

3.8. Rutting Phenomena

A major distress type in flexible (asphalt concrete) pavements is rutting. Rutting is the mechanism that produces depressions in the wheel-paths of asphalt concrete pavements. Rutting is the result of volumetric compression and/or shear deformation of one or more layers of the pavement system (asphalt concrete, base, and/or subgrade) under repeated traffic loadings. Rutting reduces serviceability and creates the potential for hydroplaning due to the accumulation of water in the wheel-path ruts (Novak, 2007).

Rutting is the formation of twin longitudinal depressions in the wheel paths due to a progressive accumulation of permanent deformation in one or more of the pavement layers (Anani, 1990). The rate and magnitude of rutting depend on external and internal factors. External factors include load and volume of traffic, tire pressure, temperature and construction practices. Internal factors include properties of the binder, the aggregate and mix properties, and the thickness of the pavement layers.

Significant rutting normally only occurs during hot weather, when the surface of flexible pavements can reach a temperature of 60°C or higher. Furthermore, this mode of distress is also associated with relatively high traffic levels; the greater the number of vehicles and greater the proportion of heavy trucks, the greater the potential for permanent deformation. Rutting is a serious problem for a number of reasons; for example rain or melted snow and ice can pond in the ruts, increasing the chance for vehicle hydroplaning and subsequent accidents. Excessive ruts can also reduce the effective thickness of a pavement, reducing the structural capacity of the pavement and increasing the likelihood of premature failure through fatigue cracking (Christensen, 2000).

One form of rutting is known as “instability rutting.” Instability rutting is rutting which is confined only to the asphalt concrete layer. Instability rutting in asphalt pavements is primarily due to the lateral displacement of material within the asphalt concrete layer. Instability rutting is generally seen in pavements with a thick asphalt concrete layer (high trafficked roadways) and is the predominant mode of premature failure is modern asphalt pavements.

There are two types of rutting: Unbound Layers Rutting, and Mix Rutting (Dawley et al. 1990). The first is Unbound Layers Rutting or consolidation rutting. This is the traditional term used when discussing rutting. It refers to volumetric compression and/or shear deformation of the base or subgrade with an assumption that the asphalt concrete layer contributes very little to the overall rutting of the pavement system (Huang, 1993). This mode of rutting may result from possible insufficient compaction of base and subgrade layers, which undergo air void reduction and shear deformation under repeated traffic loadings. It can also be due to the consolidation phenomenon in clayey bases and subgrade Unbound Layers Rutting or consolidation rutting will occur over the design lifetime of the pavement system and is not typically premature failure mode - unless the base and subgrade are poorly compacted. Rutted roads due to this mechanism (Figure 3.4) are marked by shallow sloping ruts that are fairly wide (30- 40 inches) (Huber, 1999).

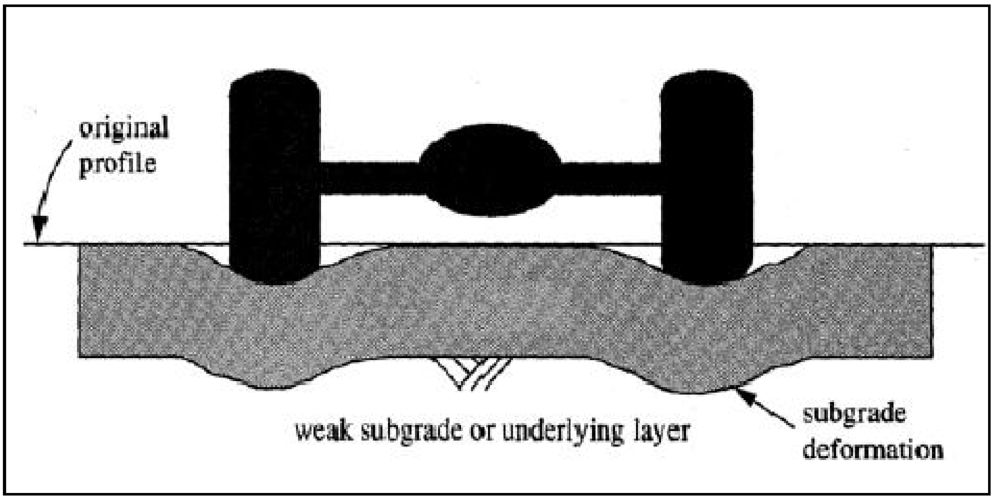

Figure 3.4. Unbound Layers Rutting (Huber 1999)

The second type of rutting is mix rutting. Mix rutting is due to lateral displacement of material within the asphalt concrete layer only. Mix rutting is a near surface phenomenon occurring in the top 2 inches of the asphalt layer (Dawley *et al.*, 1990). Mix rutting occurs when the structural properties of the compacted pavement are inadequate to resist the stresses from frequent repetitions of high axle loadings. The aggregates rigidly translate and rotate within the asphalt binder (Wang *et al.*, 1999). Mix rutting (Figure 3.5) is characterized by steep longitudinal ruts in the pavement with humps of material on either side of the rut (Huber, 1999).

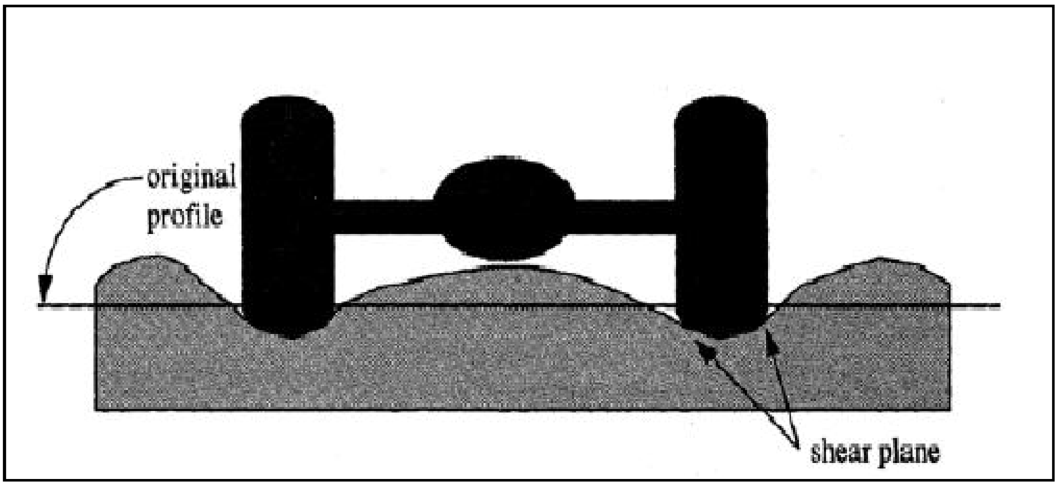

Figure 3.5. Mix rutting. (Huber 1999)

3.8.1. Factors That Affect Rutting

Rutting is strongly influenced by traffic loading, but climate can also have a large influence especially when the pavement subgrade undergoes seasonal variations in bearing capacity, or when bituminous courses are subjected to high temperatures. Ruts develop within pavement layers when traffic loading causes layer densification and/or when stresses induced in the pavement materials are sufficient to cause shear displacements within the materials (Figure 3.6).

Research performed over several decades has shown that the susceptibility to rutting can be linked to the following material attributes: excessive asphalt content, excessive fine grained aggregate, high percentages of natural sand, rounded aggregate particles, excessive permissible moisture in the mix or in granular materials and soils, temperature susceptible asphalt cement, and cold weather paving leading to low density.

Figure 3.7. Surface depression in the wheel paths

Other factors affecting rutting are temperature; precipitation; and the time, type, and extent of loading. The above factors when combined also determine measures such as Hveem and Marshal Stability, complex modulus, resilient modulus, and deflection that are normally used for pavement distress modeling. Generally, only a few of these factors are measured in experimental data sets and thus can be used in an empirical model such as the one developed herein (Archilla and Madanat, 2000).

Herein, the factor affecting the rutting performance of asphalt concrete pavements is surface stresses caused by axial repeated loads. The theoretical problem under consideration is how to minimize the rut effect. Several models have been used to relate plastic strain accumulation to the number of load or stress repetitions.

Mechanistic-empirical modeling for flexible pavements bases on the use of a mathematical model to describe the response of the pavement system to an exteriorly applied load representative of the traffic to which the roadway will be subjected (Perkins, 2001).

The response taken from the model is mostly a measure of stress, strain or deflection for one or several points within pavement system. Several types of mathematical or response models are available pavement analysis and design. Multi-layered elastic (MLE) programs, Asphalt Institute's in the same direction module, DAMA are the mathematical models for pavement analysis and design (Perkins, 2001).

4. GEOSYNTHETICS IN PAVEMENTS

4.1. Geosynthetic types and functions

The meaning of the word geosynthetic can be easily understood when broken down into two parts; “geo” and “synthetic”. “Geo” simply refers to earth, while “synthetic” describes a man made substance. Geosynthetic materials are typically made from polymers (hydrocarbons), which are derived from petroleum. As a result, biodegradation is not a problem. In addition to polymers, rubbers, fibers, glass, or other materials can be incorporated into product (Koerner, 1998). The primary polymers used in the manufacturing of geosynthetic materials are (Koerner, 1998), (Rolling and Rollings, 1996), and (Van Santvoor, 1995);

- Polyester (PE)
- Polypropylene (PP)
- Polyethylene (PE)
- Polyamide (PA)
- Polyvinyl Chloride (PVC)

Over the past few years, geosynthetic usage has increased tremendously. Reasons for this include the need for such a material, ease of installation, quality control in manufacturing, cost competiveness, and their ability to replace raw materials in designs (Koerner, 1998).

The term geosynthetic is a broad term used to encompass several different classifications of materials. The four most common classifications are geotextiles, geogrids, geocomposites, and geomembranes. Of these, only geotextiles and geogrids are of interest herein. They are the only geosynthetics used in this study, and are described in more detail in the following sections.

4.1.1. Geotextiles

The types of geotextiles currently in use were initially intended as an alternative for granular soil filters, a use which explains their alternative names “filter fabrics” (Koerner, 1986). Geotextiles are created by taking individual fibers and transforming them into a porous and flexible material. This is accomplished by either standard weaving machinery, random matting, or knitting. These processes result in woven, non-woven, and knitted textiles, respectively. Of these woven and non-woven geotextiles are the most common (Koerner, 1998). While woven textiles are formed by standard weaving methods, non-woven textiles are created in series of four steps. They are: fiber preparation, web formation, web bonding, and post-treatment.

Geotextiles are typically used to perform one or more o the following tasks: separation, reinforcement, filtration, and/or drainage. While there are other uses for geotextiles, these are the primary functions of interest herein. In order to adequately perform these functions, certain properties are required. The following list gives ranges of available geotextile properties (Koerner, 1986).

- Mass: 0.1-1 kg/m2 (3-30 oz/yd2)
- Thickness: 0.25- 7.5 mm (10-30 mil) Note : 1 mil = 0.001 in
- Specific Gravity: 0.9 – 1.4
- Percent Open Area: Up to 36%
- Equivalent Opening Size: 30-300 US Sieve No.
- Grab Strength: 8.75-875 N/mm (50-5,000 lb/in)
- Grab Elongation: 20 – 200%
- 10% Secant Modulus: 17.5-1,750 N/mm (100 – 10,000 lb/in)
- Cross Plane Permeability: 0.01-5.0 cm/s (0.004-1.97 in/s)
- In-Plane Permeability: Up to 2 cm/s (0.787 in/s)

When used for separation, geotextiles are placed between two materials that are otherwise prone to intermixing. A classic example is the subgrade and base of roadway. The intrusion of either of the two materials into the other weakness the roadway, eventually compromising its integrity. Soil and/or unbound aggregate bases possess little to no tensile strength, but do possess sustainable compressive strength. The opposite is true for a geotextile. As a result, the addition of the geotextile can provide tensile reinforcement to the system (Rollings and Rollings, 1996).

Filtration is the action of preventing specific matter from flowing across a given plane. When geotextiles are used for filtration, the flow is filtered perpendicular to its plane (Rollins and Rollings, 1996). Drainage is achieved by flow within the geotextile. Bulky flet like fabrics have been shown to be effective in drainage, thereby dissipating pore water pressure (Koerner, 1986).

4.1.2. Geogrids

Geogrids are typically made from polypropylene, polyster or high-density polyethylene. They represent a small, but growing, portion of the geosynthetic market share (Koerner, 1986). Geogrids are generally manufactured in two ways. One technique is by taking a piece of heavy gage (4-6 mm [0.16-0.24 in] thick) material and punching apertures (holes), in a regular pattern, into it. The sheet is then drawn (stretched) either uniaxially or biaxially to improve its physical properties and obtain the desired thickness and opening sizes. This process is similar to "cold working" steel and the opening sizes are typically from 10-100 mm (0.4-4 in) (Koerner, 1998). A second technique that was developed later is to weave bundles of fibre with openings so to create a mesh.

Geogrids are by and large used for reinforcement and confinement, but occasionally they are used for separation of large sized particles (Koerner, 1998). Reinforcement is achieved since unbound aggregate and/or soils typically cannot carry large (if any) tensile forces. The additional of the geogrid allows this to occur.

The geogrid "locks" the aggregate in place; this is the purpose of the apertures. A portion of the bottom layer of aggregate settles into these apertures, which makes proper matching of aggregate sizes with the geogrid holes essential.

Confinement of the unbound material is also an important function of the geogrid. Loads applied to a pavement structure create lateral forces that tent to spread the unbound base. As a result, tensile strain is created as the material moves both down and out away from the loading. The inclusion of a geogrid, which is much stiffer with respect to tension, helps to prevent this phenomenon (Perkins, 1999).

In order to adequately perform, some research findings indicate an example of this can be seen in a design chart given by Koerner. This design chart relates equivalent thickness values of reinforce and non-reinforce base courses. According to the chart, the geogrid under investigation is to be placed at the bottom of thinner base (<250 mm [10 in]). Various placement schemes in base courses are discussed in the case studies that follow

.

4.1.3. Geocomposites

A geocomposite consists of a combination of geotextile and geogrid, or geogrid and geomembrane, or geotextile, geogrid, and geomembrane, or any one of these three materials with another material (e.g., with soil, Styrofoam, deformed plastic sheets, steel cables, steel anchors, etc.). This exciting area brings out the best creative efforts of the engineer, manufacturer, and/or contractor. The application areas are numerous and growing steadily. The major functions encompass the entire gambit of functions listed for the geosynthetics discussed previously (Koerner, 1986).

4.2. Geosynthetic Materials used in pavements

Geosynthetic products, explained herein as textiles, grids, composites, or membranes, have been used for asphalt reinforcement since 1980 in U.S. The primary purpose of incorporating the use of geosynthetics in the pavement design process is to reduce reflective cracking in HMA overlays and to resist moisture intrusion into the underlying pavement structure. Geosynthetics can be part of an overall rehabilitation strategy that will, as a minimum, include the placement of a new wearing/surface course of hot mix asphaltic concrete (HMAC). One concern that the geosynthetic users should keep in mind is future rehabilitations as any anticipated milling of HMAC layers must avoid RAP contamination and possible fouling of milling equipment.

4.3. Geosynthetics in Flexible Pavements

Geosynthetics are used for tensile reinforcement of soil structures, such as retaining walls, embankments, and unpaved roads. In the unpaved road, a geosynthetic has been identified to function as a separator for the aggregates and foundation soils, and also to reinforce the subgrade.

The idea of incorporating geosynthetics to provide reinforcement in flexible pavements was started and developed in the two decades. Since this time, several laboratory based studies have been conducted to test the performance of the flexible pavement systems reinforced with geosynthetics. (Perkins *et al.*, 1999). By increasing pavement construction materials and construction costs, and compulsive environmental protection requirements make it important to inquire of finding alternative construction methods with longer service life but at the same time cost efficient (Leng, 2002).

Geosynthetics provide tensile reinforcement through frictional interaction with base course materials, thereby reducing applied stresses on the subgrade and preventing rutting caused by subgrade overstress. By improving the performances of the pavement structure, geosynthetic incorporation can help extend the service life of the system, or reduce the base course thickness such that a pavement of equal service life is constructed. Benefits of reducing base course thickness are realized if the cost of the geosynthetic is less than the cost of the reduced base course material, and construction associated with a reduced base thickness (Leng, 2002).

The use of geosynthetics for reinforcement when placed at the bottom or within the base course aggregate layer of a flexible pavement generally provides benefit by improving the service life and/or providing equivalent performance in common with reduced structural section (Perkins, 2001).

Geosynthetic materials are increasingly being used as reinforcement for cracking and rutting occurrence in asphalt concrete pavements. Methods for controlling reflective cracking and extending the life of overlays consider the importance and effectiveness of overlay thickness and proper asphalt mixture specification. Sometimes, by increasing overlay thickness and modifications in asphalt mixture might not provide satisfying result for crack prevention. The "solutions" is found to be either marginally effective or extremely costly. The most basic way to slow down the reflective cracking is to increase the overlay thickness. In general, as the overlay thickness increases, its resistance to reflective cracks increases. Limits on the thickness of an overlay are the expense of asphalt and the increase in the height of road structure. (Shukla and Yin, 2004)

The crack resistance of the overlay can also be developed via interlayer systems. An interlayer is a layer between the old pavement and new overlay, or within the new overlay to create an overlay system.

Geosynthetic reinforcement installed at the surface course/base course interface effectively increased the rutting resistance of an asphalt pavement due to this would be beneficial in increasing the service life of the asphalt pavement. Geosynthetic reinforcement provided a more uniform load distribution and a decrement in the rut depth both at the surface asphalt course and the granular base aggregate course. (Wassage *et al.*, 2004)

4.3.1. Geosynthetics in Hot Mix Asphalt (HMA) Applications

Geosynthetics should be classified according to their using target in engineering structures. (Table 4.1). The main function of geogrids in an HMA application is to retard the occurrence of reflective cracking. In evaluating the appropriateness of use, cracking in the existing structure should be limited to cases in which the crack faulting does not fluctuate significantly with traffic loading and crack width does not fluctuate significantly with temperature differentials. The pavement should be structurally sound with existing cracks limited to less than 9 mm width. Hence, low to moderate levels of alligator cracking, or random cracking may benefit from application of grids in HMA, whereas widely spaced thermal cracking or underlying rocking/faulted Portland cement concrete (PCC) slabs will probably not benefit. It is necessary to repair localized, highly distressed/weak areas and apply a level-up course of HMA. Where rutting exceeding 12 mm. exists, milling prior to applying the level-up should be considered. A minimum 5 cm surfacing course is recommended. Installation of this type of product has proven to be problematic and will result in premature failure (fatiguing) of the surfacing overlay where a lack of bonding (surface to grid to level-up) occurs. It is highly recommended that the manufacturer's installation procedures be strictly followed and that a manufacturer's representative be present during the planning and construction process.

Table 4.1. Geosynthetic Type and Their Function

Geosynthetic Types	Separation	Reinforcement	Filtration	Drainage	Lining
Geotextiles	√	√	√	√	
Geogrid		√			
Geonet				√	
Geomembrane					√
Geosentetic Clay Liner					√
Geopipe				√	
Geofoam	√				
Geocomposite	√	√	√	√	√

Fabrics, Composites, and Membranes provide a moisture barrier in addition to varying degrees of resistance to reflective cracking. Application guidelines are similar to those recommended above for the geogrid. The impermeable qualities of these products can be a double-edged sword in that they prevent trapped moisture within the structure from transpiring out. This may result in debonding of HMA layers and/or stripping of HMA layers below the product, especially if the lower mixes are moisture susceptible. Also, if the surfacing overlay is permeable and surface moisture can not readily escape the section laterally (mill and inlay technique is especially prone), stripping of the surface mix may also occur. It is duty upon users of these products to insure laboratory testing is performed to determine HMA stripping susceptibility of existing mixes (highway cores) and the proposed level-up and overlay mixes.

4.3.2. Geosynthetics in Pavement Bases (non-HMA Applications)

Geosynthetics are placed in pavement bases to perform one or more of the following functions:

- Reinforcement
- Separation
- Filtration

Base reinforcement results from the addition of a geogrid or composite at the bottom or within a base course to increase the structural or load-carrying capacity of a pavement system by the transfer of load to the geosynthetic material.

The primary mechanism associated with this application is lateral restraint or confinement of aggregates in the base. Where very weak subgrades exist, geosynthetics can increase the bearing capacity by forcing the potential bearing capacity failure surface to develop along alternate, higher strength surfaces. Geogrids may also be considered for use in locations where chemical stabilization of the subgrade is not desirable due to possible reaction with sulfates in the subgrade, or not practical because of expedited construction concerns, particularly in urban settings. There have been assertions that the resultant increase in restraint or confinement should allow for design of thinner structures using these products versus structural designs which do not, however their benefits may only be noticeable over the long term and there appears to be an absence of long-term controlled monitoring.

Geosynthetics used for separation have classically been applied to prevent subgrade soil from migrating into the unbound base (or subbase), or to prevent aggregates from an unbound base (or subbase) from migrating into the subgrade. A small amount of fines introduced into the granular base can significantly reduce the internal friction angle and render the flex base weaker. Potential for these circumstances increases where wet, soft subgrades exist. Typically, a geocomposite will be used for this application, placed at the subgrade/unbound base interface (Koerner, 1986). Geotextile separators act to maintain permeability of the base materials over the life of the section, and they allow the use of more open-graded, free-draining base and subbase materials. Another form of separation is being increasingly explored as a high potential for reflective cracking originating in the subgrade or chemically-bound base. A grid or composite is used to dissipate stresses induced by the opening crack. Longitudinal edge cracking is particularly an issue in areas where moderate to high plasticity index (PI) soils are exposed to prolonged cycles of wetting and drying. Geogrids will typically be employed at the subgrade/bound base interface, or if a flex base is placed above a bound base (e.g., full-depth reclamation/recycling [FDR] projects), the grid may be placed at this location. Grids should be a minimum of 10-ft. wide to reduce the potential for longitudinal cracking due to edge drying.

The function of filtration is to allow for in-pavement moisture transfer, but restrict movement of soil particles; hence composites or fabrics that are placed for the classical purpose of separation will usually incorporate this function as well.

4.4. Functions and Installation of Paving Geosynthetics

Geosynthetics are being increasingly used at the asphalt overlay base level to enhance the overall performance of the paved roadways. Pavements are civil engineering structures used for the purpose of operating wheeled vehicles safely and economically. Paved roadways that include the carriageways and the shoulders have been constructed for more than a century. Their basic design methods and construction techniques have undergone some changes, but the development of geosynthetics in the past three decades has provided the strategies for enhancing the overall performance of the paved roadways. Various levels of government, in most of the countries, devote unprecedented time and resources to roadway construction, maintenance and repair. Efforts are also being made to apply newfound technology to old pavement problems. Commonly a paved road becomes a candidate for maintenance when its surface shows significant cracks and potholes.

Cracks in the pavement surface cause numerous problems, including riding discomfort for the users, reduction of safety, infiltration of water and subsequent reduction of the bearing capacity of the subgrade and Pumping of soil particles through the crack.

Moreover, progressive degradation of the road structure in the vicinity of the cracks due to stress concentrations.

The construction of asphalt overlays is the most common way to renovate both flexible and rigid pavements. Most overlays are done predominantly to provide a waterproofing and pavement crack retarding treatment. A minimum thickness of the asphalt concrete overlay may be required to provide an additional support to a structurally deficient pavement. An asphalt overlay is at least 25 mm thick and it is placed on top of the distressed pavement. Overlays are economically practical, convenient, and effective. The cracks under the overlay rapidly propagate through to the new surface. This phenomenon is called reflective cracking, which is major drawback of asphalt overlays. Because asphalt overlays are otherwise an excellent option, research and development has focused on preventing reflective cracking.

Reflective cracks in an asphalt overlay are basically a continuation of the discontinuities in the underlying damaged pavement. When an overlay is placed over a crack, the crack grows up to the new surface. The causes of crack formation and enlargement in asphalt overlays are numerous but the mechanisms involved may be categorized as: traffic induced, thermally induced and surface initiated (Ingold, 1994).

