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Performance of flexible pavements enhanced using geogrid-reinforced asphalt overlays

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Dedication

*To Professor Benedito de Souza Bueno,
one of the research pioneers of
Geosynthetics in Brazil.*

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ABSTRACT

CORREIA, N. S. **Performance of flexible pavements enhanced using geogrid-reinforced asphalt overlays**. 2014. 205 p. Doctoral Dissertation – Sao Carlos School of Engineering of the University of Sao Paulo, Sao Carlos, SP, 2014.

The study of innovative pavements is of significant importance in geotechnical engineering in Brazil, due to the continued need to increase the network of roadways. This requires optimized projects, not only for economic, but also for technical reasons. Technical solutions that use geosynthetics in asphalt overlays have been identified to minimize fatigue and reflective cracks. However, the majority of the application of this technology has ignored the possible additional structural benefits brought by the inclusion of geosynthetics as reinforcement in asphalt layers. The objective of this research is to assess the reinforcement benefits of geogrids placed within asphalt overlays on the structural performance of flexible pavements. In addition, this study investigates the tensile-strain response of geogrids under traffic conditions, induced by cyclic wheel loads generated by a new accelerated pavement testing facility (APT) that was specifically developed for this research. The APT facility consists of a large steel testing box, in which field-scale pavement layers could be constructed. Pavement materials included subgrade soil, aggregate base, hot mix asphalt concrete, asphalt emulsion and a PVA geogrid. Pavement performance was assessed by applying a cyclic wheel load pressure of 700 kPa to the pavement surface. The pavement sections investigated in this study included a geogrid-reinforced and an unreinforced asphalt overlay sections, a single new geogrid-reinforced asphalt layer, and a geogrid-reinforced asphalt overlay with reduced base course thickness. A variety of sensors were used to measure asphalt concrete strains, surface plastic and elastic displacements, and induced traffic loads. Displacements along the geogrid specimens were measured using a tell-tail system. As result, several reinforcement mechanisms of this technique could be quantified in the present study. Polymeric geogrid reinforcements were found to have considerably reduced strains developed at the bottom of asphalt layers, as well as to have reduced vertical stresses in pavement lower layers. Resistance to rutting and lateral movement induced by the geogrids were also clearly evidenced in the presented study. The measurement of displacements along the geogrid provided understanding of the distribution of strains during traffic loading. A mobilized length was identified in geogrid-reinforced sections, showing that the bonding between geogrids and asphalt layers and the stiffness of the geogrid ensured satisfactory performance of the pavement sections. The results also illustrated that the lateral restraining mechanisms effect is a governing mechanism to improve the performance of the asphalt layers by the development of shearing resistance with the geogrids. Overall, it was concluded that geogrids within asphalt overlays act as reinforcement and not merely to delay cracks, providing enhanced performance to flexible pavement structures.

Keywords: geogrid; asphalt overlay; reinforcement; flexible pavement; accelerated pavement testing.

RESUMO

CORREIA, N. S. Desempenho de pavimentos flexíveis utilizando geogrelha como reforço de capa asfáltica. 2014. 205 p. Tese de Doutorado – Escola de Engenharia de São Carlos – Universidade de São Paulo, São Carlos, SP, 2014.

O estudo de pavimentos é de grande importância na Engenharia Geotécnica brasileira devido à crescente necessidade de melhora da situação da rede rodoviária nacional. Para tanto, o desenvolvimento e a aplicação de novas técnicas são necessários, principalmente no âmbito econômico. A técnica do uso de reforços geossintéticos em capa asfáltica é identificada como uma alternativa ao aumento da vida útil do pavimento através da mitigação de trincas por fadiga e de reflexão. No entanto, a maioria das aplicações desta técnica não correlaciona os benefícios estruturais da inclusão do geossintético na capa asfáltica para a melhora do desempenho global do pavimento. O objetivo desta pesquisa é investigar os benefícios estruturais no desempenho de pavimentos flexíveis trazidos pelo reforço de geogrelhas em camadas asfálticas. Ainda neste estudo, será investigada a reposta tensão-deformação destas geogrelhas sobre as condições de tráfego através do uso de ensaios acelerados de pavimento. Um equipamento foi desenvolvido para esta pesquisa e consiste numa caixa metálica de grande porte, em que seções de pavimento em escala real podem ser construídas. O desempenho das seções de pavimento foi avaliado com a aplicação de cargas cíclicas de roda com pressão de contato de 700 kPa. Os materiais que compõem as seções de pavimento incluem solo de subleito, brita graduada simples, concreto betuminoso usinado à quente, emulsão asfáltica e geogrelha de PVA. Foram estudadas uma seção com geogrelha como reforço no recapeamento da camada asfáltica, uma seção idêntica não reforçada, uma seção com uma única capa asfáltica reforçada com geogrelha e uma seção com geogrelha no recapeamento da camada asfáltica, porém com espessura de base reduzida em relação aos demais ensaios. Sensores nas camadas do pavimento mediram tensões e deformações, e deslocamentos plásticos e elásticos na superfície. Deslocamentos ao longo da geogrelha foram monitorados utilizando o sistema tell-tales. Como resultado, mecanismos de reforço foram identificados neste estudo. O uso de uma geogrelha polimérica reduziu consideravelmente as deformações na fibra inferior da capa asfáltica, assim como as tensões verticais nas camadas subjacentes do pavimento. Resistência à formação de trilhas de roda e sollevamentos laterais foram também evidenciadas. As medidas de deslocamentos ao longo da geogrelha forneceram entendimento da distribuição de deformações durante o carregamento. Foi identificado o comprimento de geogrelha mobilizado durante os ensaios, mostrando que a aderência entre a geogrelha e as camadas asfálticas e a rigidez da geogrelha asseguraram o desempenho satisfatório das seções de pavimento. Os resultados também mostraram que o efeito do mecanismo de restrição lateral é um mecanismo que governa a melhora no desempenho da capa asfáltica com o uso da geogrelha através do desenvolvimento de resistência ao cisalhamento. Estas observações permitem concluir que a geogrelha na camada asfáltica atua como reforço e não apenas reduzindo o potencial de trincamento, levando a um aumento no desempenho de estruturas de pavimentos flexíveis.