Surface cracking in overlays can occur from traffic induced fatigue as a result of repeated bending condition in the pavement structure or shear effect causing the pavement on one side of a crack (in the old layer) to move vertically relative to the other side of the crack during the traffic movement. High axle loads or increased traffic can further increase the stresses and strains in the pavement that lead to surface cracking. In the case of an asphalt overlay, on top of a concrete pavement, cracks may be reflected to the overlay as the concrete slabs expand and contract under varying temperatures.

The expansion and contraction of the overlays and upper asphalt layers can lead to tension within the surfacing which can also lead to surface cracking. The stresses are at their maximum at the pavement surface where the temperature variation is the greatest. In this case, the cracks are initiated at the surface and propagate downwards. It should be noted that the term ‘reflective cracking’ is often used to describe all these types of cracking.

Methods for controlling reflective cracking and extending the life of overlays consider the importance and effectiveness of overlay thickness and proper asphalt mixture specification. Asphalt mixes have been improved and even modified by adding a variety of materials. In the past a number of potential solutions have also been evaluated including unbound granular base “cushion courses” and wire mesh reinforcement. All have found to be either marginally effective or extremely costly. The most basic way to slow down the reflective cracking is to increase the overlay thickness. In general, as the overlay thickness increases, its resistance to reflective cracks increases. Limits on the thickness of an overlay are the expense of asphalt and the increase in the height of road structure.

Asphalt additives do not stop reflective cracking, but do tend to slow down the development of cracks and convert a large crack in the old pavement into a multiple small cracks in the overlay. Mixing glass fibers, metal fibers, or polymers in asphalt prior to paving creates modified or optimized asphalt, which is not always specified because it is much more expensive than unimproved asphalt and the relationship between the investment and the improvement has not been established.

The crack resistance of the overlay can also be enhanced via interlayer systems. An interlayer is a layer between the old pavement and new overlay, or within the new overlay to create an overlay system. The benefits of the geosynthetic interlayer include the following:

- Waterproofing the pavement;
- Delaying the appearance of reflective cracks;
- Lengthening the useful life of the overlay;
- Added resistance to fatigue cracking; and
- Saving up to 50 mm of overlay thickness;

4.5. Functions of Paving Geosynthetics

A geosynthetic layer, especially a geotextile layer, is used beneath asphalt overlays, ranging in thickness from 25 to 100 mm, of asphalt concrete (AC) and Portland cement concrete (PCC) paved roads. The geotextile layer is generally combined with asphalt sealant, or tack coat to form a membrane interlayer system known as a paving fabric interlayer. Figure 4.1 shows the layer arrangement in paved roads with a paving fabric interlayer. When properly installed, a geotextile layer beneath the asphalt overlay mainly functions are fluid barrier (if impregnated with bitumen, that is, asphalt cement), protecting the underlying layers from degradation due to infiltration of road-surface moisture; and cushion, that is, stress-relieving layer for the overlays, retarding and controlling some common types of cracking, including reflective cracking. (Holtz *et al.*, 1997)

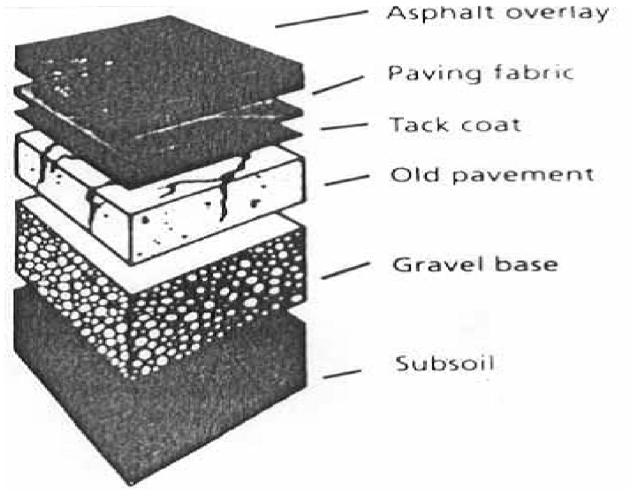

Figure 4.1. Typical cross-section of a road with a paving fabric (Barazone, 2000)

A paving fabric, in general, is not used to replace any structural deficiencies in the existing pavement. However, the above functions combine to extend the service life of overlays and the roadways with reduced maintenance cost and increased pavement serviceability. (Shukla and Yin, 2004).

The pavements typically allow 30 – 60 % of precipitation to infiltrate and weaken the road structure. The fluid barrier function of the bitumen impregnated geotextile may be of considerable benefit if the subgrade strength is highly moisture sensitive. In fact, excess moisture in the subgrade is the primary cause of premature road failures. Heavy vehicles can cause extensive damage to roads, especially when the soil subgrade is wet and weakened. The pore water pressure can also force the soil fines into the voids in the subbase/base, weakening them if a geotextile is not used as a separator/filter.

Therefore, efforts should be made to keep the soil subgrade at fairly constant and low moisture content by stopping moisture infiltration into the pavement and providing proper pavement drainage.

A stress-relieving interlayer retards the development of reflective cracks in the overlay by absorbing the stresses induced by underlying cracking in the old pavement. The stress is absorbed by allowing slight movements within the paving fabric interlayer inside the pavement without distressing the asphalt concrete overlay significantly. In fact, the addition of a stress relieving interlayer reduces the shear stiffness between the old pavement and the new overlay, creating a buffer zone (or break layer) that gives the overlay a degree of independence from movements in the old pavement. Pavements with paving fabric interlayers also experience much less internal crack developing stress than those without. This is why fatigue life of a pavement with a paving fabric interlayer is many times that of a pavement without, as shown in Figure 4.2. A stress-relieving interlayer also waterproofs the pavement, so when cracking does occur in the overlay, water cannot worsen the situation.

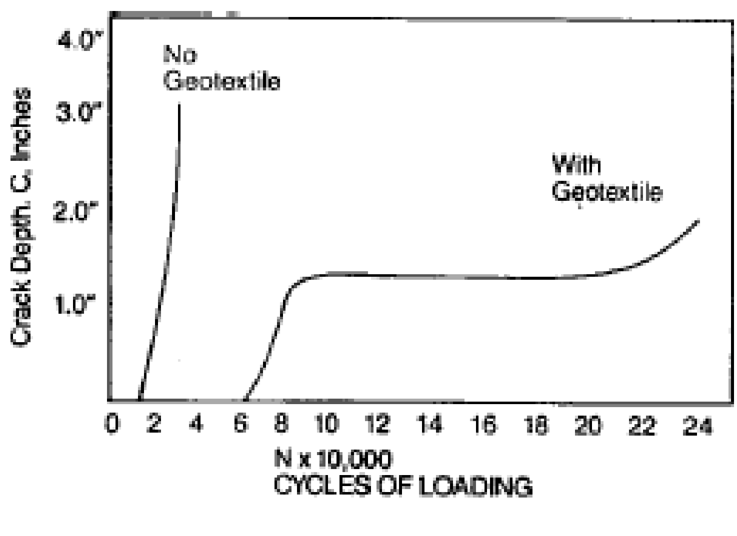

Figure 4.2. Fatigue response of asphalt overlay (Shukla and Yin, 2004)

4.6. Installation of Paving Geosynthetics

A paving fabric interlayer system is looked upon as an economical tool, which effectively solves general pavement distress problems. It is easy to install and readily complements any paving operation. The ideal time to place a paving fabric interlayer system is in the very early stages of hairline cracking in a pavement. It is also appropriate in new pavement construction to provide a waterproof pavement from day one. The installation of a paving fabric generally follows the same pattern wherever it is used. There are four basic steps in the proper installation of an overlay system with a geosynthetic interlayer. Surface preparation is followed by the application of tack coat, installation of the geosynthetic, and finally the placement of the overlay (Marienfeld and David, 1994). These steps along with general guidelines are described below, incorporating the experiences of the authors as a consultant.

Step 1 – Surface Preparation: The site surface is prepared by removing loose material and sharp/angular protrusions, and sealing cracks, as necessary. The prepared surface should be leveled, dry, and free of dirt, oil and loose materials. Cracks, 3 mm wide or greater, should be cleaned with pressurized air or brooms and filled with a liquid asphalt crack sealant. This will prevent the tack coat from entering the cracks and reducing available tack for saturation of the fabric. Very large cracks should be filled with a hot or cold asphalt mix. Commercial crack filler can also be used. Cracks should be level with the pavement surface and not overfilled. If the quality of the existing pavement is poor, a leveling course of asphalt concrete is placed over it prior to the placement of the paving fabric interlayer system. On existing cement concrete pavements, a layer of asphalt concrete should be provided before laying the fabric. The surface on which a moisture barrier interlayer is placed must have a grade which will drain water off the pavement.

Step 2 -Tack Coat Application: Proper application of tack coat is crucial; mistakes can lead to early failure of the overlay. Straight paving-grade bitumen is the best and the most economical choice for the paving fabric tack coat. Cutbacks and emulsions which contain solvents should not be used for tack coat; if they are used, they must be applied at a higher rate and allowed to cure completely. Minimum air and pavement temperature should be at least 100 °C or more for placement of tack. The temperature of tack coat should be sufficiently high to permit a uniform spray pattern. It should be spread at between 140 °C and 160 °C, to permit uniform spray and to prevent damage to paving fabric. The target width of tack coat application should be equal to the paving fabric width plus 150 mm. Tack coat should be restricted to the area of immediate fabric lay-down.

Besides proper quantity, uniformity of the sprayed asphalt cement (bitumen) tack coat is of great importance. Application of hot bitumen should be done preferably by means of a calibrated distributor spray bar for better uniformity. Hand spraying and brush application may be used in locations of fabric overlap. When hand spraying, close attention must be paid to spraying a uniform tack coat.

Step 3 - Geosynthetic Placement: The paving fabric is placed prior to the tack coat cooling and loosing tackiness. The paving fabric is placed onto the tack coat with its fuzzy side down leaving the smooth side up using mechanical or manual lay-down equipment capable of providing a smooth installation without wrinkling or folding. Today most paving fabric is applied using tractor-mounted rigs. Slight tension can be applied during paving fabric installation to minimize wrinkling. However, stretching is not recommended, because it will reduce the thickness, changing the bitumen retention properties of the fabric. Too little elongation may result in wrinkles. Too much elongation produces excessive stretch, thinning the geosynthetic so that it may not be thick enough to absorb the tack coat, leaving excess that may later bleed through the bituminous concrete on a hot day. Wrinkles and overlaps can cause cracks in the new overlay if not properly handled during construction process. Overlaps and all overlapped wrinkles for fabric and grid composites should have an additional tack coat placed. Tack must be sufficient to saturate the two layers and make a bond. If not done correctly, a slip plane may exist at each overlapped joint, resulting in a possible crack of the asphalt from the fabric. Overlaps should be no more than 150 mm on longitudinal and transverse joints. This is different for grids, and each manufacturer has its own recommendations for overlaps. Paving multiple lanes has inherent installation problems. It is best to install in one lane and pave it for traffic prior to installing in another lane. Leave 150 mm of fabric unpaved for overlap on the adjacent panel of fabric to be installed.

A paving reinforcement geogrid is installed into a light asphalt binder or it may be attached to the existing pavement by mechanical means (nailing) or by adhesives, preventing the geogrid from being lifted by paving equipment passing over it. When a composite of geogrid and geotextile is installed, the tack coat is applied the same way as would be applied when geotextile used alone.

Installing geosynthetic around curves without producing excessive wrinkles is the most difficult task for installers of paving synthetics. However, with the proper procedures, it can be accomplished with ease. Attempt should not be made to roll the geosynthetic around a curve by hand. It will wrinkle too much. Placing fabric around a limited curve with machinery is preferable. Some minor wrinkles may occur. Grids have low elongation and thus do not stretch in curves. In most cases, the grid will need to be installed by hand or in short sections by machine to avoid wrinkles (Barazone, 2000). Excess tack coat, which bleeds through the paving fabric, is removed by spreading hot mix, or sand should be spread over it. Any traffic on the geosynthetic should be carefully controlled. Sharp turning and braking may damage the fabric. For safety reasons, only construction traffic should be allowed on the installed paving fabric. (Shukla and Yin, 2004).

Step 4 - Overlay Placement: All areas with paving geosynthetic placed are paved on the same day. In fact, asphalt concrete overlay construction should be done immediately after the placement of paving geosynthetic. Asphalt can be placed by any conventional means. Compaction should take place immediately after dumping in order to ensure that the different layers are bonded together. The temperature of asphalt concrete overlay should not exceed about 160 ºC to avoid damage to the paving fabric.

Overlays should not be attempted with its temperature less than 120 ºC and air temperature less than 10 ºC. Adequate overlay thickness, if used, generates enough heat to draw the tack coat up, into and through the paving fabric, thus making a bond. In fact, the heat of the overlay and the pressure applied by its compaction force the tack coat into the paving fabric and complete the process. If sufficient residual heat after compaction is not present, the bonding process is disrupted, the results being slippage and eventual overlay failure. Thickness of the asphalt overlay should not be less than 40 mm. Compacting the asphalt concrete immediately after placement helps to concentrate the heat and supply pressure to start the process of the bitumen moving up into and through the fabric. This is very important when using a thinner overlay as it cools more rapidly. In cold weather, a thicker overlay may be necessary to achieve the same objective.

A paving fabric interlayer can also be used beneath seal coat or other thin surface applications. In such applications, there is not sufficient heat applied to reactivate the asphalt sealant. Therefore, the installed paving fabric must be trafficked or rolled with a pneumatic roller to push the fabric completely into the asphalt sealant. Sand can be applied lightly to avoid bitumen tackiness during trafficking. Once the paving fabric has absorbed the asphalt sealant, the seal surface treatment is applied exactly as it would be over any road surface. It is suggested that the first-time users of paving fabric interlayer should obtain help from the paving fabric manufacturers, keeping in view the site and material variables. It should be noted that choosing proper application sites for the paving geosynthetic is a function of the existing pavement's structural integrity and crack types – not its surface condition. For successful performance, proper installation must occur on a pavement without significant vertical or horizontal differential movement between cracks or joints and without local deflection under design loading (Marienfeld and David, 1994).

4.7. Geosynthetic Reinforced Flexible Pavement Performance

The concept of using geosynthetics to provide reinforcement in flexible pavement systems was introduced and developed in the late 1980's. Since this time, numerous experimentally based studies have been conducted to examine the performance of flexible pavement systems reinforced with geosynthetics. Many of these studies have been summarized by Perkins and Ismeik (1997 a, b) and more recently by Berg et al. (2000).

The use of geosynthetics for reinforcement when placed at the bottom or within the base course aggregate layer of a flexible pavement generally provides benefit by improving the service life and/or providing equivalent performance with a reduced structural section. The principal categories of pavement distress are rutting due to permanent deformation in the base and subgrade layers, asphalt concrete fatigue cracking, asphalt concrete low temperature cracking, rutting due to asphalt concrete high temperature flow, surface raveling, loss of skid resistance, contamination and/or saturation of base aggregate layers and frost heave. Base reinforcement is applicable for the support of vehicular traffic over the life of the pavement and is designed to address the pavement distress mode of permanent surface deformation or rutting and possibly asphalt fatigue cracking.

The principle mechanism responsible for reinforcement in paved roadways is one generally referred to as base course lateral restraint and is schematically illustrated in Figure 4.3.

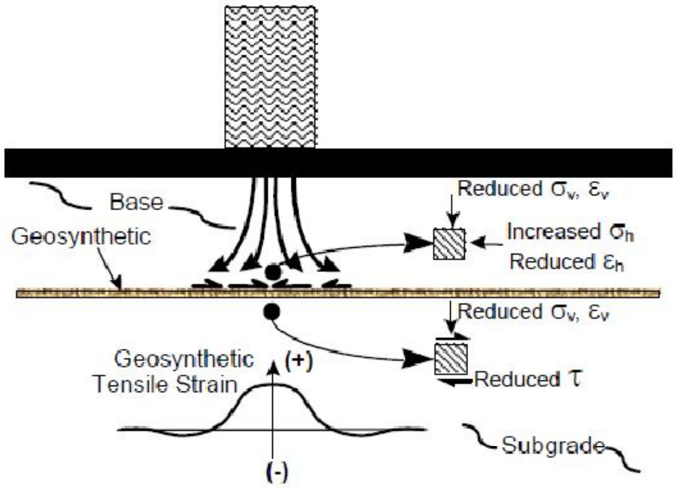

Figure 4.3. Schematic illustration of base reinforcement mechanisms.

Vehicular loads applied to the roadway surface create a lateral spreading motion of the base course aggregate. Tensile lateral strains are created in the base below the applied load as the material moves down and out away from the load. The geosynthetic restrains the base thus reducing or restraining this lateral movement. The term lateral restraint involves several components of reinforcement including: (i) restraint of lateral movement of base aggregate; (ii) increase in modulus of base aggregate due to confinement; (iii) improved vertical stress distribution on the subgrade due to increased base modulus; and (iv) reduced shearing in the top of the subgrade. These mechanisms, most of which were experimentally verified in the study by Perkins (1999a), lead to a reduction in vertical strain in the base and subgrade layers.

The benefits of reinforcement on the design of flexible pavements are generally expressed in terms of an extension of life of the pavement or an allowable reduction in base course thickness. An extension of life of the pavement is typically expressed in terms of a Traffic Benefit Ratio (TBR). TBR is defined as the ratio of the number of traffic loads between an otherwise identical reinforced and unreinforced pavement that can be applied to reach a particular pavement permanent surface deformation. TBR indicates the additional amount of traffic loads that can be applied to a pavement when a geosynthetic is added, with all other pavement materials and geometry being equal.

The benefit of reducing the base aggregate thickness is typically defined by a Base Course reduction Ratio (BCR). BCR defines the percentage reduction in the base course thickness of a reinforced pavement such that equivalent life is obtained between the reinforced and the unreinforced pavement with the greater aggregate thickness. Since TBR as defined above does not involve a reduced base course layer, the resulting TBR corresponds to a BCR of 0 and is denoted as TBRBCR=0. Similarly, the BCR defined above is for equal life or for a TBR of 1 and is denoted by BCRTBR=1. Combinations of BCR and TBR are possible if the base course thickness is not reduced by the full amount yielding equivalent life. A number of combinations of TBR between 1 and TBRBCR=0 and BCR between 0 and BCRTBR=1 are possible as schematically illustrated in Figure 4.4

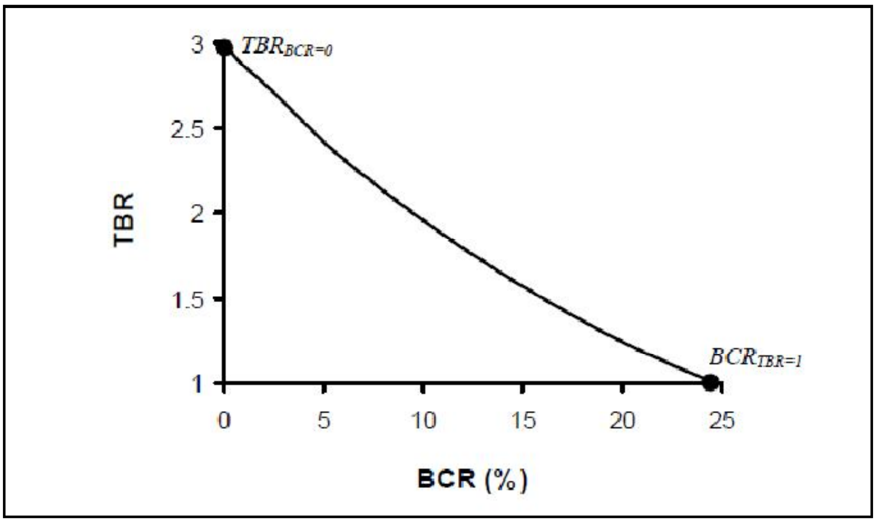

Figure 4.4. Schematic illustration of combinations of TBR and BCR.

Based on the studies reviewed in Berg et al. (2000), values of TBRBCR=0 up to 10 can generally be anticipated for roadways resting on a subgrade with a California bearing ratio (CBR) Values of BCRTBR=1 up to 50 % can be anticipated for subgrade CBR values lying between 3 and 8. For subgrade CBR less than 3, the margin of safety for reduction of base course thickness becomes smaller and designs using a BCR must be treated with caution. Existing information to date indicates that reinforcement benefit begins to diminish quickly for subgrade CBR values greater than 8.

Tables 4.2 and 4.3 from Berg et al. (2000) provide a more detailed listing of the variables that are believed to influence reinforcement benefit for flexible pavements.

Table 4.2. Qualitative application guidelines for geosynthetic type (Berg et al., 2000).

Roadway Design Conditions		Geosynthetic Type					
		Geotextile		Geogrid		GG-GT Composite	
Subgrade	Base/Subbase Thickness (mm)	Nonwoven	Woven	Extruded	Knitted or Woven	Open-Graded Base	Well-Graded Base
CBR<3 (MR<30 Mpa)	150-300	**4**	**X**	**X**	<	**X**	É
	>300	**4**	**4**	<	>	>	É
3≤ CBR ≤ 8 (30≤ MR ≤ 80)	150-300	**6**	<	**X**	<	**X**	É
	>300	**6**	**6**	<	>	>	É
CBR>8 (MR<80 Mpa)	150-300	+	+	<	>	>	É
	>300	+	+	+	+	+	É

Key: X - usually applicable < - applicable for some (various) conditions
+ - usually not applicable > - insufficient information at this time É : see notes

Notes: **1**.Total base or subbase thickness with geosynthetic reinforcement. Reinforcement may be placed at bottom of base or subbase, or within base for thicker (usually>300mm) thicknesses. Thicknesses less than 150 mm not recommended for construction over soft subgrade. Placement of less than 150mm over a geosynthetic not recommended.
2. For open-graded base or thin base over wet, fine-gained subgrade, a separation geotextile should be considered with geogrid reinforcement.
3. Potential assumes base placed directly on subgrade. A subgrade also may provide filtration.
4. Reinforcement usually applicable, but typically addressed as subgrade stabilization application.
5. Geotextile component of composite likely is not required for filtration with a well graded base course; therefore, composite reinforcement usually not applicable.
6. Separation and filtration application; reinforcement usually not applicable.
7. Usually applicable when placed up in the base course aggregate. Usual not applicable when placed at the bottom of the base course aggregate.

Table 4.3. Variables influencing reinforcement effect (Berg et al., 2000).

Pavement Component	Variable	Range from Test Studies / Remarks	Condition where Reinforcement Appears to Provide Most Benefit
Geosynthetic	Structure	Rigid (extruded) and flexible (knitted and woven) geogrids, woven and nonwoven geotextiles, geogrid-geotextile composites	See Table 4.2
	Modulus (@ 2% and/or 5% strain)	100 kN/m to 750 kN7m	Higher modulus improves potential for performance
	Location	Geogrid	Moderate load (≤80kN axle load): Bottom of thin bases (≤250 mm), middle for thick (>300mm) bases Heavy load (>80kN axle load): Bottom for thin bases (≤300 mm), middle for thick bases (>350 mm)
		Geotextile	Bottom of base, on the subgrade
		Geogrid-geotextile composite	Bottom of pen-graded base OGB
	Surface	Slick versus rough	Rough
	Geogrid Aperture	15 mm to 64 mm	>D_{50} of adjacent base/subbae
	Aperture Stiffness	Rigid to flexible	Rigid
Subgrade Condition	Soil Type	SP, SM, CL, CH, ML, MH, Pt	No relation noted
	Strength	CBR from 0.5 to 27	CBR ≤ 8 (M_R ≤ 80 Mpa)
Subbase	Thickness	0 to 300 mm	No subbase
	Practice Angularity	Rounded to angular	Angular
Base	Thickness	40 mm to 640 mm	≤ 250 mm for moderate loads
	Gradation	Well graded to poorly graded	Well graded
	Angularity	Angular to subrounded	Angular
Pavement	Type	Asphalt, concrete, unpaved	Asphalt and unpaved
	Thickness	25 mm to 180 mm	75 mm
	Resilient Modulus	Not typically measured	Unknown
Design	Pavement Loading	200 kPa to 1800 kPa	Does not perform on significantly under-designed pavements
Construction	Pre-rutting	None in lab to pre-rutted in field	Unknown

From Tables 4.2 and 4.3, several critical design variables that influence the effect of the reinforcement are noted. The strength and/or stiffness of the subgrade appear to be a critical design parameter as discussed above.

The thickness of the structural section appears to have a significant impact on reinforcement benefit. Very few studies are available that used a thickness for the asphalt concrete (AC) greater than 75 mm. Several studies have shown that as the thickness of the base course aggregate becomes greater than approximately 250 mm, reinforcement benefit begins to decrease. It should be noted, however, that several studies have demonstrated significant values of TBR for base aggregate thicknesses as great as 400 mm. In contrast to a reduction of reinforcement benefit for thick structural sections, several studies have demonstrated that sections that are designed for a low number of traffic passes (i.e. under designed sections) are not appreciably influenced by base reinforcement.

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4.7.1. The Effect of geosynthetic reinforcement on Asphalt Concrete

Asphalt concrete is a type of material which is produced by compacting a special mixture, consisting of crushed rock or gravel, sand or crushed stone, filler and bitumen, all selected in relevant proportions. Asphalt concrete acquires the required physical and mechanical qualities only after compaction. Under different environmental conditions asphalt concrete can have different forms of physical existence:

i). Plastic;
ii). Viscoelastic;
iii). Elastic.

The theory of elasticity and plasticity describes the qualities of asphalt concrete exclusively at some selected points of states of existence and does not provide a complete view of asphalt concrete operation. It is rheology, a science about the fluidity of materials, that gives the most complete and precise description of the asphalt concrete operation. When making the calculating model of asphalt concrete, rheology makes use of dependences of several mechanical models. For investigating the asphalt concrete as viscoelastic material, usually Burgers' model is considered the most appropriate and described by the following dependence:

$$\varepsilon = \frac{\sigma}{E_0}\left(1 + \frac{t}{T_0}\right) + \frac{\sigma}{E_1}\left[1 - exp\left(-\frac{t}{T_1}\right)\right] \qquad (4.1)$$

Burgers' Equation

Where σ stands for stresses, E_0- modulus of Elasticity of an element series. E_1- modulus Elasticity of an isolated element. T_0, T_1-time of relaxation of asphalt concrete.

Reinforcing is a structural measure increasing strength. Reinforcing road pavement is concerned with increasing pavement resistance to a variety of stresses and improving its strength characteristics. It refers to mobilizing stresses in some layers, more specifically, in geosynthetics and higher values of some selected parameters. When reinforcing pavement by geosynthetics, the rheological model of asphalt pavement changes. On the basis of Burgers' equation, reflecting the creep compliance of asphalt concrete (AC) , the following assumptions can be made:

- Reinforcing AC by geosynthetics influences its modulus of elasticity (E),
- Reinforcing asphalt concrete by geosynthetics influences its viscosity (η).

The above characteristics are the key factors in deciding the resistance of asphalt concrete to shear strains. The above stress interpretation and the results of the investigation of reinforced pavement lead to a conclusion that the modulus of elasticity of asphalt concrete is influenced by reinforcement.

Asphalt concrete viscosity characterizes the period of asphalt concrete strain under shear stresses and determines asphalt concrete in one or another physical condition. Higher viscosity characterizes asphalt concrete as an elastic body and vice versa. In the elastic asphalt concrete no shear strains emerge (Laurinavičius and Oginskas ,2006).

5. LABORATORY STUDIES

5.1. General

Miscellaneous forms of full-scale track tests and laboratory- scaled wheel tracking models have been adopted to evaluate rutting potential of pavement materials. Although full-scale test sections are ideal for investigating the pavement resistance to rutting, it is costly to construct and maintain. Laboratory wheel tracking tests remain the most practical tool to study the rutting behavior of pavement materials under simulated moving traffic loads.

Measurements taken from full- scale test express more reliable result for assessment resistance ability of geosynthetics against rutting potential. Unfortunately, neither in General Directorate of Highways of Turkey nor other agencies and universities have full scale test section for rutting tests. Fortunately, results derived from this study lead municipality of Istanbul to make application in one of the important bus line, METROBUS, to solve rutting caused by braking and acceleration of buses in station sections.

Hamburg Wheel Tracking Device (HWTD) was used for rutting measurement in laboratory condition. ISFALT's (Istanbul Asphalt Plant Inc) accredited Performance Test Laboratory work in discipline and consider all principles and rules as obedience. Thanks to the great effort of ISFALT's laboratory staff, all testing stages were gone without problem. Specifications for both material and specimen preparations strictly were took into consideration.

5.2. Scope of Test

Laboratory testing stages are organized as follows, and a brief summary of each stage has been included for reference. Key questions concerned rutting test are listed before tests. Questions provide a vision for better understanding of each step of tests. Test description which stated in thesis proposal provides to define frame of concerned test and preconditions for procedure. Selected asphalt mixes are designed and tested according to European Norms (EN). Selected geosynthetics are prepared in a suitable form for test specimens. Calibrations of test instruments are checked and their output systems are examined before test.

Methodology for rutting test is defined related EN standard for HWTD according to test machine's work mentality. All specimens are prepared by using same and controlled materials stocked before these tests. A test result which has exceeded defined limitations and acceptable criteria are retested by using a new specimen. Test results and other outputs are examined and compared with themselves and noted to test diary.

5.3. Key Questions before testing

The main key questions about laboratory tests are listed before testing below as;

- What is the effectiveness of using of geosynthetics in asphalt overlays to prevent the occurrence of rutting?
- What should be ideal application for geosynthetics in relation to the pavement layer types and thicknesses for the intended use?
- What is the impact of geosynthetic reinforcement in relation to the characteristics of asphalt concrete?
- What is the benefit of geosynthetic use for cost reduction via decreasing pavement thickness?
- How much does Hamburg Wheel Tracking Device (HWTD) simulate field rutting occurrence conditions?

6. TEST DESCRIPTION

6.1. Methodology

A sufficient description of the testing methodology is given as a flow chart below.

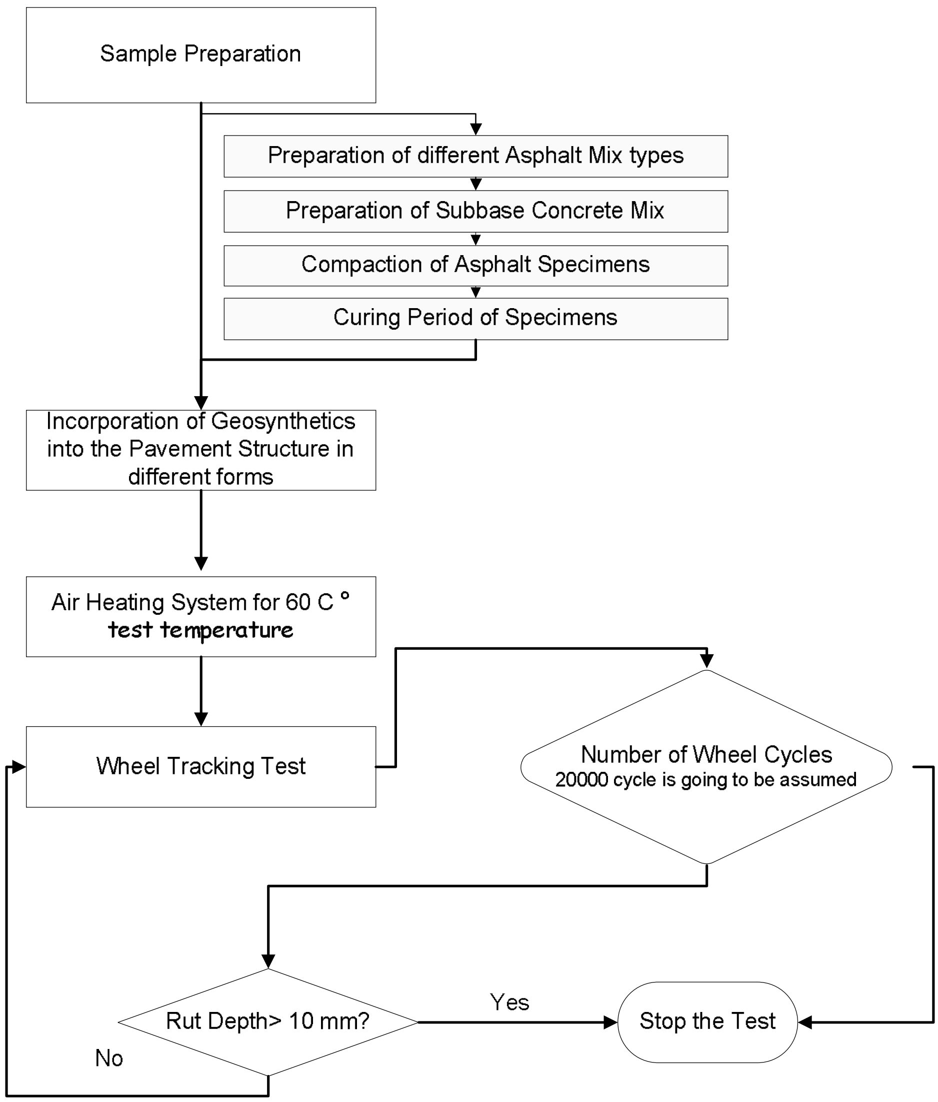


Figure 6.1. Flow Chart of rutting test using HWTD in laboratory

Wheel Tracking Test with a solid rubber-faced tire was used to obtain the data of the rut accumulation with the repetitions of loading. Both geosynthetic incorporated sandwich slab specimens and unreinforced sandwich slab specimens (control specimens) to be used in this test are 200 mm wide, 300 mm long with thickness of 100 mm. Tests were carried out at a temperature of 60 °C under the application of a constant tire weight of 70 kg for up to 10.000 load cycles (20.000 crossings). Figure 6.2 shows a typical plot from a HWTD test and the key plot parameters. The following parameters were measured and reported:

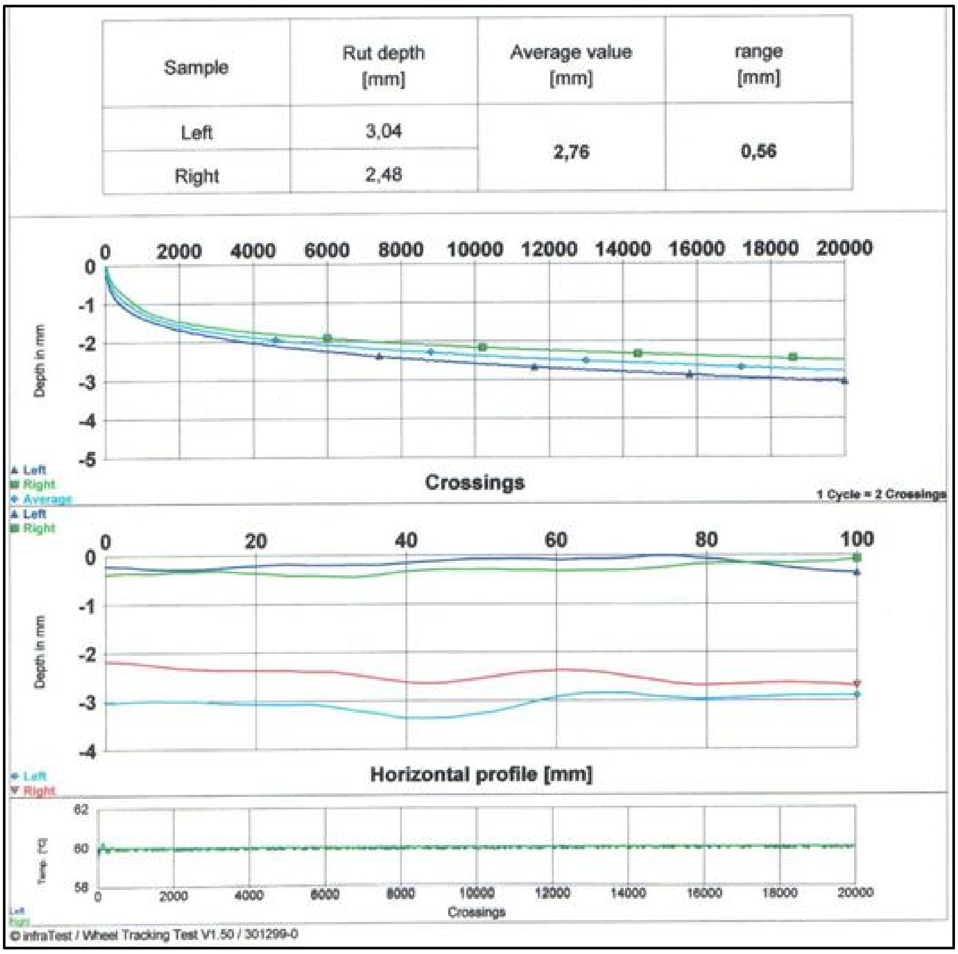


Figure 6.2. A Typical Depth versus Crossing Chart of Wheel Tracking Test

- Rut Depth: Vertical displacement of asphalt slab under rubber coated tire.
- Left Side Rut Depth Value: Rut Depth measured from left side of HWTD.
- Right Side Rut Depth Value: Rut Depth measured from right side of HWTD.
- Average Value: Average or mean rut depth measured from two sides.
- Range (mm): Difference between left and right sides rut depth values
- Horizontal Profile (mm): The profile of rut depth in horizontal section.
- Crossings: Number of Wheel passes in one way (1 Cycle = 2 Crossings).

6.2. Preparation of Test Materials

6.2.1. Mix Design

Flexible specimen elements, Stone Mastic Asphalt (SMA), Dense Graded Hot Mix Asphalt (HMA), and Dense Graded Binder HMA, were prepared according to EN standards. SMA, Dense Graded, HMA and Dense Graded Binder HMA mixes were prepared in laboratory from beginning to end. Controlled material samples were used for specimens in same group. Mix gradation of all Asphalt Specimens was designed according to EN standards.

6.2.2. DGAC-Wearing Course Mix Design

Aggregates were graded according to typical wearing course standards which generally used in Istanbul roads. Table 6.1 shows sieve analysis plot of Wearing Course DGAC. Figure 6.3 define gradation range of Dense Graded Asphalt Wearing Course as sieve analysis graph. Table 6.2 shows Marshall Design outputs for Dense Graded Asphalt Wearing Course.

Table 6.1. Wearing Course Aggregate Gradation for DGAC

Sieve No	Mixture % Pass	Mixture Formula % Pass	Specification % Pass
1’’ (25 mm)			
¾’’ (19 mm	100,0	100	100
½’’(12.5 mm)	98,8	96-100	83-100
3/8’’(9.5 mm)	86,3	82-90	70-90
4.75 mm	50,3	46-54	40-55
2.00 mm	29,4	26-32	25-38
0.425 mm	13,3	10-16	10-20
0.180 mm	9,2	6-12	6-15
0.075 mm	6,4	4-8	4-10

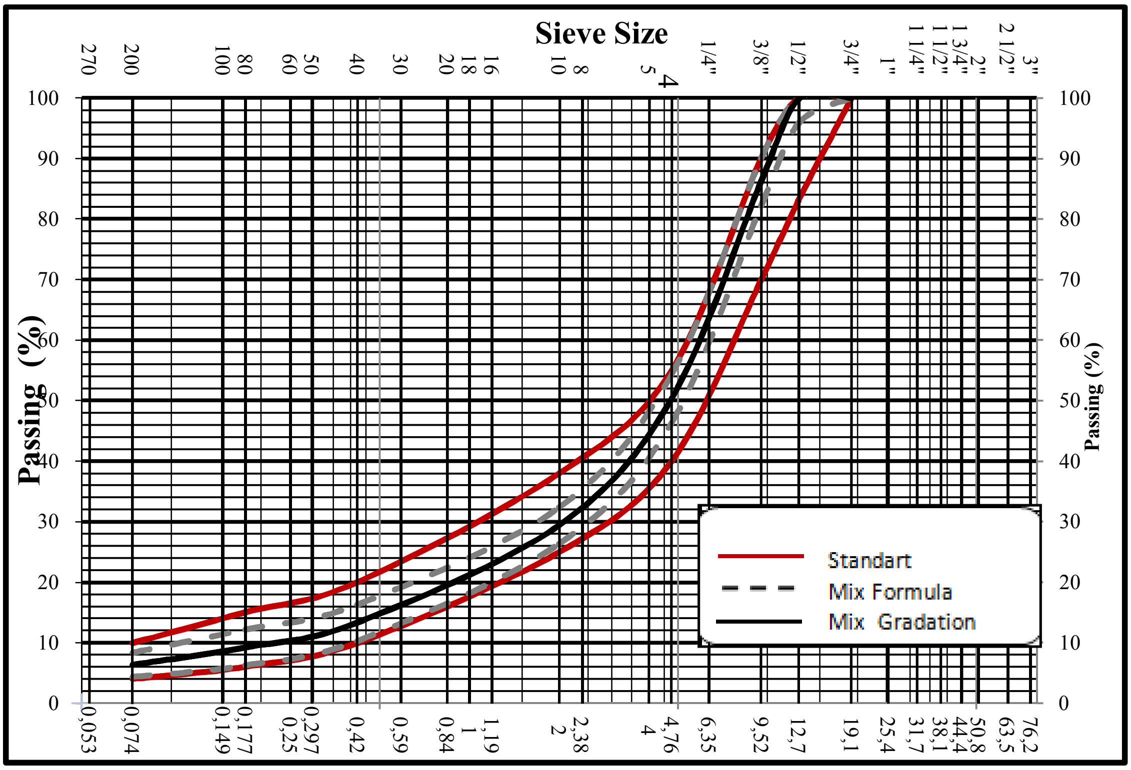


Figure 6.3. Sieve Analysis of Dense Grade Wearing Course

Table 6.2. Marshall Design outputs of DGAC- Wearing Course Asphalt

Optimum Bitumen %	4,43	Coarse Agg.Sp.Weight (gr/cm3)	2,735
Penetration	59	Fine Agg.Sp.Weight (gr/cm3)	2,715
Specific Weight (gr/cm3)	2,449	Filler Sp. Weight (gr/cm3)	2,793
Void %	4,8	Agg.Effective Sp.Weight (gr/cm3)	2,757
VFA (Voids filled with asphalt)	66	Bitumen Sp.Weight (gr/cm3)	1,021
Stability (kg)	1210	Max.Theoric Sp.Weight (gr/cm3)	2,571
Creep (mm)	2,9	Rammer Number	75
VMA (Voids in the mineral aggregate)	14,15	Rummer Heat (^{0}C)	135

6.2.3. DGAC-Binder Course Mix Design

Aggregates were graded according to typical binder course standards which generally used in Istanbul roads. Table 6.3 shows sieve analysis plot of Binder Course DGAC. Figure 6.3 define gradation range of Dense Graded Asphalt Binder as sieve analysis graph. Table 6.4 shows Marshall Design outputs for DGAC.

Table 6.3. Binder Course Aggregate Gradation for DGAC

Sieve No	Mixture % Pass	Mixture Formula % Pass	Specification % Pass
1’’ (25 mm)	100,0	100	100
¾’’ (19 mm	98,6	95-100	80 – 100
½’’(12.5 mm)	68,3	64-72	58 -80
3/8’’(9.5 mm)	54,3	50 - 58	48 – 70
4.75 mm	37,2	33 - 41	30 – 52
2.00 mm	22,8	20 - 26	20 – 40
0.425 mm	9,4	8 -12	8 – 22
0.180 mm	6,2	5 - 9	5 – 14
0.075 mm	4,4	2 - 6	2 - 7

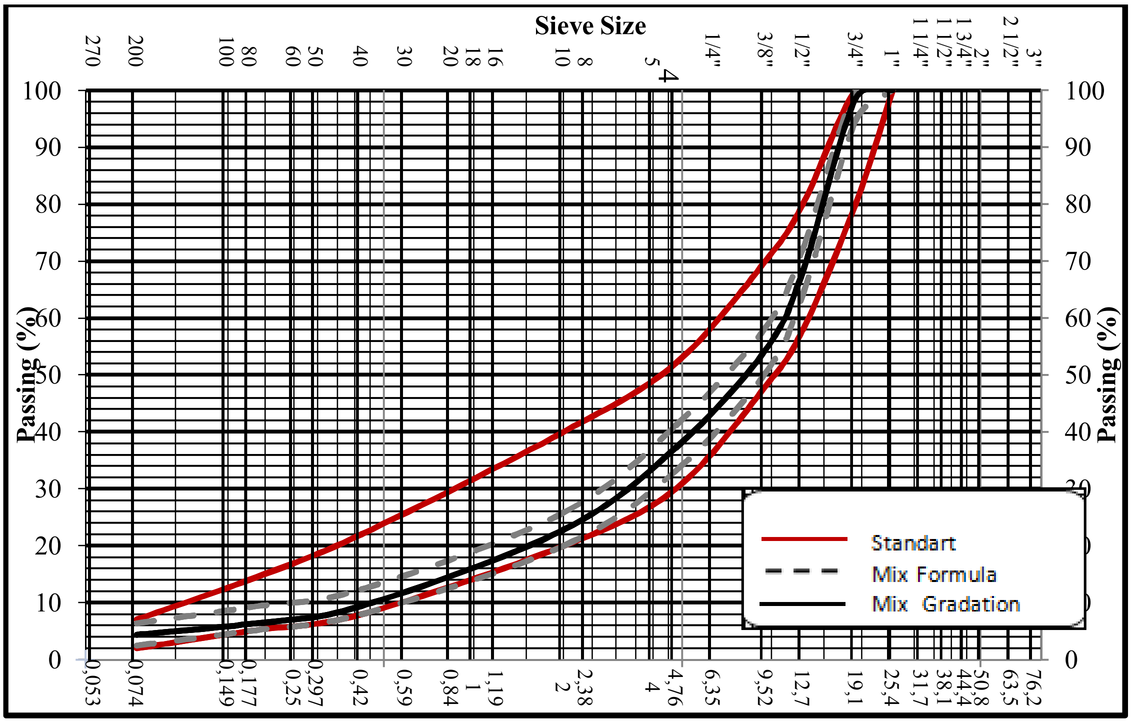


Figure 6.4. Mix Gradation of Dense Grade Binder Course Asphalt Specimen

Table 6.4. Marshall Design outputs of DGAC- Binder Course Asphalt

Optimum Bitumen %	4,43	Coarse Agg.Sp.Weight (gr/cm3)	2,735
Penetration	59	Fine Agg.Sp.Weight (gr/cm3)	2,715
Specific Weight (gr/cm3)	2,449	Filler Sp. Weight (gr/cm3)	2,793
Void %	4,8	Agg.Effective Sp.Weight (gr/cm3)	2,757
VFA (Voids filled with asphalt)	66	Bitumen Sp.Weight (gr/cm3)	1,021
Stability (kg)	1210	Max.Theoric Sp.Weight (gr/cm3)	2,571
Creep (mm)	2,9	Rammer Number	75
VMA (Voids in the mineral aggregate)	14,15	Rummer Heat (^{0}C)	135

6.2.4. GGAC- Stone Mastic Asphalt Mix Design

Stone mastic asphalt (SMA) provides a deformation resistant, durable surfacing material, suitable for heavily trafficked roads. SMA has found use in Europe, Australia, the United States, and Canada as a durable asphalt surfacing option for residential streets and highways.

SMA has a high coarse aggregate content that interlocks to form a stone skeleton that resists permanent deformation. The stone skeleton is filled with a modified bitumen and filler to which fibers are added to provide adequate stability of bitumen and to prevent drainage of binder during transport and placement. Table 6.5 shows sieve analysis plot of SMA. Figure 6.5 define gradation range of SMA as sieve analysis graph. Table 6.6 shows Marshall Design outputs for SMA. SMA is called as Gap Graded Asphalt Concrete (GGAC) because of its gap based gradation in literature.