Palavras-chave: geogrelha; camada asfáltica; reforço; pavimento flexível; ensaios acelerados de pavimento.

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1 INTRODUCTION

1.1 Motivation

The study of road pavements is of significant importance in geotechnical engineering due to the continued need to increase the network of adequate roadways in Brazil. This requires better projects, not only for economic, but also for technical reasons. The use of geosynthetic-reinforced asphalt overlays may lead to significant advances in pavement rehabilitation by extending the life of roadways and consequently reducing maintenance costs.

Recently, technical solutions that use geosynthetics in asphalt overlays have been identified as an innovative approach to minimize fatigue and reflective cracks. The geosynthetics used in these solutions can minimize, redirect or intercept cracks appearing on the asphalt surface. However, the majority of the application of this technology has ignored the possible additional structural benefits brought by the inclusion of geosynthetics as reinforcement in asphalt layers. Instead, their use has solely focused on the minimization of reflective cracks in road retrofitting projects. In spite of the successful experience accumulated over the years on using geosynthetics to minimize fatigue and reflective cracks, the geosynthetic reinforcement function is not yet fully understood.

Several types of geosynthetics for pavement rehabilitation, known as geosynthetics for asphalt reinforcement are currently available. Geosynthetics can improve long-term structural performance, reducing the frequency of maintenance actions. Nevertheless, there is still skepticism by engineers regarding the use of geosynthetics as a structural reinforcement in asphalt layers, mainly because the reinforcement benefit has not yet been adequately fully quantified. Use of geosynthetic-reinforced asphalt systems demand a thorough understanding of the geosynthetic reinforcement mechanisms and their interaction with the asphalt layer. Additionally, there is no well-established design methodology to this application or a unified approach for construction. Currently, the selection of the appropriate type of geosynthetic and its position within the pavement layer is based on empirical criteria or sometimes defined from the results of laboratory tests.

Most of the current knowledge about using geosynthetics fulfilling a reinforcement function in pavement applications has focused on the reinforcement of aggregate base and subgrade layers. While relevant, the mechanisms identified for geosynthetic-reinforced base layers cannot be directly applied to understand the use of geosynthetics in reinforced asphalt

layers. Additional data is needed to understand the mechanisms by which geosynthetics can reinforce asphalt layers, as well as the overall structural benefits provided by this technique. A valuable approach to understand the reinforcement mechanisms in pavements involves studying the development of strains in the geosynthetic. This information would provide not only good understanding of the relevant mechanisms but would also facilitate the development of methodologies to predict the reinforced pavement response. So far, only limited studies have been conducted on the use of geogrids within asphalt layers to evaluate the pavement structural response. However, none of them have investigated the tensile strain response in the geosynthetics and their mobilized tension and distribution when subject to traffic loading conditions.

Therefore, the aim of this research is to improve the understanding of the reinforcement benefits of using geogrids within asphalt overlays in flexible pavements. An accelerated pavement testing facility was developed specifically for this study. The influence of geogrid reinforcement in asphalt layers was examined. The investigation was carried out by conducting laboratory large-scale instrumented pavement sections, loaded using a rolling wheel simulating a truck wheel load. In order to examine the reinforcement mechanism of geogrids within asphalt overlays, tensile strains mobilized in the geogrids and asphalt deformations were also investigated in this study. In addition, the dynamic mechanical response of the pavement sections under traffic loading was thoroughly examined. The findings of this study are expected to provide basis for future design approaches of geosynthetic pavement rehabilitation systems, particularly regarding their reinforcement benefits in flexible pavements.

1.2 Overall Objective

The objective of this research is to assess the reinforcement benefits of geogrids placed within asphalt overlays on the structural performance of flexible pavements. In addition, this study will investigate the tensile-strain response of geogrids under traffic conditions, as represented by cyclic moving wheel loads. This information is expected to provide the understanding of the mechanisms in reinforced asphalt overlays, in particular, related to the structural improvement of pavements. Tests were conducted using an accelerated pavement testing facility, which was specifically developed for this research.

1.3 Specific Objectives

In order to achieve the overall objective, this research will seek the following specific objectives:

- Build an accelerated pavement testing facility at USP Sao Carlos, which involves loading using a rolling wheel simulating a truck wheel load over large-scale instrumented pavement sections.
- Evaluate the reinforcement benefits of polymeric geogrids on the performance of flexible pavements by quantifying the permanent surface deformations (rutting), deflection basins, the load-induced vertical stresses in the pavement layers and the strains in asphalt concrete layers of different pavement configurations.
- Compare the performance of a geogrid-reinforced asphalt overlay pavement section with that of an unreinforced overlay section.
- Compare the performance of a geogrid-reinforced a single asphalt layer section with that of overlay pavement sections.
- Compare the performance of an unreinforced overlay section with that of a geogrid-reinforced asphalt overlay section with reduced base course thickness.
- Investigate the displacement and strain distribution along the transverse ribs of geogrids under traffic loads, compare this information against strains developed in unreinforced asphalt overlay interfaces in order to better understand the reinforcement mechanisms that takes place in this application.