Table 6.5. Aggregate Gradation of SMA Specimens

Sieve No	Mixture	Mixture Formula	Specification
1’’ (25 mm)			
¾’’ (19 mm			
½’’(12.5 mm)	100,0	100	100
3/8’’(9.5 mm)	90,0	90-95	90-100
4.75 mm	33,9	34-40	25-45
2.00 mm	20,4	20-26	20-30
0.425 mm	12,0	12-17	12-22
0.180 mm	9,0	9-14	9-17
0.075 mm	8,0	8-11	8-14

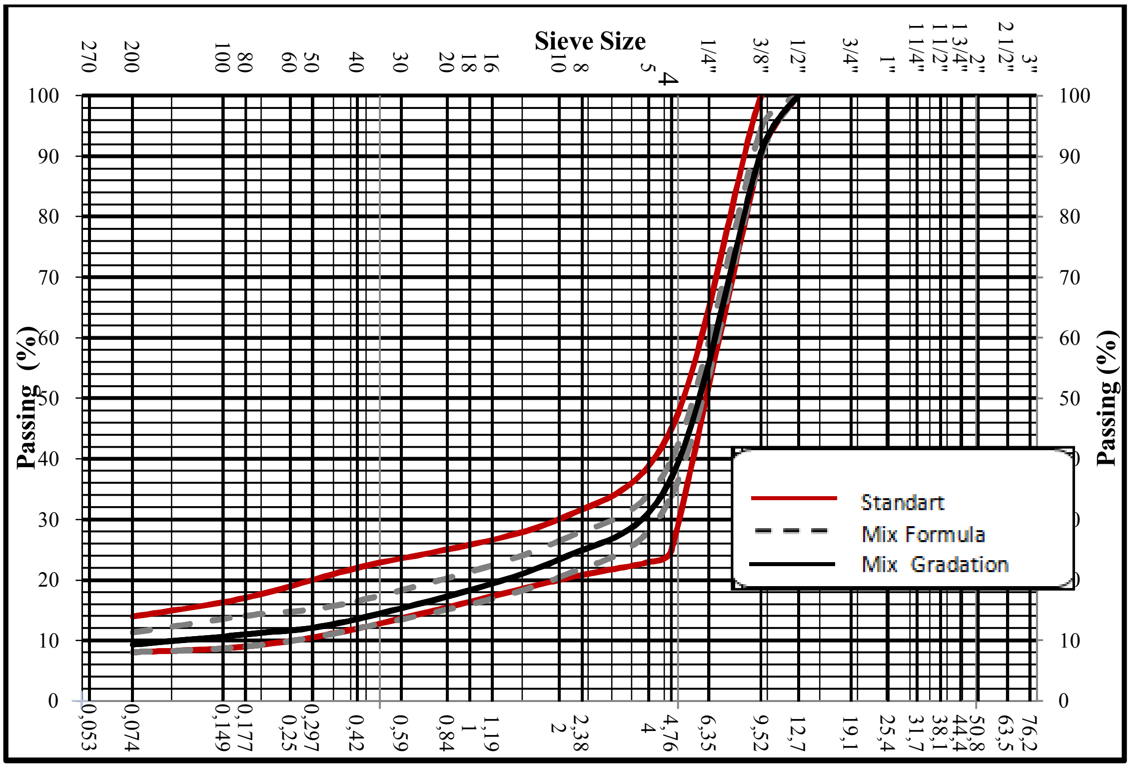


Figure 6.5. Mix Gradation of Stone Mastic Asphalt (SMA) Specimens

Table 6.6. Marshall Design outputs of GGAC- Stone Mastic Asphalt (SMA)

Optimum Bitumen %	6,65	Coarse Agg.Sp.Weight (gr/cm3)	2,805
Penetration	40	Fine Agg.Sp.Weight (gr/cm3)	2,687
Specific Weight (gr/cm3)	2,454	Filler Sp. Weight (gr/cm3)	2,793
Void %	3	Agg.Effective Sp.Weight (gr/cm3)	2,805
VFA (Voids filled with asphalt)	82	Bitumen Sp.Weight (gr/cm3)	1,016
Stability (kg)	860	Max.Theoric Sp.Weight (gr/cm3)	2,528
Creep (mm)	4,3	Rammer Number	50
VMA (Voids in the mineral aggregate)	17,05	Rummer Heat (^{0}C)	145
Schellenberger's Bittumen Filtering Experiment % : 0,15			

6.2.5. UTAC- Ultra Thin Asphalt (UTA) Mix Design

Ultra thin asphalt (UTA) surfacings are typically placed with a minimum thickness of around 15-20 mm, or about half the thickness of similar sized conventional asphalt wearing course mixes. The main characteristics of UTA surfacings are the use of a heavy tack coat or sprayed seal to form an integral bond with the underlying surface, and the adoption of coarse gap-graded mixes to provide good surface texture.

The achievement of a strong bond distinguishes UTA from conventional asphalt surfacing that achieves a level of independent integrity in each layer. The tack coat, or seal, used to bond the surfacing in place also assists in waterproofing the pavement where permeable surfacing materials are used. In this study, UTA mix prepared according to gathered data which was derived from experimental studies. Owing to lack of some performance test devices, Marshall Design for UTA could not done in laboratory,

Table 6.7. Typical Combined Grading for UTA

AS Sieve Size (mm)	Percentage (by mass)	Passing
	Range	Typical Target
13.2	100	100
9.5	80–100	90
6.7	30–55	40
4.75	20–40	30
2.36	18–36	27
1.18	14–30	22
0.60	10–25	18
0.30	7 20	13
0.15	6–12	9
0.075	4–8	6
Binder content	4.7–5.4	5.0

6.2.6. Geosynthetics

Four different types and branded geosynthetic materials were used as reinforcement to provide a resistance against rutting. Table 6.8 includes comparison of geosynthetics according to their technical specifications. The geosynthetic based reinforcement in flexible pavement structures must resist as much as much damage as possible from the stresses and strains applied during installation and overlaying/compaction of the asphalt.

Geosynthetics cut according to slab dimensions and install in asphalt slab interfaces by using tack coat for sticking. Figure 6.6 and Figure 6.7 show installation stages of geosynthetic material.

Table 6.8. Comparison of technical specifications of geosynthetic materials

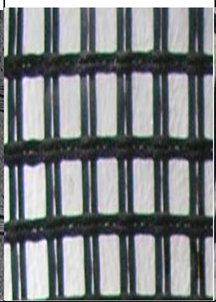
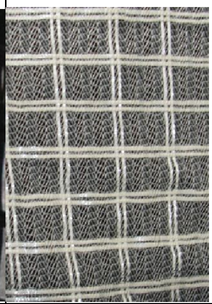
Brands	HATELİT	Aspha Glass	Syntheen GlassBitutex Composite	Tensar Glasstex
Pictures				
Product:	Nonwoven-Geogrid	Woven Geogrid	Nonwoven-Geogrid	Nonwoven-Geogrid
Raw Material	Geogrid: PET	Fiber Glass	Fiber Glass	Geogrid: PET
Coating	bituminous	bituminous	bituminous	bituminous
Weight	~ 270 g/m²	~ 650 g/m²	~ 400 g/m²	~ 430 g/m²
Ultimate tensile strength				
longitudinal	>50 kN/m	100 kN/m	115 kN/m	100 kN/m
transversal	> 50 kN/m	100 kN/m	115 kN/m	100 kN/m
Tensile strength at 3% strain				
longitudinal	>12 kN/m		107 kN/m	35kN/m
transversal	>12 kN/m		96 kN/m	35kN/m
Strain at nominal tensile strength				
Longitudinal	12%	3%	3%	3%
Transversal	12%	3%	3%	3%
Mesh size of geogrid	40 x 40 mm	10 x 10 mm	20x20 mm	40x40 mm
Heat resistance	up to 190 °C	up to 320 °C	up to 850 °C	
Standard dimensions				
Width	5.00 m	2.00 m	2.20 m	1.50 m
Lengths	150.00 m	100.00 m	100.00 m	100.00 m
Intended use	Asphalt Reinforcement	Asphalt Reinforcement	Asphalt Reinforcement	Asphalt Reinforcement

Figure 6.6. Preparation of geosynthetics for test: Cutting of material

Figure 6.7. Preparation of geosynthetic for test: Installing of geosynthetics

6.3. Specimen Preparation

Aggregate and modified or normal bitumen were mixed in big mixer according to EN 12697-35 standard. Figure 6.8 shows mixing stage of asphalt in mixer.

The flexible base parts of specimens were fabricated in Segmental Compactor according to EN-12697-33 standard. Figure 6.9 shows compaction of slabs in segmental compactor. Then geosynthetics were cut suitable for dimension of specimens and then placed onto whole surface of below element by using tack-coat as a sticker. Upper part of asphalt concrete specimen was applied after two day curing in same method.

AC mixes were designed according to Marshall Design Concept and the reference standard is TS-EN-12697 (TS-EN is a standard code for the adaptation of European Norms to Turkish Standards). Aggregate gradation design by sieve analysis and TS 3530-EN 933-1 were regarded as reference standard.

Figure 6.8. Mixing stage of aggregate and bitumen

Figure 6.9. Compaction of slabs in segmental compactor.

Figure 6.10. Rutting Test of Two Same Plastered Specimens in HWTD

Fifteen specimens were prepared for wheel tracking test program. These specimens were simply classified based on asphalt concrete (AC) mix design type, type of used geosynthetics.

6.3.1. Dense Graded Asphalt Concrete (DGAC) specimens

This type of specimen was fabricated with flexible pavement elements. Dense graded binder and wearing course was overlapped. One of the Dense Graded Asphalt Concrete (DGAC) specimens was prepared without geosynthetic. It called as DGAC-C and was a reference specimen for other geosynthetic incorporated DGAC specimens.

The procedure for preparation of asphalt mix is defined in “the materials and preparation procedures” 60 mm dense graded binder course and 40-mm dense graded wearing course were overlapped. Thicknesses are same for geosynthetic installed specimens. A little tack coat was sprayed between layers to increase adhesion. DGAC-1 specimen has included Aspha Glassgrid branded geosynthetic as reinforcement between binder and wearing course. DGAC-2 specimen has included Hatelit C 40 17 branded geosynthetic as reinforcement between binder and wearing course. DGAC-3 specimen has included Synteen Glass Bitutex Composite branded geosynthetic as reinforcement between binder and wearing course. DGAC-4 specimen has included Tensar Glasstex branded geosynthetic as reinforcement between binder and wearing course (Table 6.9).

The aim is to check rutting potential of Dense Graded Asphalt Concrete specimens since the different type of geosynthetics are used as reinforcement.

6.3.2. Gap Graded Asphalt Concrete (GGAC) specimens

This type of specimen was fabricated with flexible pavement elements. Dense graded binder and gap graded Stone Mastic Asphalt (SMA) wearing course will be overlapped. The wearing course is called as Stone Mastic Asphalt for Gap graded friction courses. One of the Gap Graded Asphalt Concrete (GGAC) specimens was prepared without geosynthetic. It called as GGAC-C and was a reference specimen for other geosynthetic incorporated GGAC specimens.

Table 6.9. Definition of Dense Graded Asphalt Concrete (DGAC) specimens

1	DGAC-C	Wearing Course Binder Course	Control test for Dense Graded Asphalt Concrete (DGAC) type specimens. DGAC-C specimen has no any geosynthetic
2	DGAC-1	Wearing Course ***Aspha Glassgrid*** Binder Course	Test for Dense Graded Asphalt Concrete (DCAG) type specimens. DGAC-1 specimen has included ***Aspha Glassgrid*** geosynthetic as reinforcement between binder and wearing course
3	DGAC-2	Wearing Course ***Hatelit C 40 17*** Binder Course	Test for Dense Graded Asphalt Concrete (DCAG) type specimens. DGAC-2 specimen has included Hatelit C 40 17 geosynthetic as reinforcement between binder and wearing course
4	DGAC-3	Wearing Course ***Synteen G.B.C.*** Binder Course	Test for Dense Graded Asphalt Concrete (DCAG) type specimens. DGAC-3 specimen has included Synteen Glass Bitutex Composite geosynthetic as reinforcement between binder and wearing course
5	DGAC-4	Wearing Course ***Tensar Glasstex*** Binder Course	Test for Dense Graded Asphalt Concrete (DCAG) type specimens. DGAC-4 specimen has included Tensar Glasstex geosynthetic as reinforcement between binder and wearing course

The procedure for preparation of asphalt mix is defined in “the materials and preparation procedures” 60 mm dense graded binder course and 40-mm gap graded wearing course (SMA) was overlapped. Thicknesses are same for geosynthetic installed specimens. A little tack coat was sprayed between layers to increase adhesion. DGAC-1 specimen has included Aspha Glassgrid branded geosynthetic as reinforcement between binder and SMA wearing course. DGAC-2 specimen has included Hatelit C 40 17 branded geosynthetic as reinforcement between binder and SMA wearing course. DGAC-3 specimen has included Synteen Glass Bitutex Composite branded geosynthetic as reinforcement between binder and SMA wearing course. DGAC-4 specimen has included

Tensar Glasstex branded geosynthetic as reinforcement between binder and wearing course (Table 6.10).

The aim is to check rutting potential of Gap Graded Asphalt Concrete (GGAC) specimens since the different type of geosynthetics are used as reinforcement.

Table 6.10. Definition of Gap Graded Asphalt Concrete SMA (GGAC) specimens

1	GGAC-C	SMA Wearing C. Binder Course	Control test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-C specimen has no any geosynthetic
2	GGAC-1	SMA Wearing C. ***Aspha Glassgrid*** Binder Course	Test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-1 specimen has included Aspha Glassgrid geosynthetic as reinforcement between binder and Stone Mastic Asphalt (SMA) wearing course
3	GGAC-2	SMA Wearing C. ***Hatelit C 40 17*** Binder Course	Test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-2 specimen has included Hatelit C 40 17 geosynthetic as reinforcement between binder and Stone Mastic Asphalt (SMA) wearing course
4	GGAC-3	SMA Wearing C. ***Synteen G.B.C.*** Binder Course	Test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-3 specimen has included Synteen Glass Bitutex Composite geosynthetic as reinforcement between binder and Stone Mastic Asphalt (SMA) wearing courses
5	GGAC-4	SMA Wearing C. ***Tensar Glasstex*** Binder Course	Test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-4 specimen has included Tensar Glasstex geosynthetic as reinforcement between binder and Stone Mastic Asphalt (SMA) wearing course

6.3.3. Ultra Thin Asphalt Concrete (UTAC) specimens

This type of specimen was fabricated with flexible pavement elements. Dense graded binder and UTA, 2 cm, wearing course was overlapped. The wearing course is called as Ultra Thin Asphalt for dense graded thin wearing courses.

Thin courses generally use for rehabilitation of rigid and flexible pavements in order to extend their life cycle. UTA is cheaper than conventional asphalt concretes due to the thickness. However, it might not provide sufficient resistance to distresses caused by heavy traffic loads. So that, reinforcing of UTA overlay structure with geosynthetic may be advantageous for load support requirement in pavements.

One of the Ultra Thin Asphalt Concrete (UTAC) specimens was prepared without geosynthetic. It called as UTAC-C and will be a reference specimen for other geosynthetic incorporated UTAC specimens.

The procedure for preparation of asphalt mix is defined in “the materials and preparation procedures” 60 mm dense graded binder course and 20-mm ultra thin wearing course (UTA) was overlapped. Thicknesses are same for geosynthetic installed specimens. A little tack coat was sprayed between layers to increase adhesion. UTAC-1 specimen has included Aspha Glassgrid branded geosynthetic as reinforcement between binder and UTA wearing course. UTAC-2 specimen has included Hatelit C 40 17 branded geosynthetic as reinforcement between binder and UTA wearing course. UTAC-3 specimen has included Synteen Glass Bitutex Composite branded geosynthetic as reinforcement between binder and UTA wearing course. UTAC-4 specimen has included Tensar Glasstex branded geosynthetic as reinforcement between binder and wearing course (Table 6.11).

The aim is to check rutting potential of Ultra Thin Concrete (UTAC) specimens since the different type of geosynthetics are used as reinforcement.

Table 6.11. Definition of Ultra Thin Asphalt Concrete (UTAC) specimens

1	UTAC-C	UTA Wearing C. Binder Course	Control test for Ultra Thin Asphalt Concrete (OGAC) type specimens. UTAC-C specimen has no any geosynthetic
2	UTAC- 1	UTA Wearing C. ***Aspha Glassgrid*** Binder Course	Test for Ultra Thin Asphalt Concrete (UTAC) type specimens. UTAC-1 specimen has included Aspha Glassgrid geosynthetic as reinforcement between binder and 2 cm thickness wearing course
3	UTAC-2	UTA Wearing C. ***Hatelit C 40 17*** Binder Course	Test for Ultra Thin Asphalt Concrete (UTAC) type specimens. UTAC-2 specimen has included Hatelit C 40 17 geosynthetic as reinforcement between binder and 2 cm thickness wearing course.
4	UTAC-3	UTA Wearing C. ***Synteen G.B.C.*** Binder Course	Test for Ultra Thin Asphalt Concrete (UTAC) type specimens. UTAC-3 specimen has included Synteen Glass Bitutex Composite geosynthetic as reinforcement between binder and 2 cm thickness wearing course
5	UTAC-4	UTA Wearing C. ***Tensar Glasstex*** Binder Course	Test for Ultra Thin Asphalt Concrete (UTAC) type specimens. UTAC-4 specimen has included Tensar Glasstex geosynthetic as reinforcement between binder and 2 cm thickness wearing course

6.4. Test Devices

6.4.1. Hamburg Wheel Tracking Device (HWTD)

The Hamburg Wheel Tracking Device (HWTD) apparatus consists of a steel casing with hinged cover. Two measuring places with rolling wheel units working in opposite directions are located inside the inner stainless steel casing. The wheels are driven by a slider crank with frequency controlled motor with a constant levering load system. The wheel units are guided by anti-dumping elements. Electric heating elements with PID-controller are integrated for water condition tests. A warm air blowing system is installed for tests in air condition. The wheels can be uncoupled and placed to a parking position left and right, this ensures that samples can be easily placed into the machine. The number of crossings, the track groove depth of both wheels and the temperature inside are recorded by a Windows software program with online display.

Sample dimensions are 260x320 mm and as you design sample as circular, diameter is 300 mm. Sample height can be change from 40 mm up to 120 mm. Rolling wheel is coated with 20 mm rubber coating. Rolling wheel width is 50 mm and rolling section is about 230 mm. Measuring section of device is between 65mm and 165 mm. Default applied load is 710 N. Temperature range using water can be change up to 70°C.

Figure 6.11. A Hamburg Wheel Tracking Device

6.4.2. Segmental Compactor

Segmental Compactor will be used for the preparation of 320 x 260 mm, 40 to 120 mm high rolled asphalt samples using a roller segment. The mould as well as the roller segment is equipped with an electric heating unit. Electronically operated motors for vertical and horizontal movement with integrated load and displacement transducer. The machine is software controlled with the possibility to create test sequences by the user.

Technical specifications for Segmental Compactor are listed below:

- Rolling force: 0 to 30 kN
- Rolled segment radius: 550 mm
- Rolling speed: 45 roll./min
- Sample dimensions: 320x260 mm
- Sample height: 40...120 mm
- Dimensions: 1320 x 840 x 2220 mm

6.4.3. Laboratory Mixer

Laboratory Mixer will be used for preparation of bituminous material mixture samples according to the synchronization principle. The cover with sealing is equipped with a spindle motor to open and close. A window permits to watch the mixing process inside. In addition, the cover is provided with connection plugs for gas, etc. The constant speed motorized stainless steel mixing drum is equipped with switch to turn left or right. An electric heating installation at the bottom and walls with PID regulator provide a constant temperate. For unloading, the mixing bowl can be tilted to the front by the installed motor system. The special mixing tool with variable speed range is optimized for mixing asphalt samples. Technical specifications for Laboratory Mixer are listed below:

- Drum contents 30 lt, Maximum drum load is 80 kg
- Mixing tool speed is between 25 - 60 1/min
- Mixing drum temperature is between 25 - 250° C
- Dimensions are 1015 x 1115 x 1490 m and Weight approximately 465 kg

6.5. Test Results

Each geosynthetic incorporated specimen was tested in HWTD approximately for ten hours. Each pass of wheels are measured by very sensitive LVDT apparatus. Results stocked in connected computer and plots for each test are presented in figures below.

6.5.1. DGAC-C Specimen

Control test for Dense Graded Asphalt Concrete (DGAC) type specimens. DGAC-C specimen has no any geosynthetic.

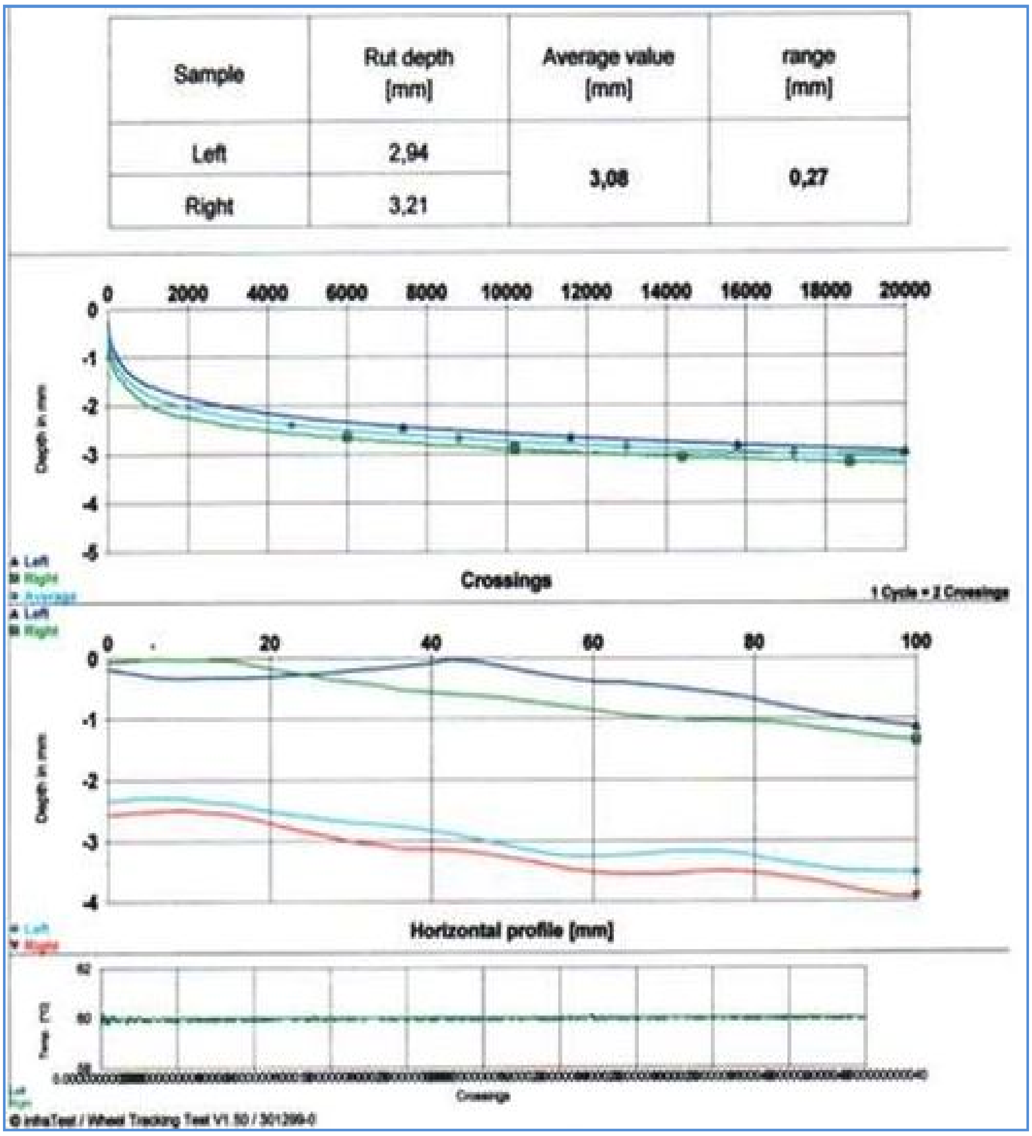


Figure 6.12. DGAC- C Specimen Rutting Test Result Plot

6.5.2. DGAC-1 Specimen

DGAC-1 - Test for Dense Graded Asphalt Concrete (DCAG) type specimens. DGAC-1 specimen has included ***Aspha Glassgrid*** geosynthetic as reinforcement between binder and wearing course

Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	2,54	2,86	0,64
Right	3,18		

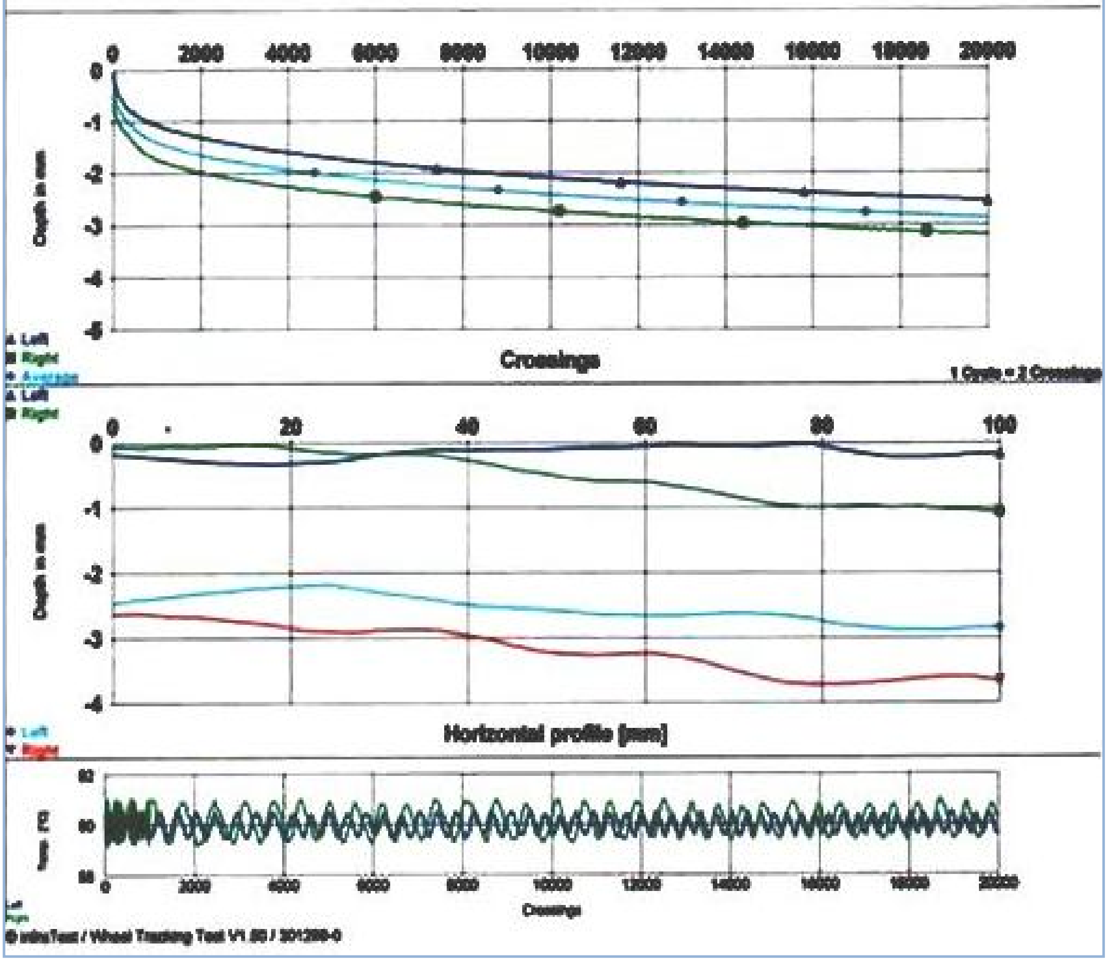


Figure 6.13. DGAC- 1 Specimen Rutting Test Result Plot

6.5.3. DGAC-2 Specimen

DGAC-2 - Test for Dense Graded Asphalt Concrete (DCAG) type specimens. DGAC-2 specimen has included ***Hatelit C 40 17*** geosynthetic as reinforcement between binder and wearing course.

Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	2,35	2,69	0,67
Right	3,02		

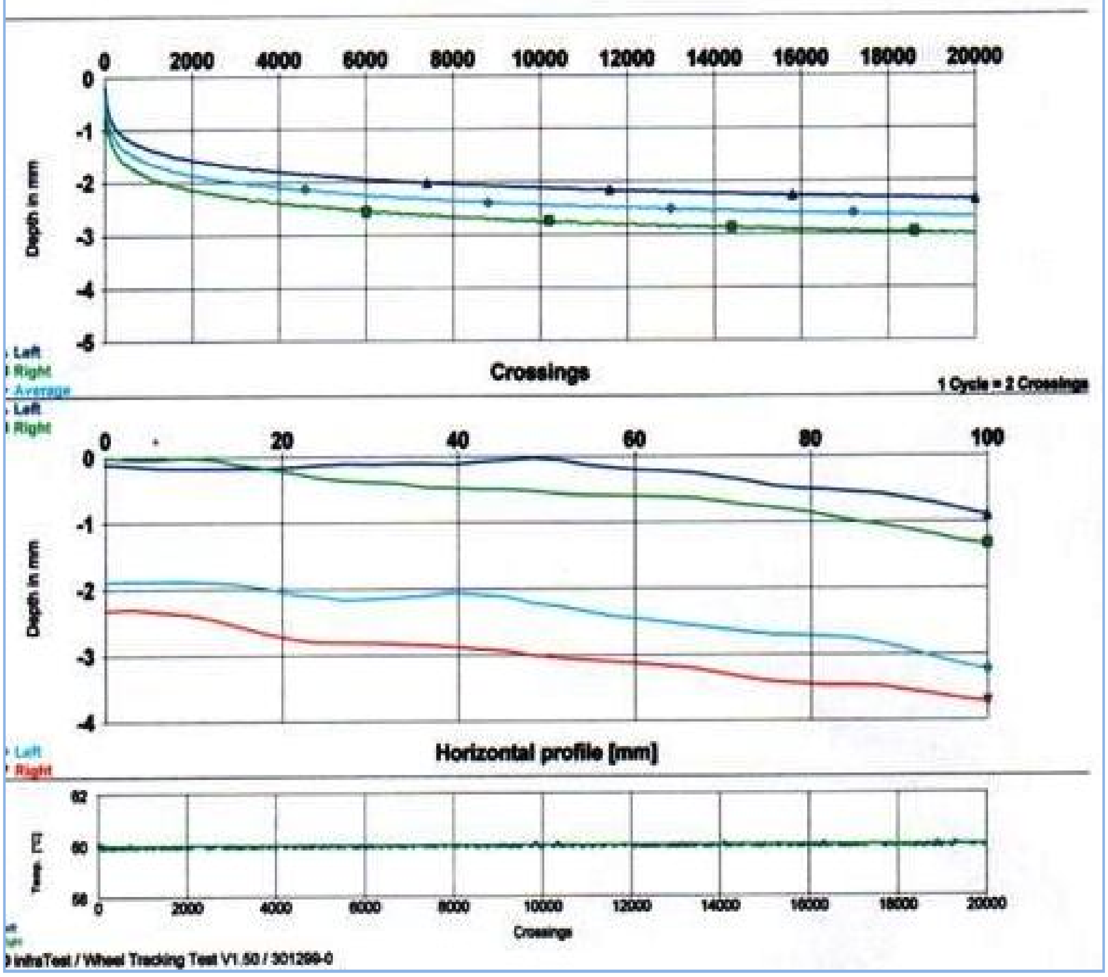


Figure 6.14. DGAC- 2 Specimen Rutting Test Result Plot

6.5.4. DGAC-3 Specimen

DGAC-3 - Test for Dense Graded Asphalt Concrete (DCAG) type specimens. DGAC-3 specimen has included ***Synteen Glass Bitutex Composite*** geosynthetic as reinforcement between binder and wearing course.

Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	3,10	3,06	0,09
Right	3,01		

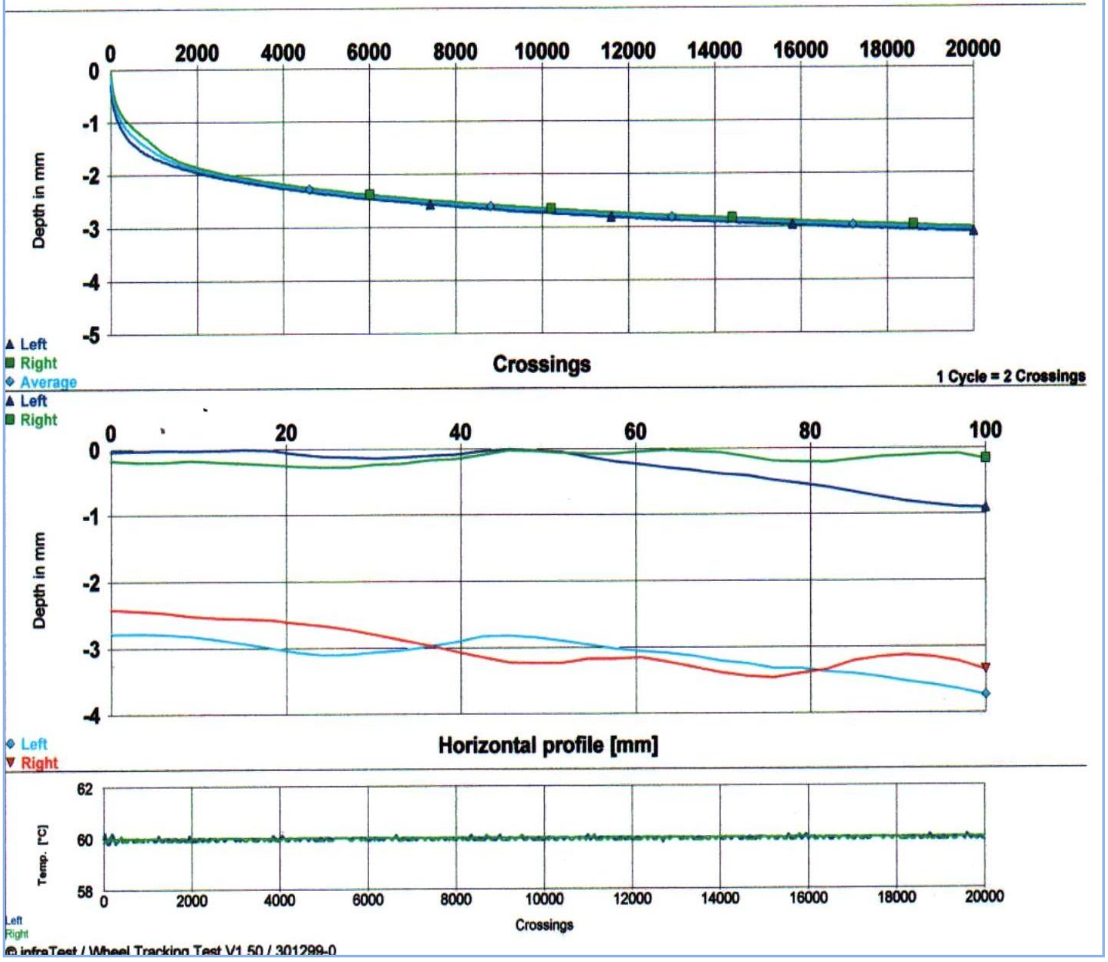


Figure 6.15. DGAC- 3 specimen rutting test result plot

6.5.5. DGAC-4 Specimen

DGAC-4 - Test for Dense Graded Asphalt Concrete (DCAG) type specimens. DGAC-4 specimen has included ***Tensar Glasstex*** geosynthetic as reinforcement between binder and wearing course.

Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	3,68	3,62	0,13
Right	3,55		

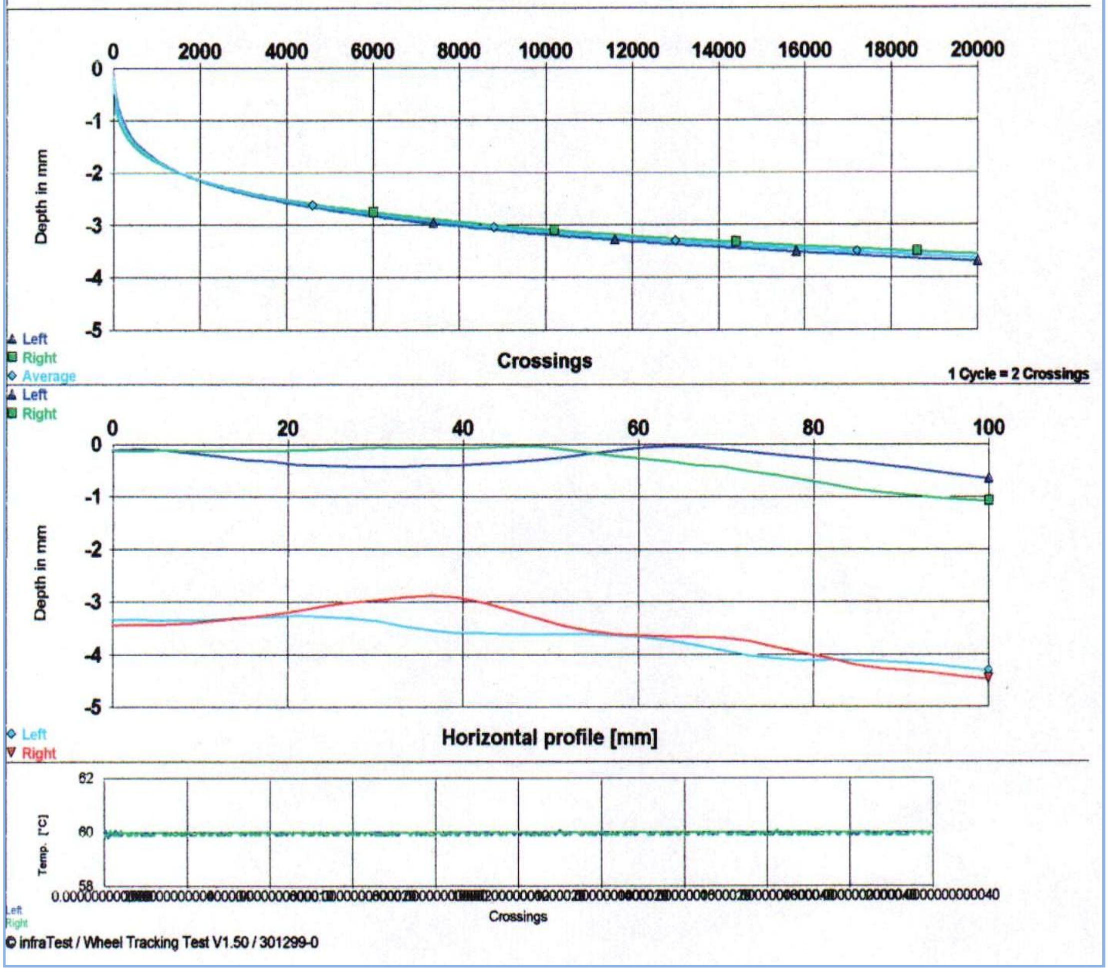


Figure 6.16. DGAC- 4 Specimen Rutting Test Result Plot

6.5.6. GGAC-C Specimen

GGAC-C - Control test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-C specimen has no any geosynthetic.

Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	2,67	2,98	0,62
Right	3,29		

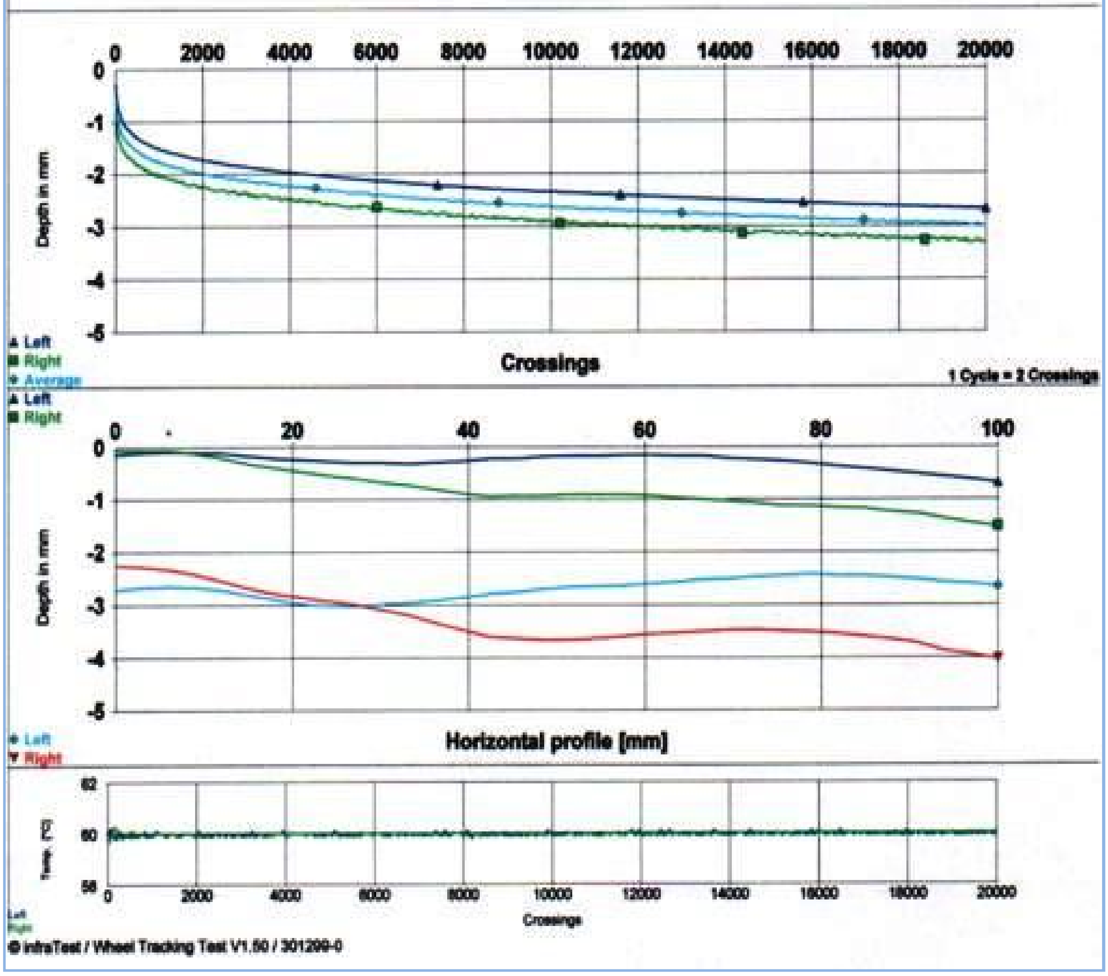


Figure 6.17. GGAC- C Specimen Rutting Test Result Plot

6.5.7. GGAC-1 Specimen

GGAC-1 - Test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-1 specimen has included ***Aspha Glassgrid*** geosynthetic as reinforcement between binder and Stone Mastic Asphalt (SMA) wearing course

Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	2,24	2,37	0,25
Right	2,49		

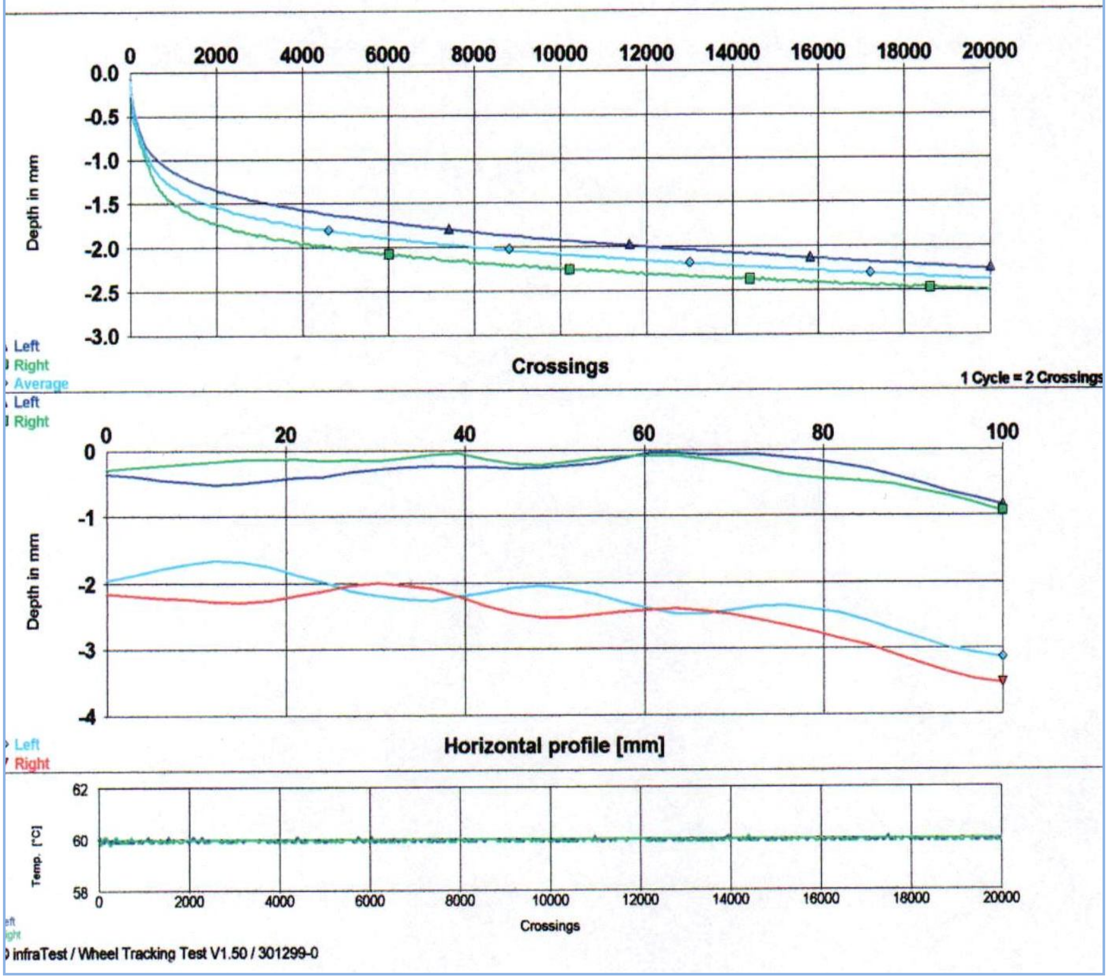
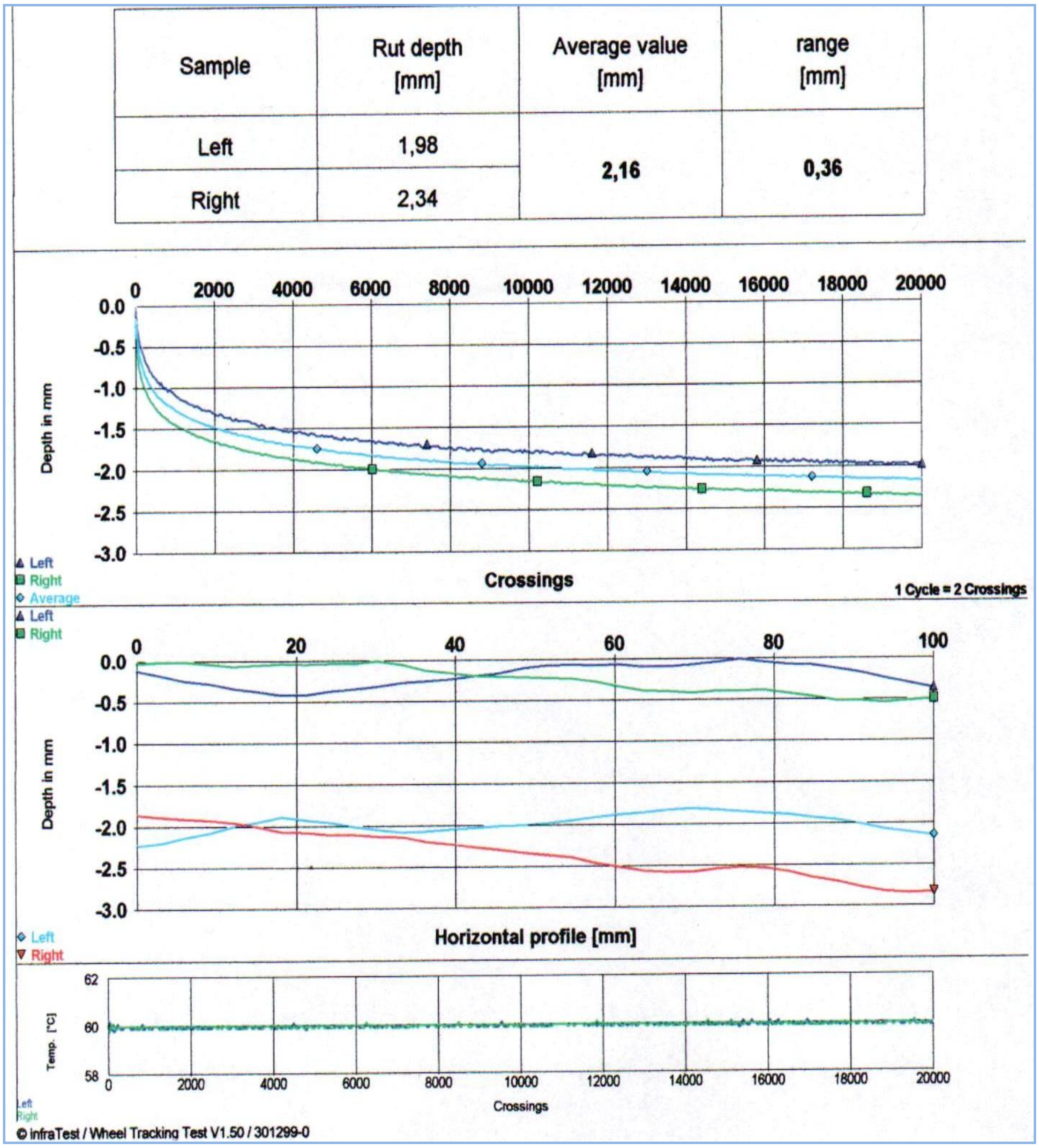


Figure 6.18. GGAC- 1 Specimen Rutting Test Result Plot

6.5.8. GGAC-2 Specimen

GGAC-2 - Test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-2 specimen has included ***Hatelit C 40 17*** geosynthetic as reinforcement between binder and Stone Mastic Asphalt (SMA) wearing course



Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	1,98	2,16	0,36
Right	2,34		

Figure 6.19. GGAC- 2 Specimen Rutting Test Result Plot

6.5.9. GGAC-3 Specimen

GGAC-3 - Test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-3 specimen has included ***Synteen Glass Bitutex Composite*** geosynthetic as reinforcement between binder and Stone Mastic Asphalt (SMA) wearing course.

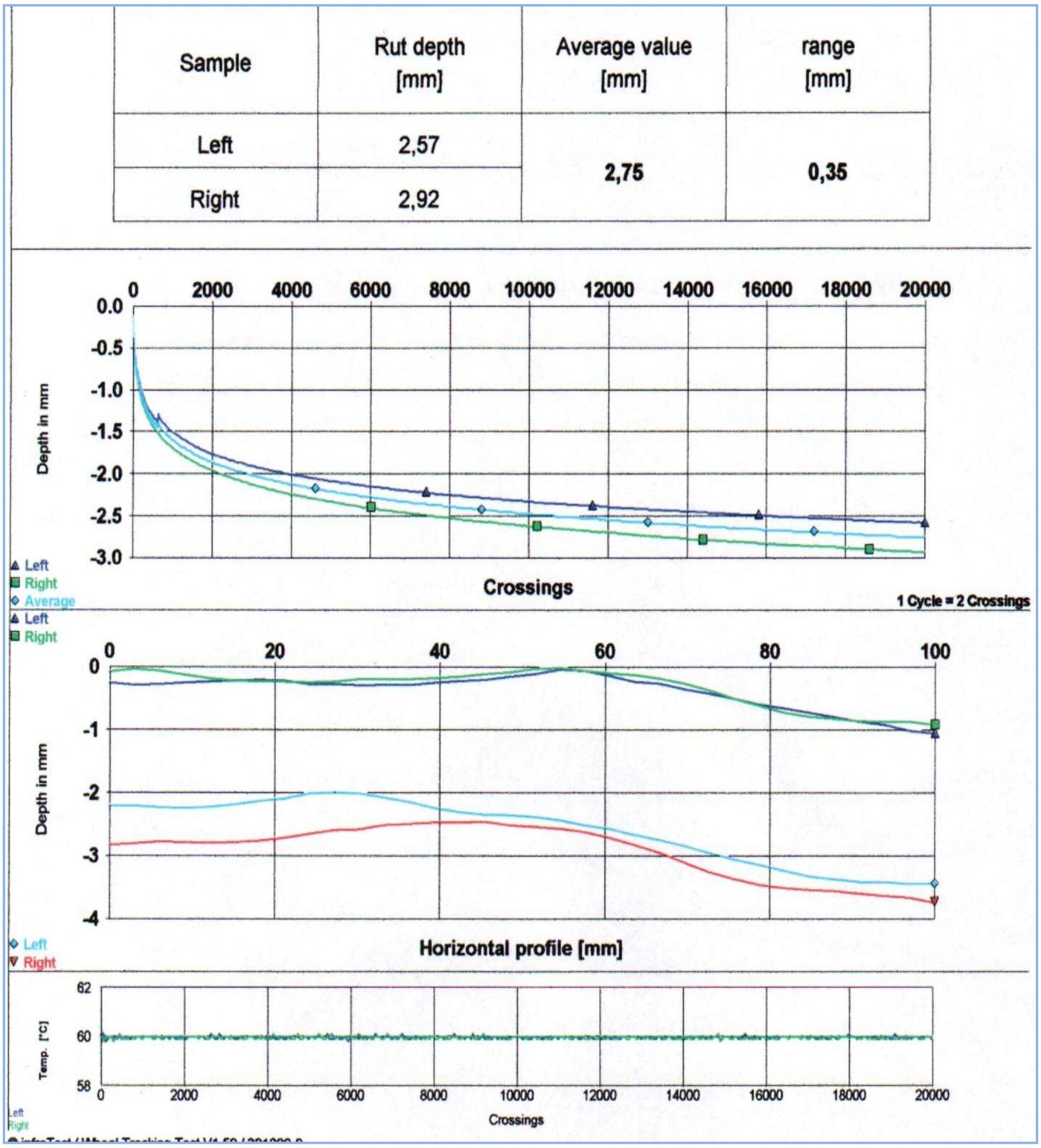


Figure 6.20. GGAC- 3 Specimen- Rutting Test Result Plot

6.5.10. GGAC-4 Specimen

GGAC-4 - Test for Gap Graded Asphalt Concrete (GGAC) type specimens. GGAC-4 specimen has included ***Tensar Glasstex*** geosynthetic as reinforcement between binder and Stone Mastic Asphalt (SMA) wearing course

Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	3,35	**3,51**	**0,32**
Right	3,67		

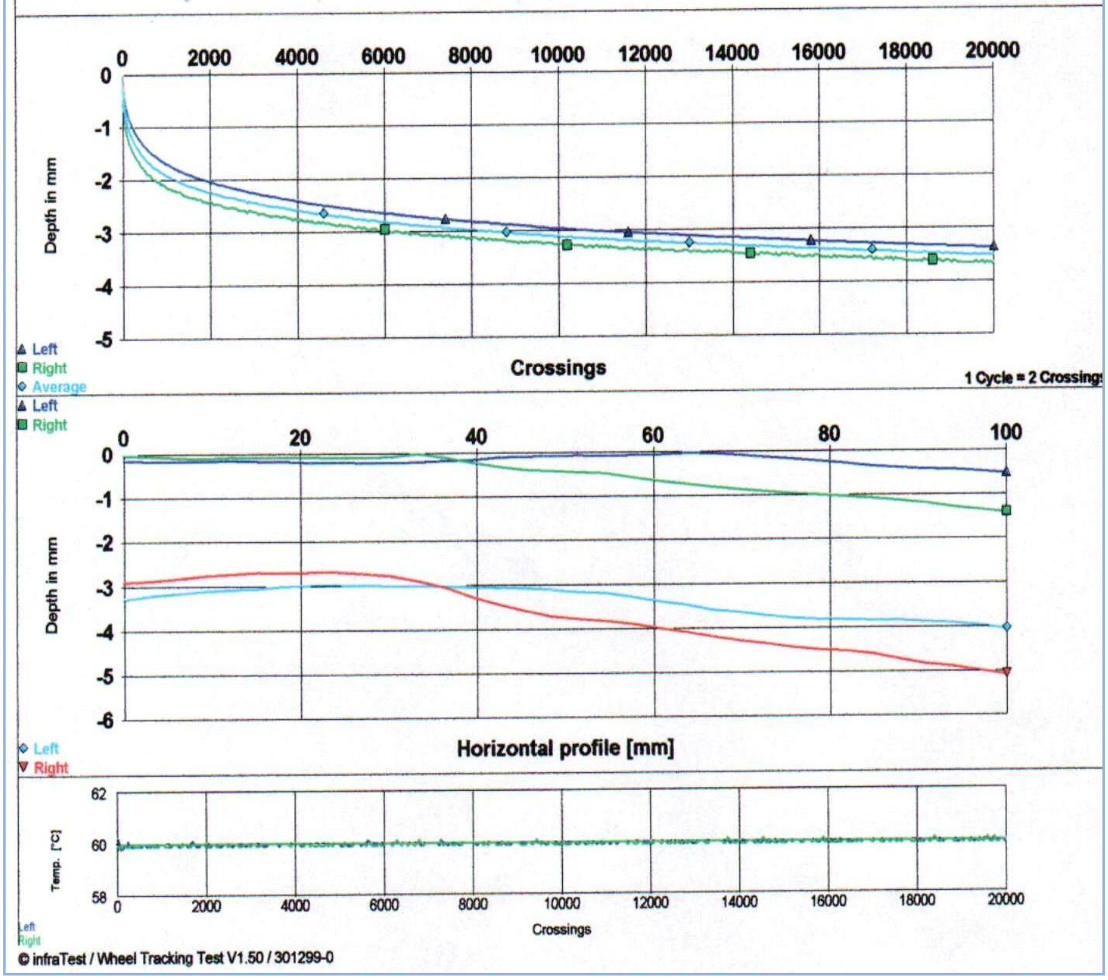
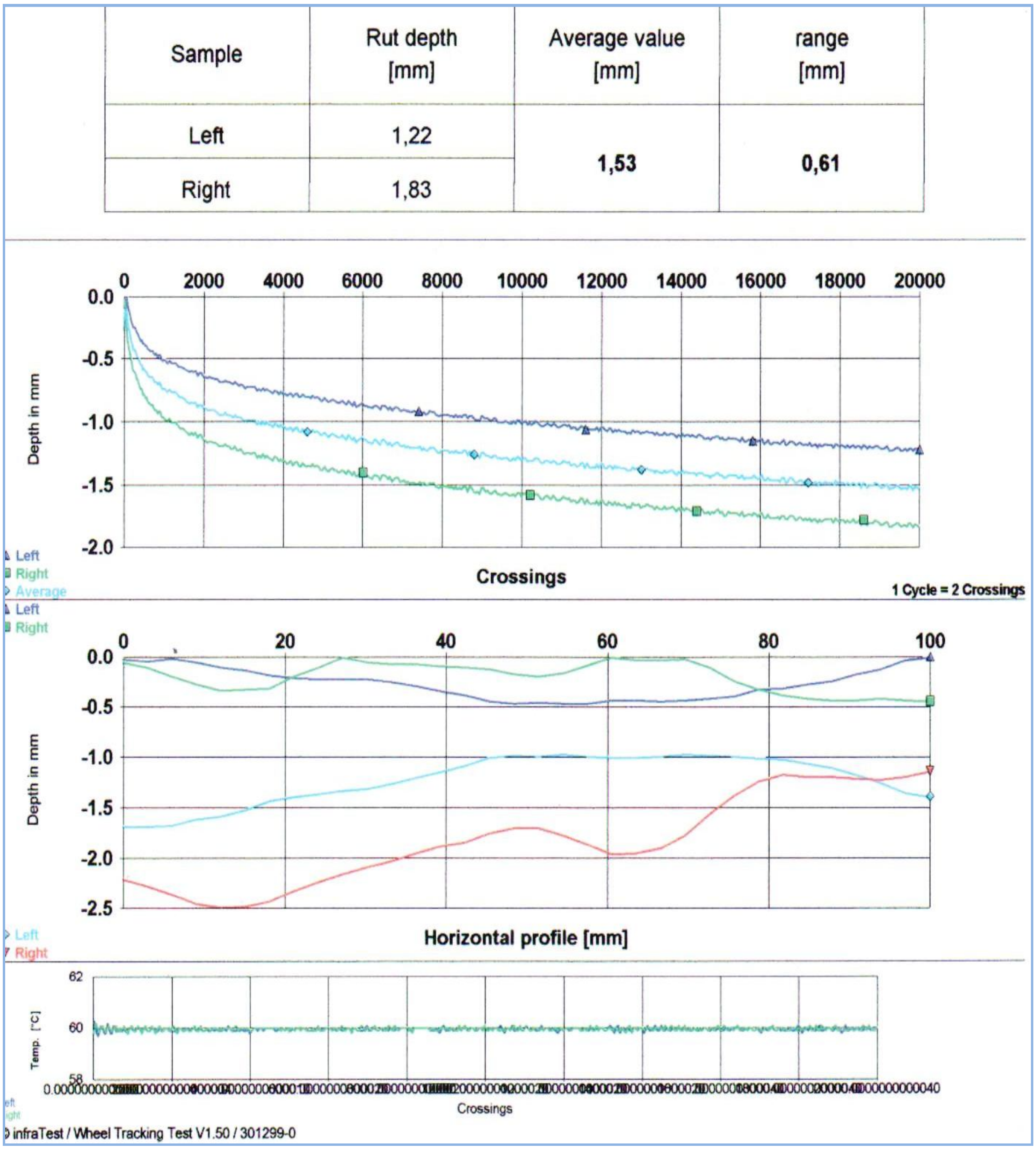


Figure 6.21. GGAC- 4 Specimen- Rutting Test Result Plot

6.5.11. UTAC-C Specimen

UTAC-C - Control test for Ultra Thin Asphalt Concrete (UTAC) type specimens. UTAC-C specimen has no any geosynthetic.



Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	1,22	**1,53**	**0,61**
Right	1,83		

Figure 6.22. UTAC- C Specimen- Rutting Test Result Plot

6.5.12. UTAC-1 Specimen

UTAC-1 - Test for Ultra Thin Asphalt Concrete (UTAC) type specimens. UTAC-1 specimen has included ***Aspha Glassgrid*** geosynthetic as reinforcement between binder and 2 cm thickness wearing course.

Sample	Wheel-tracking rate [µm/cycle]	Average Wheel-tracking rate [µm/cycle]	Average Rut depth [mm]
Left	0,2	0,2	1,0
Right	0,2	0,1	1,1

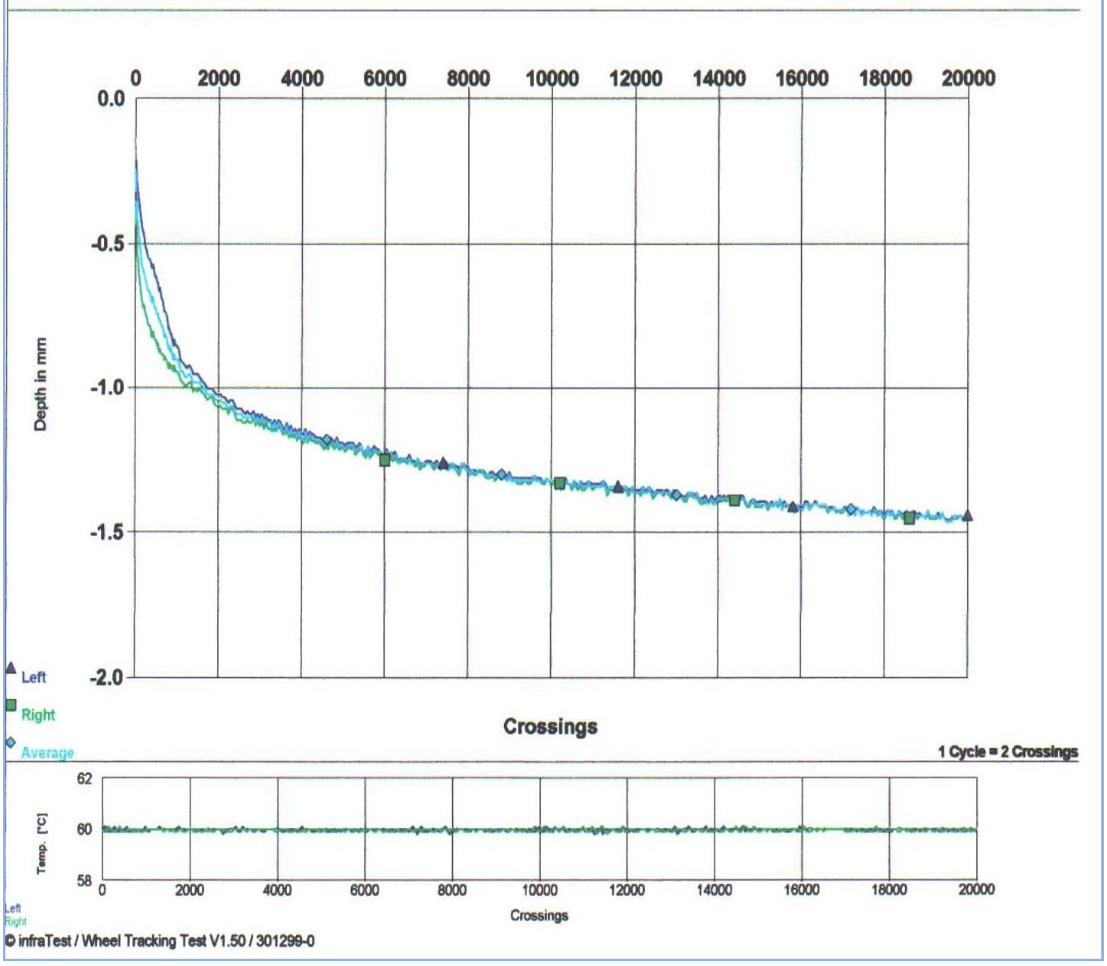
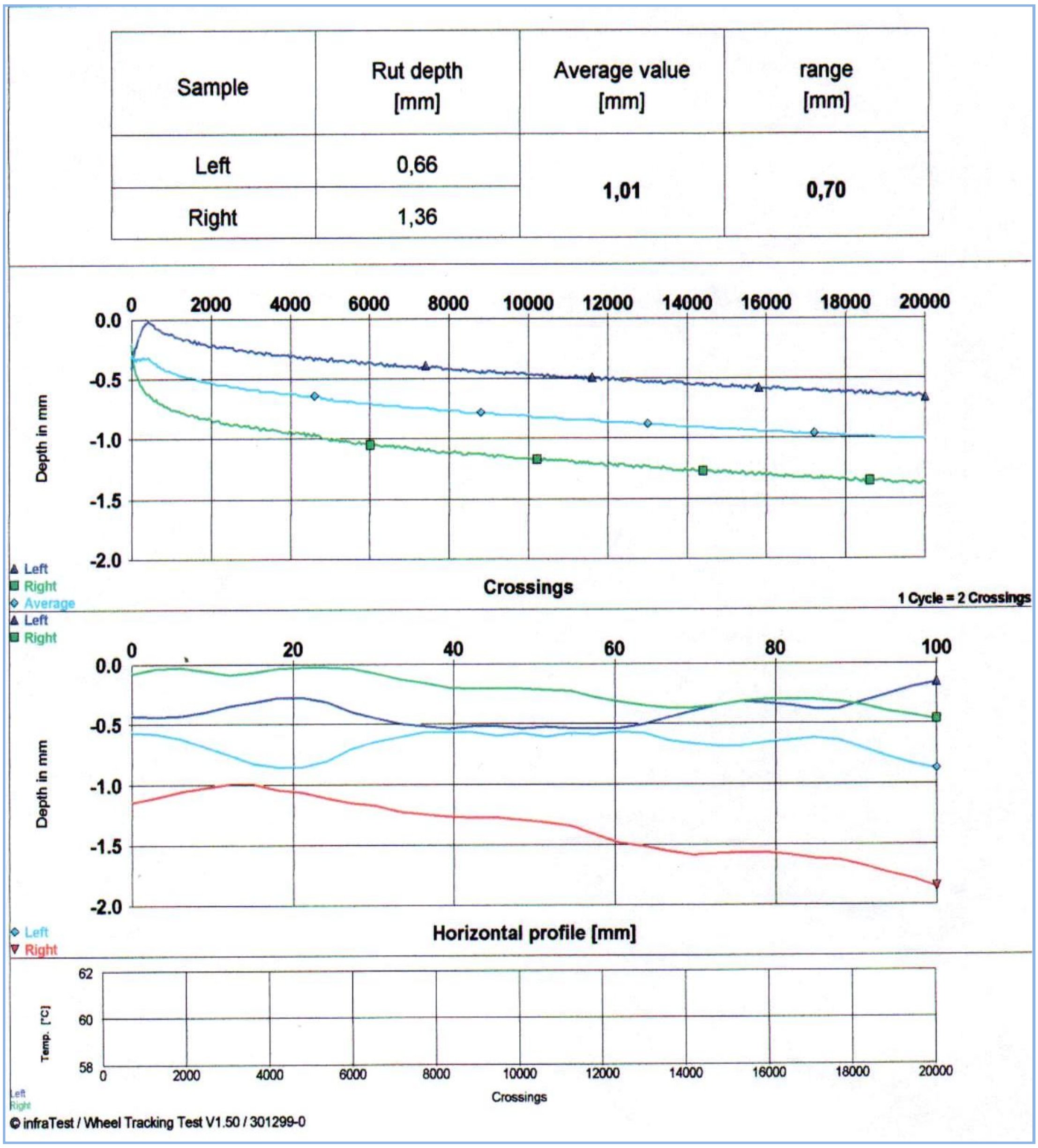


Figure 6.23. UTAC- 1 Specimen- Rutting Test Result Plot

6.5.13. UTAC-2 Specimen

UTAC-2 - Test for Ultra Thin Asphalt Concrete (UTAC) type specimens. UTAC-2 specimen has included ***Hatelit C 40 17*** geosynthetic as reinforcement between binder and 2 cm thickness wearing course.



Sample	Rut depth [mm]	Average value [mm]	range [mm]
Left	0,66	1,01	0,70
Right	1,36		

Figure 6.24. UTAC- 2 Specimen- Rutting Test Result Plot

6.5.14. UTAC-3 Specimen

UTAC-3 - Test for Ultra Thin Asphalt Concrete (UTAC) type specimens. UTAC-3 specimen has included ***Synteen Glass Bitutex Composite*** geosynthetic as reinforcement between binder and 2 cm thickness wearing course.