1.4 Dissertation Structure

This dissertation includes seven chapters. After presenting the motivation and objectives discussed in Chapter 1, Chapter 2 provides the background information on key elements involved in this research. This information includes geosynthetic-reinforced pavement structures, geogrid asphalt overlay studies (laboratory and field investigations) concerning reflective cracking and rutting prevention, and geogrid asphalt overlay stress-strain mobilization. Chapter 3 presents the equipment built to conduct this research. Chapter 4 describes the materials used to build the pavement sections, the instrumentation and data acquisition system, and the matrix of pavement section tests. Chapter 5 shows the results corresponding to each pavement section, obtained using the accelerated pavement testing

device and proposed instrumentation. Chapter 6 includes the main discussion regarding the obtained results, comparing reinforced pavement testing structures to similar non-reinforced structures. Chapter 7 presents the conclusions from the current study and recommends directions in which further research should be conducted.

2 BACKGROUND INFORMATION

This chapter presents the background information on the benefits of geosynthetics as reinforcements in asphalt overlays that motivated this research. A general description of geosynthetics reinforcements in flexible pavements is presented. Laboratory and field test methods aiming at understanding the reinforcement benefits of geosynthetic-reinforced asphalt pavements are also reported in this chapter, including a literature review to support experimental results obtained from experimental program.

2.1 Geosynthetic-reinforced flexible pavements

With an increasing traffic intensity, high axle loadings and super-single tires, an aging transportation infrastructure, insufficient maintenance, as well as political difficulties to achieve widening of roads, there is a need to optimize the return from the transportation infrastructure investments. The advent of new technologies to improve pavement performance and increase its service life can play an important role in improving the design and rehabilitation on pavement systems.

In the past decades, many technologies have been introduced and presented as systems to reduce premature pavement failures. Some of the latest and successful techniques have included the incorporation of geosynthetic products into the pavement structure to improve its performance. Geosynthetics can provide significant improvement in pavement construction and performance when installed in pavement layers. Geosynthetics are used in different functions in the pavement structure: reinforcement (subgrade stabilization, base reinforcement and asphalt reinforcement), separation (prevent fines from migration), filtration and drainage, as indicated in Figure 2.1. Indeed, certain geosynthetic products can perform multiple functions in a pavement layer; while similarly, the same function can often be performed by different types of geosynthetics (Zornberg and Christopher, 2007).

Because of the need to extend pavement service life-time, the use of geosynthetics in pavement rehabilitation projects has received increased attention nowadays and is becoming even more popular with the growing evidenced about benefits related to pavements improvement.

Figure 2.1. Potential applications of geosynthetics in layered pavement system.

Practical applications and theoretical studies have indicated that the service life of flexible pavements can be extended by installing nonwoven geotextiles or geogrids between existing and new asphalt layers due to the improved mechanical properties of pavements in terms of fatigue, rutting, and reflective cracking resistance (Lytton, 1989; Saraf et al., 1996; Austin and Gilchrist, 1996; Button and Lytton, 2007; Khodaii et al., 2009; Pasquini et al. 2013). In Brazil, studies on this alternative solution have also become more frequent (Montestruque et al., 2004; Azambuja 2004; Fritzen 2005; Buhler, 2007; Correia and Bueno 2011; Montestruque et al., 2012; Obando 2012). However, no effective method has been developed to quantitatively assess geosynthetic interlayer systems in asphalt overlays and accurately predict pavement performance (Sanders, 2001; Moses 2011). According to Graziani et al. (2014), current selection of the appropriate geosynthetic type and position is based on empirical criteria or derived from the results of laboratory tests.

2.2 Geosynthetics in asphalt overlays: Installation and specifications

Geosynthetics can be placed at the interface of bituminous layers for both new constructions and rehabilitation of existing pavements. The AASHTO Design Guide (1993) suggests that paving geosynthetics can be effective in controlling (reducing) reflection cracking from low and medium severity alligator-cracked pavements and that they may also help control the reflection of thermal cracks. This procedure is typically accomplished by attaching a geosynthetic product to the existing pavement (flexible or rigid) with an asphalt tack coat and then overlaying a hot mix asphalt layer with a specified thickness. Figure 2.2 presents an example of geogrid installation using this technique.

(a)

(b)

Figure 2.2. Geogrid installation in asphalt overlays: (a) field work by Button and Lytton (2003); (b) field work by Laurinavičius and Oginskas (2006).

In Brazil, the use of geosynthetic reinforcement in asphalt layers as anti-reflective cracking system is often recommended for pavements severely cracked or rutted. It is specified by DER ET-DE-P00/043: Anti-reflective cracking treatment with Geosynthetics (2006). This specification governs the sampling, testing, material requirements, and construction methods for geosynthetics in asphalt overlays. Table 2.1 presents the DER ET-DE-P00/043 (2006) construction process and specifications for both nonwoven geotextiles and geogrid products.

In the construction process, the sealant shall be applied by distributor spray bar. The asphalt emulsions are the preferred sealant. Temperature of the asphalt sealant (emulsion) is usually 20 °C to 50 °C. The emulsion shall be cured prior to placing the paving fabric and final wearing surface. This means essentially that no moisture remains. The use of asphalt cement must be approved by supervisory or indicated by the project. The specified rate of asphalt sealant application (tack coat) must be sufficient to satisfy the asphalt retention properties of the paving fabric, and bond the geosynthetic and overlay to the existing pavement. The rate of asphalt sealant for nonwoven geotextiles depends on pavement surface condition, while for geogrids it varies from 0.40 to 0.60 l/m² (residual tack coat).

In other countries, there is a trend in using asphalt cement or polymer modified asphalt emulsions. The AASHTO ME 288-05 (2001) suggests asphalt cement as tack coat and bases the rate of asphalt sealant on the asphalt retention properties of the geosynthetic (ASTM D 6140-05). The TXDOT 3285 (2004) specification suggests the use of trackless tack coat (hot applied emulsion) with a residual application rate of 0.48 to 0.7 l/m². Indeed, there have been discussions about the rate and type of asphalt sealants (Correia et al., 2014) with still no consensus on the best installation and construction process. Differences in the installation

process have been found in specifications, as well as unsuccessful studies cases showing unexpected pavement performance.