Sample	Wheel-tracking slope [mm/1000 cycles]	Wheel-tracking slope (linear part) [mm/1000 cycles]	Rut depth [mm]	prop. rut depth [%]
Left	0,02 (d10000-d5000)	0,02 (d9325-d7325)	1,5	3,6
Right	0,02 (d10000-d5000)	0,04 (d5725-d3725)	1,2	2,9
Average value	0,02	0,03	1,3	3,2

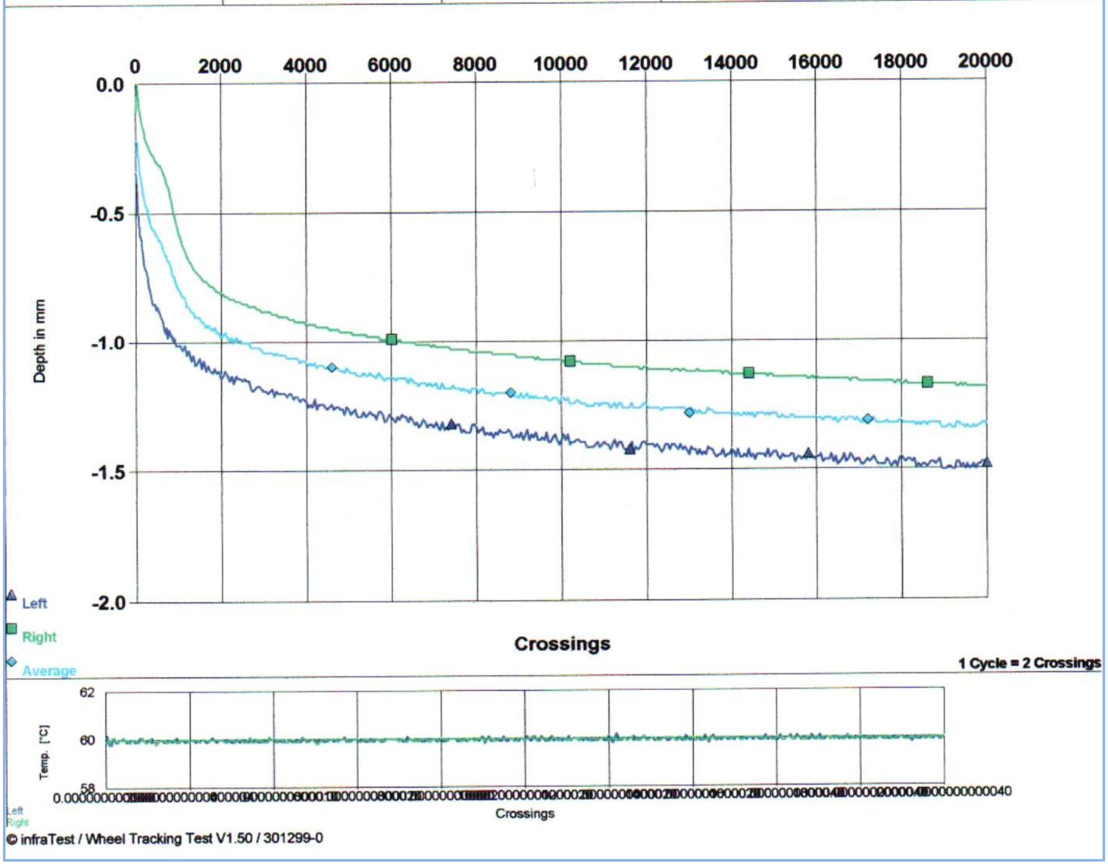


Figure.6.25. UTAC- 3 Specimen- Rutting Test Result Plot

6.5.15. UTAC-4 Specimen

UTAC-4 - Test for Ultra Thin Asphalt Concrete (UTAC) type specimens. UTAC-4 specimen has included ***Tensar Glasstex*** geosynthetic as reinforcement between binder and 2 cm thickness wearing course.

Sample	Wheel-tracking slope [mm/1000 cycles]	Wheel-tracking slope (linear part) [mm/1000 cycles]	Rut depth [mm]	prop. rut depth [%]
Left	0,02 (d10000-d5000)	0,02 (d9875-d7875)	2,1	5,2
Right	0,02 (d10000-d5000)	0,02 (d9875-d7875)	1,8	4,4
Average value	0,02	0,02	1,9	4,8

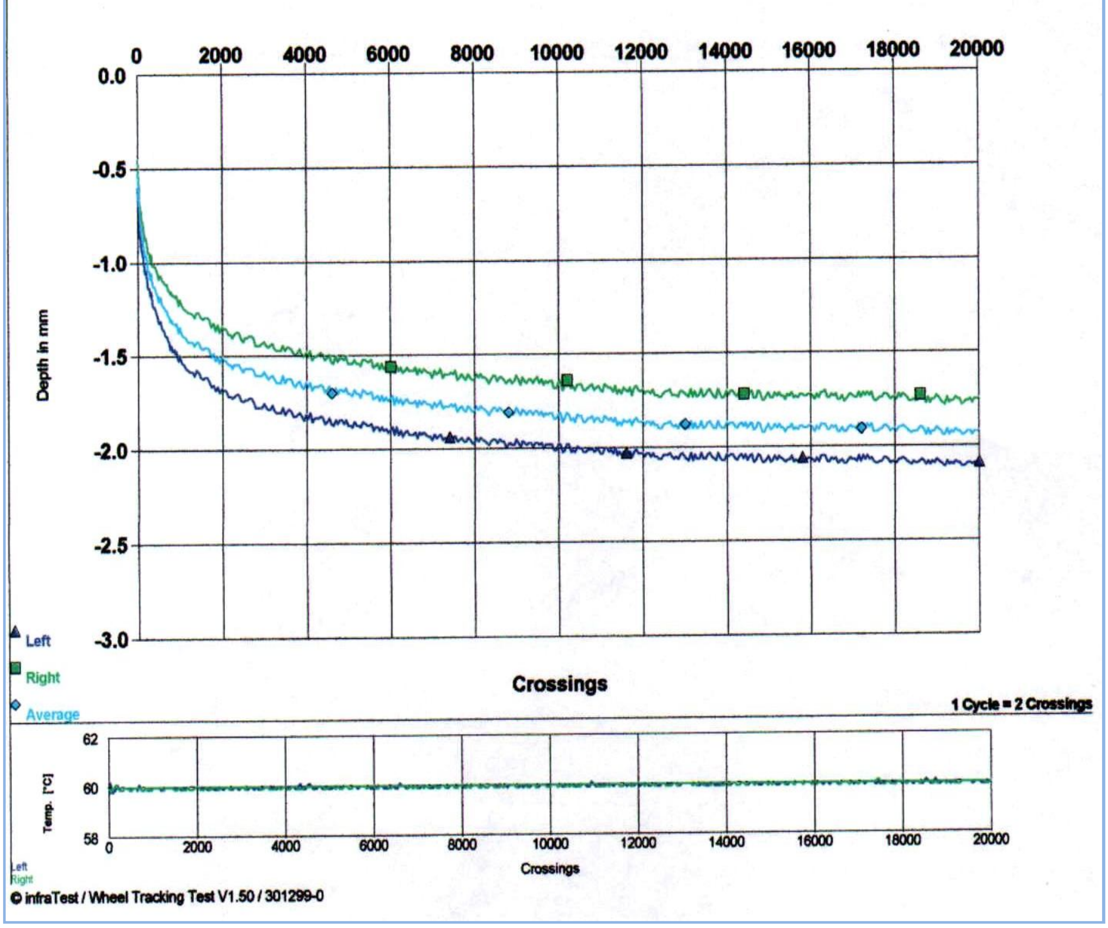
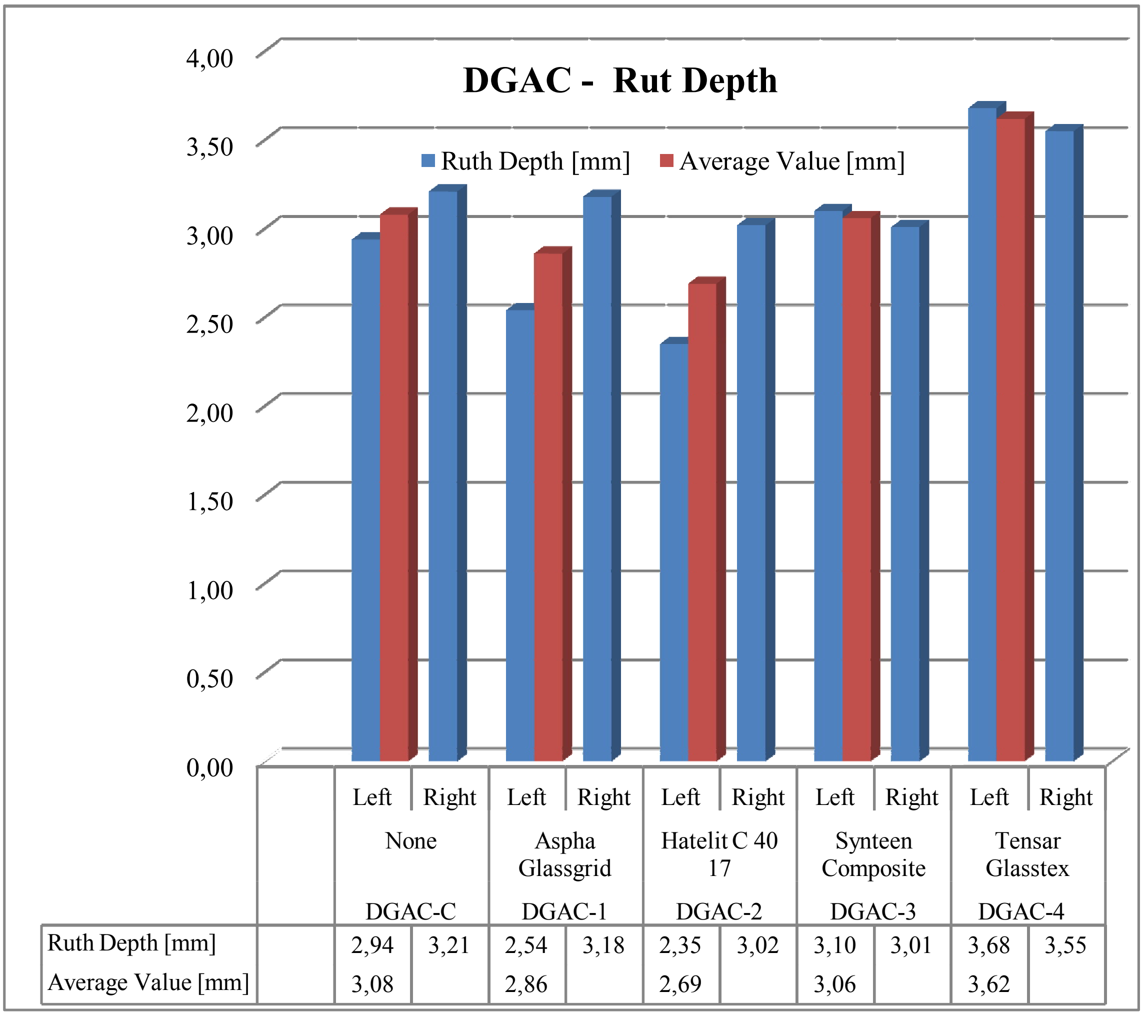


Figure 6.26. UTAC- 4 Specimen- Rutting Test Result Plot

Table 6.12. Rut Depth Values of Three Types of Specimens

Specimen Code	Geosynthetic	Sample	Ruth Depth [mm]	Average Value [mm]	Range [mm]
DGAC-C	***None***	Left	2,94	3,08	0,27
		Right	3,21		
DGAC-1	***Aspha Glassgrid***	Left	2,54	2,86	0,64
		Right	3,18		
DGAC-2	***Hatelit C 40 17***	Left	2,35	2,69	0,67
		Right	3,02		
DGAC-3	***Synteen Composite***	Left	3,10	3,06	0,09
		Right	3,01		
DGAC-4	***Tensar Glasstex***	Left	3,68	3,62	0,13
		Right	3,55		
GGAC-C	***None***	Left	2,67	2,98	0,62
		Right	3,29		
GGAC-1	***Aspha Glassgrid***	Left	2,24	2,37	0,25
		Right	2,49		
GGAC-2	***Hatelit C 40 17***	Left	1,98	2,16	0,36
		Right	2,34		
GGAC-3	***Synteen Composite***	Left	2,57	2,75	0,35
		Right	2,92		
GGAC-4	***Tensar Glasstex***	Left	3,35	3,51	0,32
		Right	3,67		
UTAC-C	***None***	Left	1,22	1,53	0,61
		Right	1,83		
UTAC-1	***Aspha Glassgrid***	Left	1,00	1,05	0,10
		Right	1,10		
UTAC-2	***Hatelit C 40 17***	Left	0,66	1,01	0,70
		Right	1,36		
UTAC-3	***Synteen Composite***	Left	1,50	1,30	0,20
		Right	1,20		
UTAC-4	***Tensar Glasstex***	Left	2,10	1,90	0,30
		Right	1,80		

7. CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK



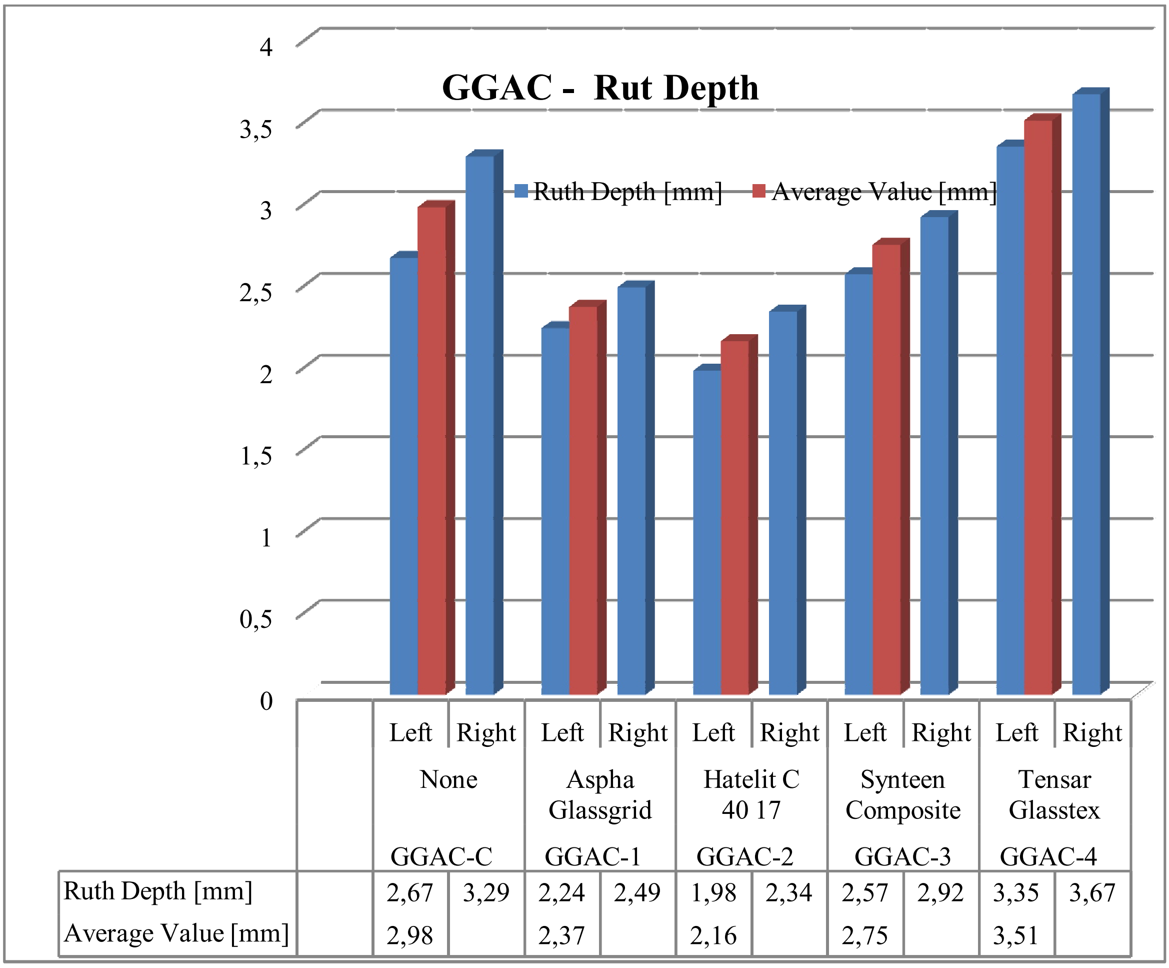
	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
	None		Aspha Glassgrid		Hatelit C 40 17		Synteen Composite		Tensar Glasstex	
	DGAC-C		DGAC-1		DGAC-2		DGAC-3		DGAC-4	
Ruth Depth [mm]	2,94	3,21	2,54	3,18	2,35	3,02	3,10	3,01	3,68	3,55
Average Value [mm]	3,08		2,86		2,69		3,06		3,62	

Figure 7.1. DGAC- Rut Test Result Chart

DGAC specimen test results have a consistency while comparing each result with another one. The first chart in Figure 7.1 which refers to reference specimen has lower value than other specimens except the last chart, *Tensar Glasstex* reinforced specimen.

DGAC-2 specimen has shown biggest resistance to rutting occurrence according to other specimen types. In order of specimens DGAC-2, DGAC-1, DGAC-3 have lower rut depth value according to reference specimen

Despite having a geosynthetic which has substantially similar technical specification according to DGAC-1 and DGAC-3, the DGAC-4 shown lower performance than reference specimen for rutting resistance. One of the possible causes should be that *Tensar Glasstex* was not stiff enough as the other geosynthetics. The installation of Tensar Glastex into asphalt slabs was too difficult so this can also be the reason for the poor behavior Moreover; this material did not provide enough fixations despite applying enough tack coat and waiting for its curing.



	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
	None		Aspha Glassgrid		Hatelit C 40 17		Synteen Composite		Tensar Glasstex	
	GGAC-C		GGAC-1		GGAC-2		GGAC-3		GGAC-4	
Ruth Depth [mm]	2,67	3,29	2,24	2,49	1,98	2,34	2,57	2,92	3,35	3,67
Average Value [mm]	2,98		2,37		2,16		2,75		3,51	

Figure 7.2. GGAC- Rut Depth Result Chart

GGAC specimens shown better performance according to DGAC specimens. This was an expected result. Because, upper course of GGAC specimens were SMA and it has better resistance ability for rutting occurrence and more durable than conventional HMA (the upper part of DGAC specimens).

All other results have a tendency in terms of rut depth in DGAC specimen chart. GGAC-2 specimen has also shown the biggest resistance ability to mitigate rutting occurrence when compared to other specimen types.

In order of specimens GGAC-2, GGAC-1, GGAC-3 have lower rut depth value according to reference specimen same as in DGAC specimen result order.

Despite having a geosynthetic which has substantially similar technical specification according to GGAC-1 and GGAC-3, the GGAC-4 shown lower performance than reference specimen for rutting resistance.

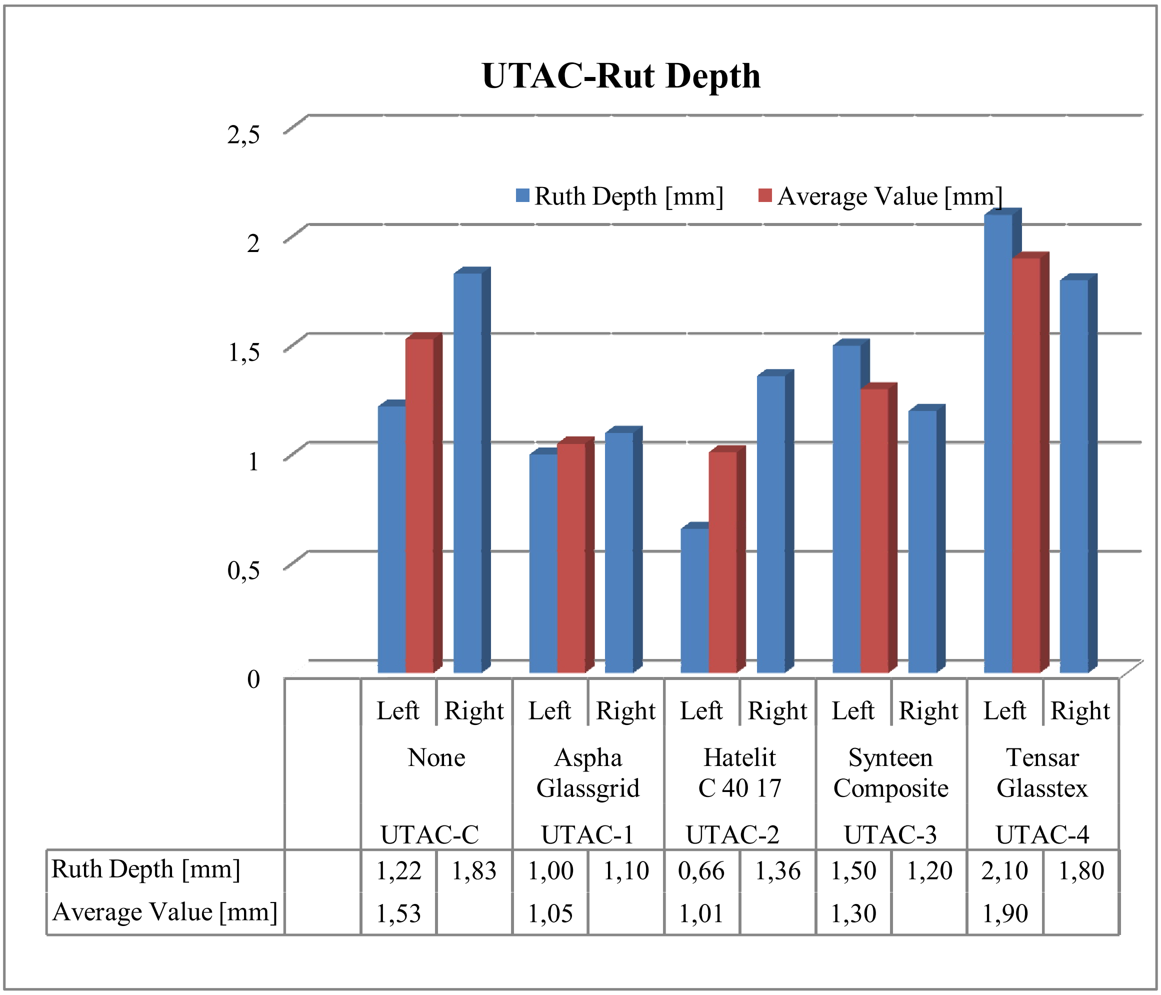


Figure 7.3 UTAC- Rut Depth Result Chart

UTAC rutting test results have carried consistency in other two specimen groups. UTAC –C control specimen has been resulted with lower rut depth value than other three specimens, usually except UTAC-4 specimen which reinforced *Tensar Glasstex.* The UTAC rutting test results represent that using of geosynthetics under the 2 cm overlay asphalt should be given positive result for rutting mitigation. All UTAC results are in parallel with DGAC and GGAC specimen result in terms of rut depth occurrence.

Rutting is one of the main distress types to lead to pavement failure and is difficult to track and simulate with deformation/strain measurements in majority of materials of asphalt concrete. The purpose of this research study was to investigate the effectiveness of geosynthetics in mitigation of rutting in asphalt concrete pavements**.** While starting to this study, the expected contribution of this study to the state of the art was to present a new laboratory study and its findings to help better understand rutting occurrence in asphalt concrete layer and its mitigation with the use of geosynthetic reinforcement. After a group of rutting tests, some results can be listed as following;

i). Both Dense Graded Asphalt Concrete (DGAC) and Gap Graded Asphalt Concrete (GGAC) tests has revealed that geosynthetic usage reduce rutting potential of pavements.

ii). The in-isolation tensile strength of the geosynthetic is not the major parameter that affects the pavement behavior. This was seen by the fact that the geosynthetic that has the lowest in-isolation tensile strength provided the best improvement.

iii). Installation ease is an important part of geosynthetic usage. Despite having good technical specification for reinforcement, due to difficulty in installation, Tensar Glasstex incorporated specimen did not provide good performance for rutting mitigation.

iv). Hamburg Wheel Tracking Device (HWTD) provides an opportunity of measuring of geosynthetic installed sandwich specimens' rutting potential.

v). Using of geosynthetics in Ultra Thin Asphalt Concrete (UTAC) provides good performance for rutting.

Istanbul Metropolitan Municipality's' highway maintenance and rehabilitation division and ISFALT ran some applications in their most important bus line, METROBUS.