Table 2.1. Construction process: DER ET-DE-P00/043 (2006) Anti-reflective cracking treatment with Geosynthetics

Construction process	DER ET-DE-P00/043 (2006) - Anti-reflective cracking treatment with geosynthetics
Surface preparation	The surface on which the paving geosynthetic will be placed shall be reasonably free of dirt, water, vegetation and other debris.
Surface condition	Cracks exceeding 3 mm in width shall be filled with suitable crack filler. Potholes shall be properly repaired as directed by the Engineer. Fillers shall be allowed to cure prior to paving fabric placement.
Nonwoven geotextile application	<p>Nonwoven geotextiles shall be placed onto the first application of asphalt sealant (asphalt emulsion), after surface preparation (with minimum wrinkling during installation). It is recommended 70 to 80% of the total asphalt sealant rate (residual). The emulsion shall be cured prior to nonwoven geotextile placement.</p> <p>The nonwoven geotextile can be placed manually or mechanically. Pneumatic rolling will be required to maximize geosynthetic contact with the pavement surface. Compaction of 0.28 MPa a 0.35 MPa (2.8 kg/cm² a 3.6 kg/cm²). The compaction process will be used to promote the reverse impregnation and better saturate the geotextile.</p> <p>The second application of asphalt sealant, after geotextile placement, will be that to total 100% of the asphalt sealant rate (residual).</p> <p>A certain amount of asphalt concrete will be manually applied on the nonwoven geotextile, to assure the material protection since heavy traffic vehicles may damage the material. Trafficking is not permitted.</p>
Geogrid application	<p>Geogrids shall be placed onto the asphalt sealant (minimum 0.4 l/m²) under sufficient tension to eliminate wrinkling and ripples.</p> <p>If the surface is too cracked or porous, the asphalt sealant quantity must be increased. The emulsion shall be cured prior to geogrid placement.</p> <p>The geogrid can be placed manually or mechanically. Trafficking on the geogrid will be permitted for construction vehicles and emergency only.</p>
Asphalt concrete application	<p>Placement of the hot-mix overlay should closely follow pre-concrete asphalt application or geogrid installation.</p> <p>The temperature of the mix shall not exceed 160 °C.</p> <p>A minimum 3.0 cm of asphalt concrete over the geotextile. A Minimum 4.50 cm of asphalt concrete should be placed over the geogrid.</p> <p>To avoid movement of, or damage to, the seal-coat saturated paving fabric, turning of the paver and other vehicles shall be gradual and kept to a minimum.</p>

Cleveland et al. (2002) have reported an unsuccessful field study on geosynthetics reinforcing asphalt overlays at Pharr, Waco and Amarillo-Texas USA. Polymeric and fiber glass geogrids, nonwoven geotextiles and geogrid composites were used in the case studies. Based on this study, no conclusions could be reached regarding the life-cycle cost effectiveness of the various evaluated products. Several installation problems were reported. Also, the road lanes did not have the same surface condition, leading to a difficult evaluation of serviceability. Thus, it was not possible to show which category of products had the best performance.

A poorly executed installation may impair the working conditions of geosynthetics within the asphalt layer, decreasing its efficiency on pavement performance. As important as the installation procedure, the selection of the geosynthetic material is also an open question. According with DER ET-DE-P00/043 (2006), nonwoven geotextiles and geogrids can be used if they meet relevant physical and mechanical specifications. Nonwoven geotextiles, for example, must have tensile strength greater than 7 kN/m, while for geogrids and geogrid composites, the tensile strength property should exceed 50 kN/m (ultimate elongation inferior to 12%). However, there is no design methodology for choosing tensile properties of geosynthetics for asphalt layer reinforcement. This material specification is strictly empirical. Among several similar specifications, TXDOT 3285: Reinforcement grid for asphalt pavement overlays (2004) does not include tensile strength properties required for geogrids, while AASHTO ME 288-05: Standard Specification for Geotextile Specification for Highway Applications (2001) is based on grab tensile properties.

Among different geosynthetic categories, polymeric and fiber glass geogrids and geogrid composites (combining a nonwoven geotextile and a reinforcing geogrid) are the ones mostly used in pavement rehabilitation due to its high stiffness in comparison to impregnated nonwoven geotextiles. Hence, the selection is not limited to one type of product (e.g. geogrid or nonwoven geotextile), but, rather how mechanical properties of the geosynthetic and geosynthetic-asphalt layer interaction properties impact the reinforcement benefits in pavements performance.

2.3 Mechanisms of geosynthetics in asphalt overlays

Although there is research reporting the possible mechanisms governing the behavior of geosynthetics in asphalt overlays, there is no consensus on the reinforcement mechanisms. The main functions generally attributed to geosynthetic reinforcements in asphalt layers are described as increase in asphalt fatigue life, thermal and reflective cracking mitigation, with

three associated group of mechanisms: strain-relief, waterproofing and reinforcement (Lytton, 1989; Sprague et al., 1998; Maurer and Malasheskie 1989; Khodaii et al., 2009). Figure 2.3 illustrates the mechanisms attributed to geosynthetics in asphalt overlays.

Figure 2.3. Mechanisms attributed to geosynthetic in asphalt overlays: strain-relieving, waterproofing and reinforcement.

A stress relief interlayer is a soft layer that is usually placed at the bottom of an HMA overlay to absorb a large portion of the energy that would otherwise contribute to the crack propagation process (Barksdale, 1991). A geosynthetic stress-relief interlayer will act retarding the development of reflective cracks in the overlay by absorbing the stresses induced by the underlying cracking in the existing pavement. The stress is absorbed by allowing slight movements within the geosynthetic interlayer inside the pavement without distressing the asphalt concrete overlay significantly. In fact, the addition of a stress-relief interlayer reduces the shear stresses between the existing pavement and the new overlay, creating a break layer that gives the overlay a degree of independence from movements in the existing pavement (de Bondt, 1999). Pavements with geosynthetics interlayers also experience much less stresses from internal cracks development than those without (Shukla and Yin, 2004). This is why fatigue life of a pavement with a geosynthetic interlayer can be many times that of a pavement without it (Marienfeld and Guram, 1999; Hosseini et al., 2009; Mounes et al., 2011).

Moisture is frequently the main source of pavement damage and roughness. In the early stages of development, reflection cracks may barely be visible and may not considered a structural problem (Button and Lytton, 2007). However, once a reflective crack reaches the surface, an open path exists for the flow of surface water into the lower layers of the pavement.

If untreated, this situation will lead to deterioration of the pavement structure and a reduction in overall serviceability. One of the geosynthetic functions is that they can act as means to prevent moisture from infiltrating into the pavement structure. Such waterproofing action may limit the base and subgrade movement, consequently, delaying the deterioration of the pavement structure (Button and Lytton, 2007). Asphalt-impregnated fabrics or geosynthetic composites control infiltration of surface water into a pavement and may remain intact after the asphalt overlay has cracked, providing a moisture barrier (Marienfield and Baker, 1999). The paving geosynthetic must be saturated with sufficient asphalt to provide a continuous moisture barrier; insufficient tack would diminish this waterproofing effect (Lytton 1989; Hosseini et al., 2009, Correia and Bueno, 2011, Pasquini et al., 2013). However, if a moisture barrier is necessary, paving fabrics and composites offer this added benefit, which cannot be expected when using geogrids (Button and Lytton, 2007).

A geosynthetic interlayer system can also add tensile strength to the existing pavement and/or the new asphalt overlay. The reinforcement will contribute to increase the fatigue life of the overlay because it retains stiffness even after the asphalt has lost stiffness due to repeated traffic loading (Austin and Gilchrist, 1996). For a pavement with geosynthetic reinforcement, whenever a crack starts to form in the pavement overlay, the ribs of the geogrid will develop tensile stress subsequent to the movement or opening of the crack surface. With this, the geogrid can restrain the growth of the opening fracture. As the growth of the crack is restrained, the vertical shearing displacement of crack surface will be prevented or reduced because of the interlocking effects between aggregates, and hence prolong the fatigue life of the pavement. When the crack extends from the overlay to the reinforcing layer, the geogrid will elongate itself to absorb the energy at the tips of the crack. The absorbing effect will weaken the energy balance at the tips of the crack and decelerate the plastic blunting process, and hence hinder the growth of the crack (Chang et al., 1998). According with Sobhan et al. (2005), embedded geogrids can provides physical reinforcement as well as energy absorption. Another function is from the deformation of the geogrid which absorbs the energy at the tips of the crack and thus stops the crack from growing outward. The reinforcement will strengthen the interlock of the crack edges and weaken the shear stress concentration around the crack tips (Saraf et al., 1996).

By incorporating geosynthetics in asphalt layers, the following effects are expected: absorption of thermal stresses; absorption of traffic loads deformations, deviation of reflective cracking and, consequently, delaying and reducing the surface cracking reflection and providing the waterproofing. Although technical solutions using geosynthetic reinforcements

in asphalt overlays have been mostly indicated to minimize fatigue and reflection of cracks, the great majority of solutions are not associated to the function of absorbing traffic loads deformations. In general, the deformations due to pavement stresses are relatively low and, consequently, traffic-induced stresses will be absorbed by the geosynthetic reinforcement, changing the response of the pavement to loading (Mounes et al., 2011). In spite of the successful experience accumulated after decades of using geosynthetics to minimize fatigue and reflective cracks, the reinforcement function of this rehabilitation system is not yet fully understood.

2.3.1 Reinforcement mechanisms in geosynthetic-reinforced pavements

Although there has been limited detailed studies on the reinforcement mechanisms of geosynthetics in asphalt overlays, important concepts on the reinforcement mechanisms of geosynthetic-reinforced base and subgrade layers of flexible pavements have been established.

The purpose of using geosynthetics as reinforcements in layered pavement system is to increase the structural or load-carrying capacity of a pavement system by transferring the load to the geosynthetic material (Holtz et al. 1998). The addition of geosynthetics for reinforcement purposes generally involves placing them at the interface between the base and sub-base layers or the interface sub-base and subgrade layers.

Flexible pavements are constructed so that traffic loads are distributed from the surface to underlying layers. The pavement flexes under the load, resulting in a stress distribution that spreads the load over a large areas. The improved performance of the pavement due to geosynthetic reinforcement at the base or subgrade layers of flexible pavements has been attributed to three mechanisms: (1) lateral restraint, (2) increased bearing capacity, and (3) tensioned membrane effect (Giroud and Noiray 1981, Giroud et al. 1984, Perkins and Ismeik, 1997, Holtz et al. 1998, US Army Corps ETL 1110-1-189, 2003, Zornberg, 2011). These three mechanisms are illustrated in Figure 2.4.

The primary mechanism associated with the reinforcement function for flexible pavements (Figure 2.4a) is lateral restraint or confinement (Bender and Barenberg 1978). When an aggregate layer is subjected to traffic loading, the aggregate tends to move laterally unless it is restrained by the subgrade or by geosynthetic reinforcement. Interaction between the aggregate base and the geosynthetic allows transfer of the shearing load from the base layer to a tensile load in the geosynthetic. The tensile stiffness of the geosynthetic limits the lateral strains in the base layer. Furthermore, a geosynthetic layer confines the base course layer

thereby increasing its mean stress and leading to an increase in shear strength. Both frictional and interlocking characteristics at the interface between the soil and the geosynthetic contribute to this mechanism. For a geogrid, this implies that the geogrid apertures and base soil particles must be properly sized (Perkins and Ismeik, 1997). A geotextile with good frictional capabilities can also provide tensile resistance to lateral aggregate movement (Zornberg, 2011).