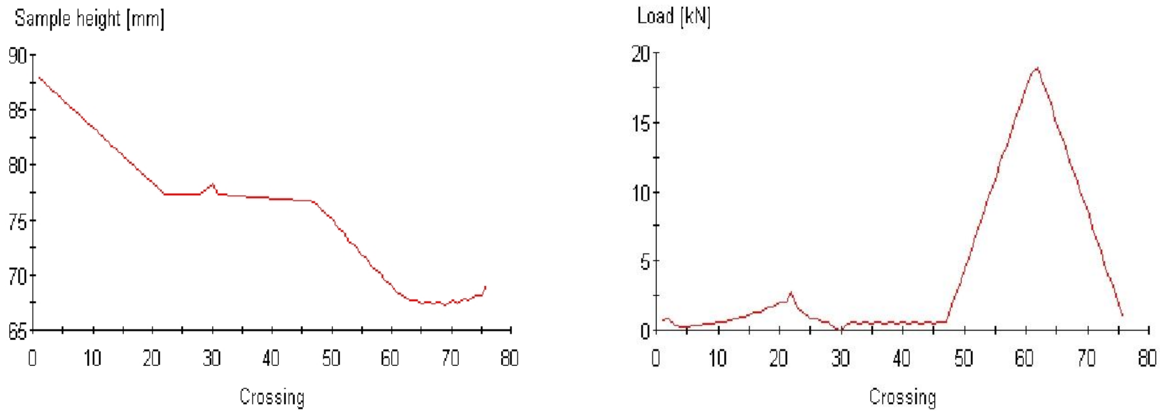
METROBUS line was constructed two years ago by the Istanbul Greater Municipality in order to provide public bus passengers a separate lane. In this separate lane, approximately 250 passenger capacity buses are used. Owing to less wandering of bus wheels, channelization in wheel truck occurs so that riding quality decreases. Rutting caused by braking and acceleration of buses in station sections has threatened driving safety. The Best performed geosynthetic in this study was used as reinforcement for rutting problems in station sections. (Figure 7.4, Figure 7.5)

Figure 7.4. Permanent Deformation on METROBUS Line Surface

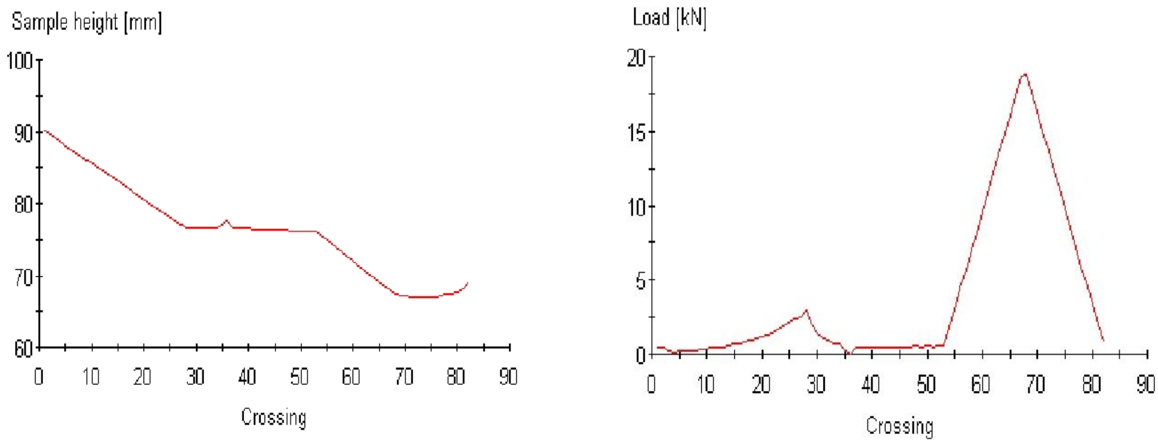
Figure 7.5. Geosynthetic application on METROBUS Line

APPENDIX A: DGAC (BINDER COURSE) COMPACTION RESULTS

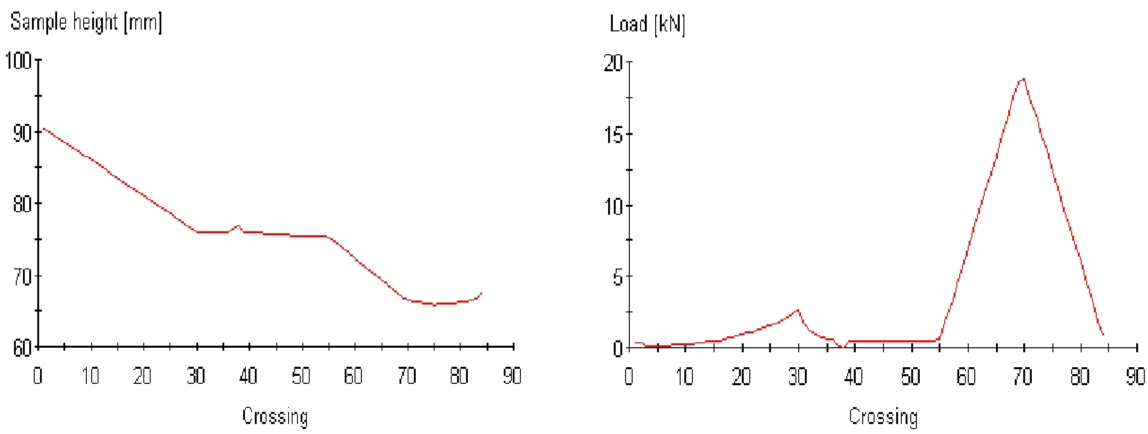
DGAC- C- Binder Course Compaction Result



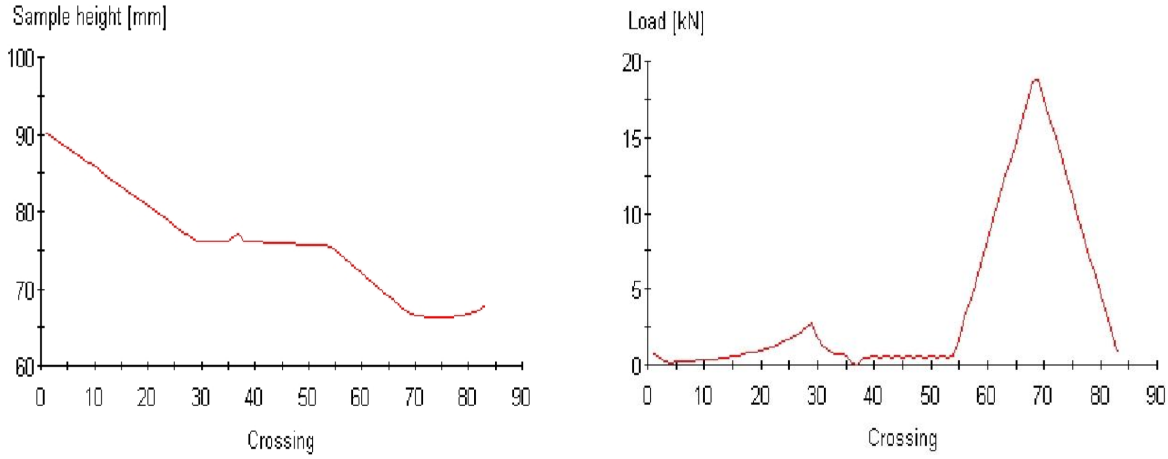
DGAC- 1- Binder Course Compaction Result



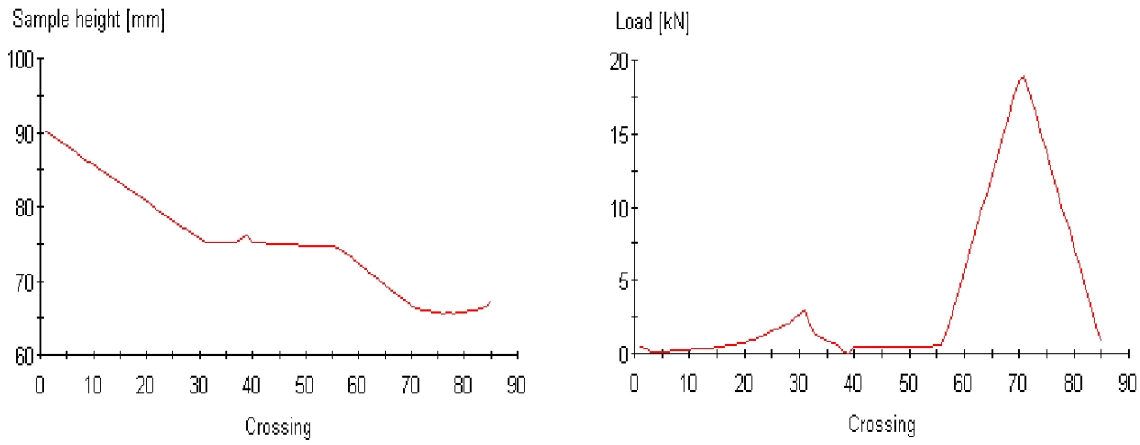
DGAC- 2- Binder Course Compaction Result



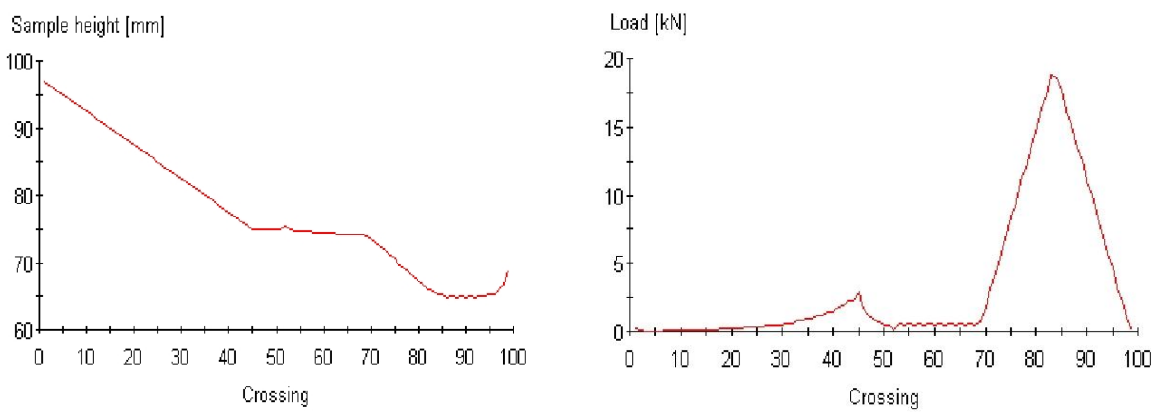
DGAC- 3- Binder Course Compaction Result



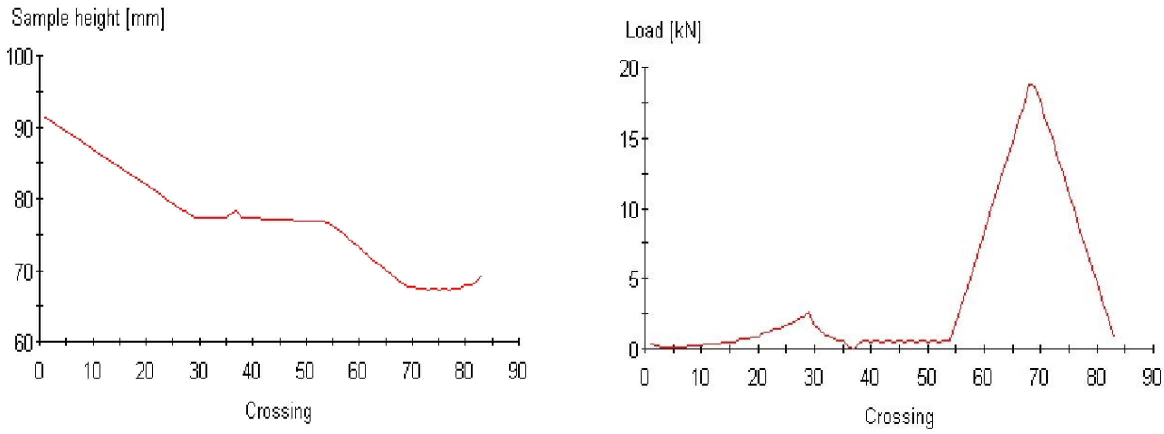
DGAC- 4- Binder Course Compaction Result



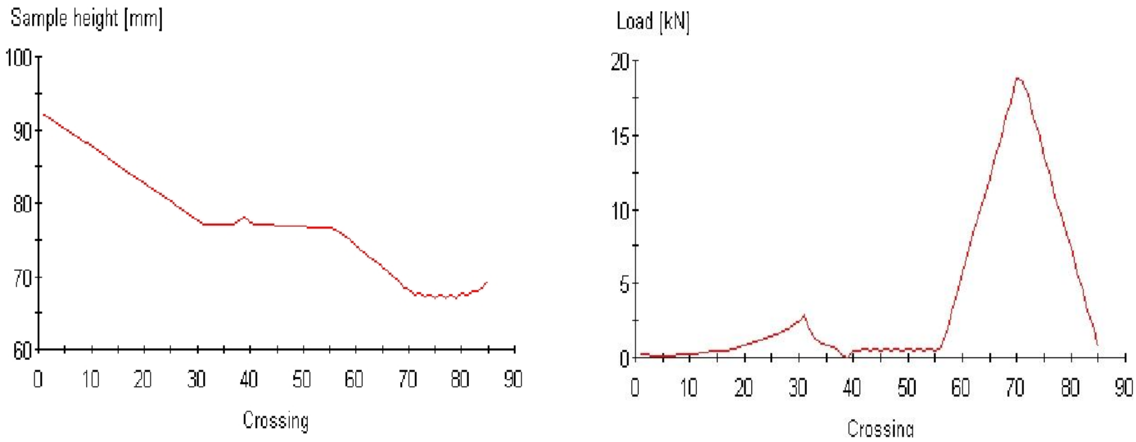
GGAC- C- Binder Course Compaction Result



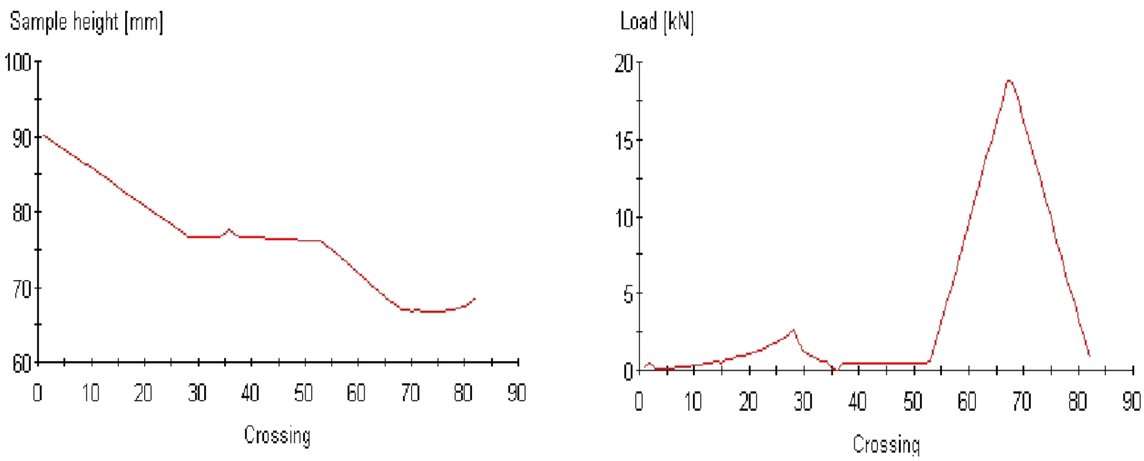
GGAC-1- Binder Course Compaction Result



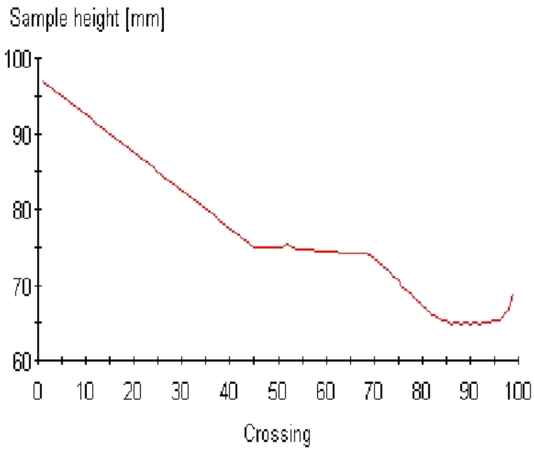
GGAC-2- Binder Course Compaction Result

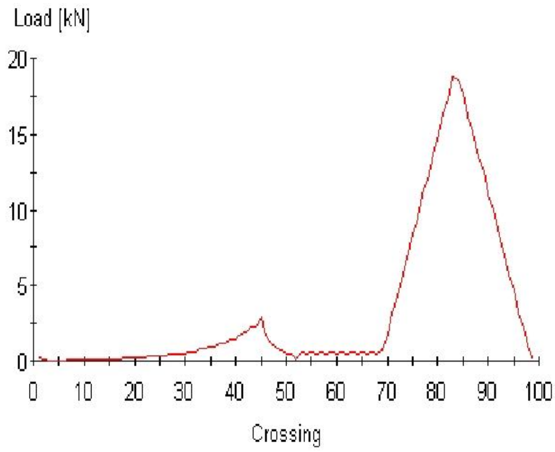


GGAC-3- Binder Course Compaction Result

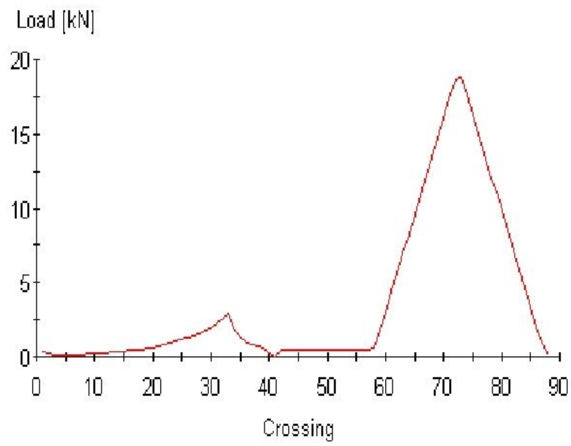


GGAC-4- Binder Course Compaction Result

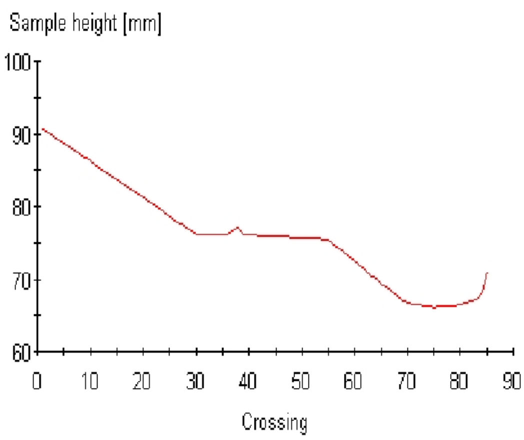


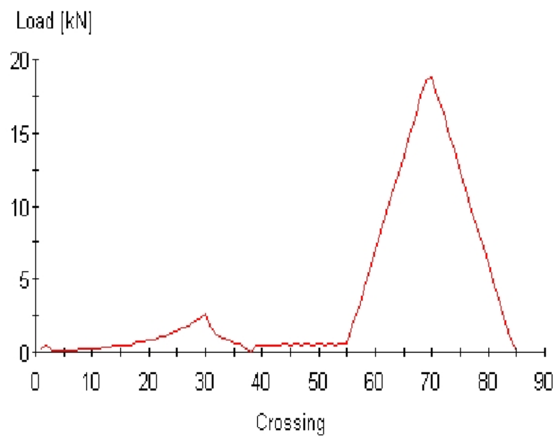


UTAC-C- Binder Course Compaction Result

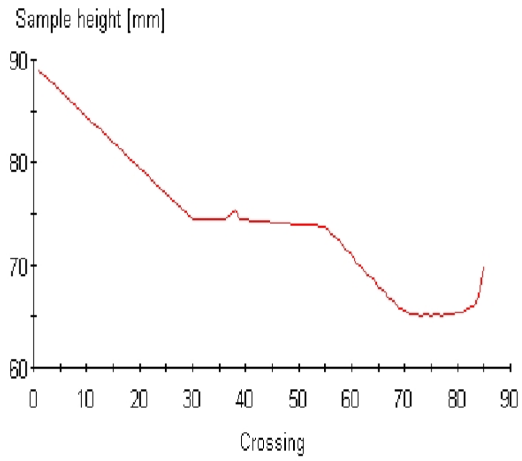


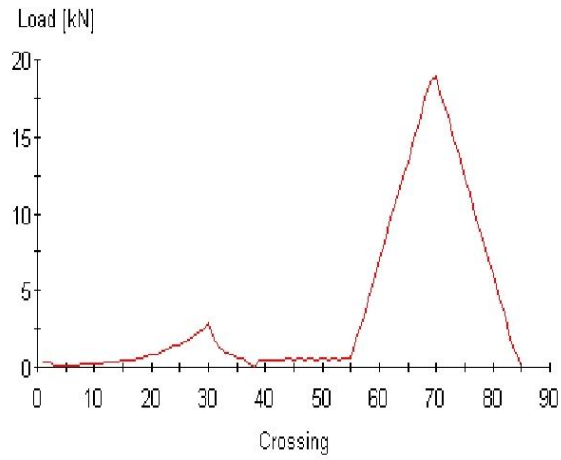
UTAC-1- Binder Course Compaction Result



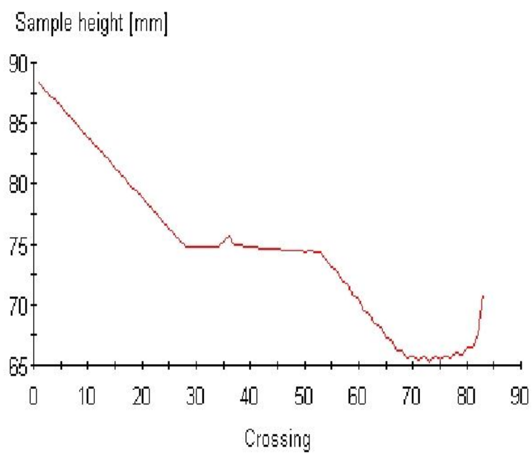


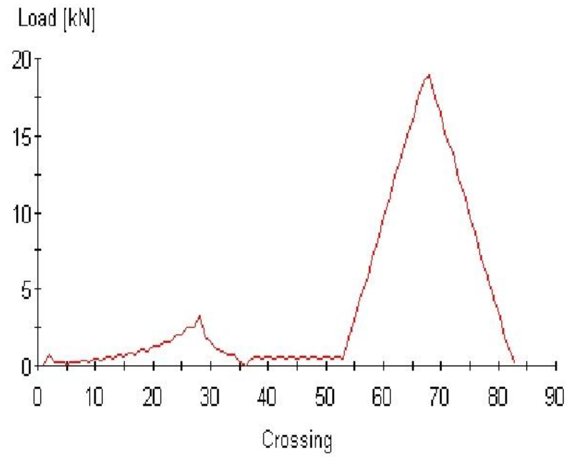
UTAC-2- Binder Course Compaction Result



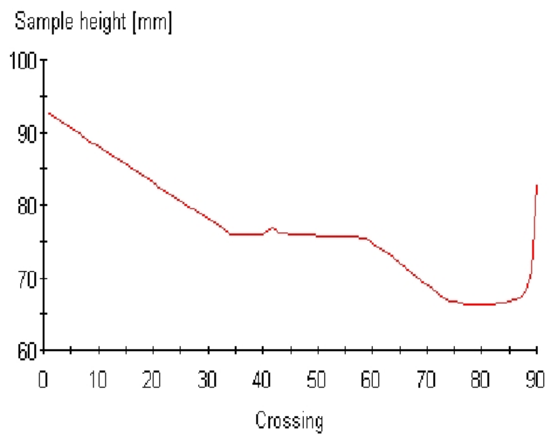


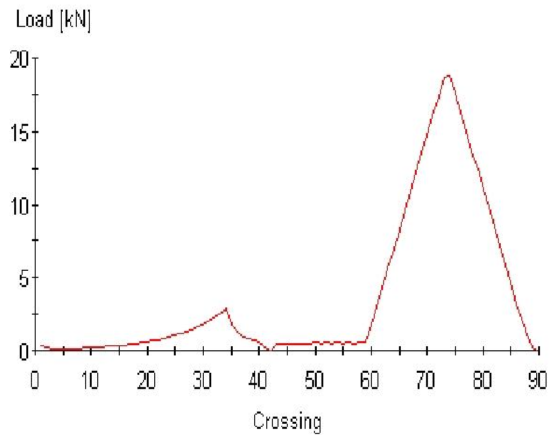
UTAC-3- Binder Course Compaction Result





UTAC-4- Binder Course Compaction Result





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