Figure 2.4. The reinforcement functions attributed to geosynthetic in roadways: (a) lateral spread; (b) bearing capacity; (c) tensioned membrane (Perkins and Ismeik, 1997).

As illustrated in Figure 2.4b, the increased bearing capacity mechanism leads to soil reinforcement when the presence of a geosynthetic imposes the development of an alternate failure surface. This new alternate plane provides a higher bearing capacity. The geosynthetic reinforcement can decrease the shear stresses transferred to the subgrade and provide vertical confinement outside the loaded area. The bearing failure mode of the subgrade is expected to change from punching failure without reinforcement to general failure with reinforcement. The geosynthetic can also be assumed to act as a tensioned membrane, which supports the wheel loads (Figure 2.4c). In this case, the reinforcement provides a vertical reaction component to the applied wheel load. This tensioned membrane effect is induced by vertical deformations, leading to a concave shape in the geosynthetic. The tension developed in the geosynthetic contributes to support the wheel load and reduces the vertical stress on the subgrade. However, significant rutting depths are necessary to identify this effect. Higher deformations are required to mobilize the reinforcement (Perkins and Ismeik, 1997). This reinforcement mechanism has been reported by Barksdale et al. (1989) to develop only in cases with subgrade CBR values below 3.

According to Perkins (1999), the reinforcement function develops through the interaction between base-course layer and geosynthetic contained in or at the bottom of the base-course layer or subgrade, potentially consists of four separate reinforcement mechanisms:

- *Reduction of lateral spreading (lateral restraint)*
- *Increase in stiffness of a pavement layer*
- *Vertical Stress Distribution in the lower layers*
- *Reduction of shear stress in the subgrade*

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. Lateral movement of the base allows for vertical strains to develop leading to permanent surface deformation in the wheel path. Placement of a geosynthetic layer or layers in the base course allows for shear interaction to develop between the aggregate and the geosynthetic as the base attempts to spread laterally. Shear load is transmitted from the aggregate base to the geosynthetic and places the geosynthetic in tension. The relatively high stiffness of the geosynthetic acts to retard the development of lateral tensile strain in the base adjacent to the geosynthetic. Lower lateral strain in the base results in less vertical deformation of the roadway surface. Hence, the first reinforcement mechanism corresponds to direct reduction of lateral spreading of the aggregate base. Shear stress developed between the base-course aggregate and the geosynthetic provides an increase in lateral stress within the base. This increase in lateral confinement leads to an increase in the mean effective stress (Perkins, 1999).

The second reinforcement mechanism results from an increase in stiffness of the base-course aggregate when adequate interaction develops between the base and the geosynthetic. The increased stiffness of this layer results in lower vertical strains in the base. It would also be expected that an increase in modulus of the base would result in lower dynamic, recoverable vertical deformations of the roadway surface, implying that fatigue of the asphalt concrete layer would be reduced. The presence of a geosynthetic layer in the base can also lead to a change in the state of stress and strain in the subgrade. For layered systems, where a less-stiff subgrade material lies beneath the base, an increase in modulus of the base layer results in an improved, more broadly distributed vertical stress on the subgrade. In general, the vertical stress in the base and subgrade, directly beneath the applied load, should decrease as the base layer stiffness increases. The vertical stress on the subgrade becomes more widely distributed, meaning that surface deformation will be less and more uniform. Hence, a third reinforcement mechanism results from an improved vertical stress distribution on the lower layers of pavement (Perkins 1999).

The fourth reinforcement mechanism results from a reduction of shear stress in the subgrade soil. Love et al. (1987) demonstrated, using scaled model monotonically loaded tests involving a granular base over a weak subgrade and for 50 mm of footing penetration, that shear stress transmitted from the base course to the subgrade decreases as shearing of the base transmits tensile load to the reinforcement. Less shear stress, coupled with less vertical stress results in a less severe state of loading, leading to lower vertical strains in the subgrade (Perkins, 1999).

In accordance with US Army Corps (2003) and Zornberg (2011) about the relevance of the various mechanisms, each mechanism requires different magnitudes of deformation in the pavement system to be mobilized. In the case of unpaved roads, significant rutting depths (in excess of 25 mm) may be tolerable. For paved roads, the increased bearing capacity and tensioned membrane support mechanisms can be considered. However, the deformation level to mobilize these mechanisms generally exceeds the serviceability requirements of flexible pavements. Thus, for the case of flexible pavements, lateral restraint is considered to contribute mostly for the improving the performance of geosynthetic-reinforced pavements.

Austin and Gilchrist (1996) suggest that the stiffness added by the geogrid to the asphalt concrete contributes to the lateral restraining effect as reinforcement. According to this study, pavements reinforced with either a geogrid or a geogrid composite have shown considerably improved performance regarding surface deformation. Reduction in rutting was found to be as high as 70%. Also, a comparison between unreinforced and reinforced slabs have shown that the reinforcement provided significant resistance in the lateral flow and, hence, the buildup of permanent deformation.

Pavement rutting resistance has also been considered to improve with geogrids and geogrid composites in asphalt overlays by the increase of stiffness in the asphalt layer (Brown et al., 1985; Montestruque et al., 2004; Laurinavičius and Oginskas, 2006; Buhler 2007; Solaimanian 2013; Ferrotti et al., 2012; Canestrari et al., 2013). Brown et al. (1985) conducted a series of studies to look at the effectiveness of a polypropylene geogrid in improving the rutting resistance of pavements using an accelerated testing facility. In this research, the study concluded the inclusion of a geosynthetic in the asphalt layer have reduced the rut depth by 20 to 58%. Montestruque et al. (2004) have showed results of plastic deformation in geogrid-reinforced asphalt beams with reduction between 30 and 36%. Solaimanian (2013) has reported up to 86% reduction in permanent deformation of experimental setup with a layer of geogrid composite at the interface of the concrete slab with the asphalt layer in comparison to an

unreinforced slab. An experimental study conducted by Laurinavičius and Oginskas, 2006 with geogrids reported 50% less rutting. Buhler (2007) presented an experimental study with 1.4 times less rutting in the experimental lane reinforced with geogrid.

Some others laboratory and field experiments studies have been conducted to assess the impact of geosynthetics reinforcements within asphalt overlays to increase the overall performance of pavements (Brown and Brodrick, 1985; Komatsu et al., 1998; Button and Lytton, 2007). Recently, Siriwardane et al. (2010) presented the results of a flexible pavement sections (with and without fiberglass geogrid) as a part of the laboratory-scale experimental study. Vertical displacements have decreased with the inclusion of the geogrid within the asphalt layer with an improvement of approximately 38%. Observations showed that the inclusion of a geosynthetic reinforcement spreads the circular load over larger area in the lower layers of the pavement section reducing vertical stresses.

In general, geosynthetics in asphalt layers have been focused to provide structural improvements to the pavement in the following situations:

- a) Because it is a reinforcing material, the elastic stiffness of asphalt layer under transient load may ultimately be increased, leading to a reduction in vertical stresses that acts in the underlying layers, which implies a reduction of rutting due to the plastic deformation under the action of repeated traffic loads;
- b) In the post-cracking stage, by introducing a stiff tensile element at the base of an asphalt overlay, the pavement performance can be improved due to the change in the cracking pattern (great length and large openings cracks to less damaging micro cracks);
- c) In the restoration of pavements, especially very cracked wearing surfaces to the point of reconstruction, the application a geogrid reinforcement and a resurfacing asphalt layer, may be the most cost effective alternative (especially when considering the inconvenience to traffic operation);
- d) In new pavements, the inclusion of a geosynthetic reinforcement in the asphalt layer within the tensioned area can lead to an increase in fatigue life of the pavement, allowing an asphalt overlay thickness reduction (major benefit for very heavy traffic);
- e) The resulting heterogeneity of construction processes is an important factor that may affect the future performance of the pavement. Thus, the inclusion of a synthetic

reinforcement may, theoretically, homogenizing the structure, improving pavement overall performance (Montestruque and Silva, 2001).

Although the use of geosynthetic in the asphalt layer has demonstrated structural benefits to the pavement, research is needed on the reinforcement mechanisms governing the behaviour of geosynthetics in asphalt overlays. Experimental results have been used to estimate the geosynthetic contribution in increasing pavement service life. Some studies suggests the geosynthetic reinforcement mechanisms of base and subgrade layers of flexible pavements may also act in the asphalt layer. Although the geosynthetic interlayer system contribution has been inconclusive (Al-Qadi et al., 2008), research conducted in this field help better understand the mechanisms of geosynthetic-reinforced asphalt overlay systems.

2.4 Laboratory and field quantification of geosynthetic-reinforced asphalt overlays

Several experimental studies have been conducted to understand the behavior of geosynthetic-reinforced asphalt overlays through fatigue testing of asphalt concrete beams or small to large scale accelerated trafficking devices, as well as field or numerical investigations. Some of these studies include the impact of geosynthetics, especially geogrids, as reinforcements within the asphalt overlays to increase the overall performance of flexible and rigid pavements.

2.4.1 Asphalt beam tests

Enhancements by geogrids on the resistance of asphalt concrete to the development of reflection cracks usually involve quantifying the effect of reinforcement on the fatigue life of an asphalt beam. The fatigue testing of asphalt beams is usually carried out by placing the beam on an elastic foundation, and loading the beam at the center. A fatigue load at a predetermined frequency is applied to each beam. The total number of fatigue loads to crack the entire depth of beam is recorded as its fatigue life (Saraf et al., 1996). The data obtained from different sets of the beams determine the effect of geogrid reinforcement on the fatigue lives of the beams. The factor of effectiveness of the reinforcement (FER) is given in the equation 1:

$$FER = \frac{N_R (geogrid\ reinforcement)}{N_{UR} (unreinforced)} \quad (1)$$

where N_r is the number of cycles to failure of reinforced asphalt concrete and N_{ur} is the number of cycles to failure of unreinforced asphalt concrete.

In selecting testing techniques and equipment, field stress conditions must be simulated as closely as possible, including applying appropriate relative movements and levels of stress and strain to model an overlay. These tests typically determine the number of load cycles required to produce a certain measured crack length.

In early 1980's, a research group at the University of Nottingham conducted a pioneering research on the potential use of high strength polymer grids in the reinforcement of asphalt for highways and other paved areas. Laboratory test work investigated the benefits of geogrids with respect to the control of surface deformation, the control of reflective cracking and the improvement of the fatigue life of the pavement. One of these works, concerning fatigue cracking analysis of geogrid-reinforced asphalt overlays was conducted by Brown et al. (1985). The apparatus shown in Figure 2.5a was used to obtain fatigue data for a typical asphalt concrete layer. A polymeric geogrid was tested at two positions in the asphalt concrete beam: 11 mm and 22 mm from the bottom. The crack initiator cut into the base was 5 mm long. Failure was arbitrarily defined when the crack reached 52 mm above the base. Figure 2.5b summarizes the results of asphalt fatigue curves, showing increase in fatigue life at any strain level for the reinforced cases, although the best performance was achieved when the geogrid was in the lower position in the beam (nearer to the point where the cracks have initiated). The data indicates a FER in fatigue life up to 10 times when a geogrid was located in the lower position.

(a)

(b)

Figure 2.5. Brown et al. (1985) laboratory tests: (a) apparatus for fatigue cracking test; (b) asphalt fatigue curves.

Lytton (1989) reported results from a series of asphalt beam fatigue tests showing that the number of cycles to reach a particular target strain within the asphalt layer is more than 10

times greater for a 3" thick reinforced beam than for a 4" thick unreinforced beam. Kim et al. (1998) described beam tests similar to those carried out at Nottingham. They investigated a number of crack inhibiting alternatives, including geogrids, which showed an improvement factor ranging from 4 and 8 (to an advanced state of cracking), depending on the asphalt mixture type involved.

In a study conducted by Chang et al. (1998), fiber glass geogrids with ultimate strength of 100kN/m and 200kN/m were used as reinforcement materials to evaluate the effect of reinforcement on the fatigue life of asphalt beams. Asphalt beam specimens were placed over two pieces of plywood with a 10 mm gap at center to simulate an existing joint or crack underneath the overlay, with the entire system placed on a rubber base representing the soil foundation. The fatigue test system, based on ASTM STP 561 (1974), is shown in Figure 2.6a. Failure was defined as the time when crack grew throughout the entire cross area of the beam. A series of testing for a beam reinforced with geogrid of 100 kN/m strength showed a FER of 1.5 to 2.5 times greater than the unreinforced beam for fatigue life. For a beam reinforced with 200 kN/m geogrid, the FER was 5 to 9 times greater. Figure 2.6b indicates that the use of geogrid have significantly enhanced the resistance of asphalt concrete to the formation and development of reflection cracks. Similar observations were reported by Saraf et al. (1996).

Figure 2.6. Chang et al. (1998) laboratory tests: (a) fatigue test system for asphalt beam; (b) asphalt fatigue curves.

Brown et al. (2001) have reported fatigue tests to analyze a geosynthetic interlayer material to resist crack propagation in a beam constructed of asphalt cement. A pre-crack was placed at the bottom of the beam to facilitate crack propagation. Tests were undertaken on

control samples and several geosynthetic products. Results have indicate geogrids performing significantly better than any other type of geosynthetic product. When compared with an unreinforced section of the same thickness, the geogrid was able to reduce the rate of crack propagation by a factor of 2 to 3 times.

Similar results were found recently by Delbono et al. (2012) on a research with specimens subjected to flexure strength by repeated loads (Figure 2.7a). Asphalt slabs were formed as follows: (1) concrete + hot mix asphalt layer (HMA); (2) concrete + polypropylene (PP) geogrid composite + HMA; (3) concrete + asphalt sand + polyester (PET) geogrid + HMA, and (4) concrete + asphalt sand + HMA. The time until crack apparition in lower fiber was measured on both comparisons, as well as the time until crack reflected in the tip surface. The geogrid and geogrid composite had similar behavior in delaying the cracks development rate, as evidenced by slopes of the Figure 2.7b. The non-reinforced samples had bigger slopes, which means the spreading rate of the crack was higher. The geogrid composite system was able to reduce the rate of crack propagation by a factor of 10 times and the geogrid only in 2 times.

Figure 2.7. Delbono et al. (2012) laboratory tests: (a) equipment for repeated load flexure and (b) curves of crack evolution.

In Brazil, studies were conducted by Montestruque et al. (2004) based on quantitative and qualitative analysis of fatigue tests in asphalt concrete beams with and without polyester geogrid reinforcements. Tests were conducted to simulate a cracked pavement after rehabilitation, with the load applied at the two critical positions: on bending and on shearing, with a pre-crack with an opening of 3, 6 and 9 mm. The geogrid was positioned exactly over the extremity of a pre-crack, with an elastic base as a support. The criterion for the end of the

test was considered when the first visible crack appeared on the surface. The effective reinforcement factor ($FER = N_f \text{ (with geogrid)} / N_f \text{ (without geogrid)}$) represents the beneficial effect of the geogrid calculated as:

$$N_f = \frac{1}{c_f}, c_f \text{ is the total fatigue given for: } c_f = \frac{1}{N_{f(B)}} + \frac{2}{N_{f(S)}} \quad (2)$$

where $N_{f(B)}$ represents the fatigue life of the beam with the load in the bend mode and $N_{f(S)}$ the fatigue life in the shear mode. For 3 mm pre-crack opening, tests have demonstrated FER values of 6.14, while 6 and 9 mm pre-crack openings with FER up to 4.6 and 5.11, respectively. After load cycles in the unreinforced beam, micro cracks developed and became more visible, interconnecting to each other, leading to the formation of new cracks of less severity spread over a greater volume of asphalt concrete (Figure 2.8a). In the reinforced beams (Figure 2.8b), the growth of the crack was interrupted and a quite different pattern of cracking was observed: micro-cracks have initiated, associated with asphalt fatigue, but without any clear relation to a reflective cracking mechanism. The plastic deformation in geogrid-reinforced beams was reduced between 30 and 36% (Figure 2.9), with smaller movements of the pre-crack and the reflection crack opening in comparison to beams without reinforcement.

Figure 2.8. Montestruque et al. (2004) fatigue tests results – shear mode: (a) unreinforced beam (b) geogrid-reinforced beam.

Figure 2.9. Montestruque et al. (2004) plastic deformation results – pre-crack opening 3mm: (a) bend mode; (b) shear mode.

Caltabiano (1990) have carried out a series of beam tests to assess the performance of interlayers delaying cracks propagation through asphaltic concrete overlays. The beam test used a servo-hydraulic device shown in Figure 2.10. The testing program included series: A, B, and C, with thicknesses of 100 mm (A), 75mm (B and C), and maximum applied traffic load pressure of 810kPa (A and B) and 555kPa (C). The interlayers included polymer modified asphalt binder, geotextile interlayer, geogrid interlayer and a standard asphalt concrete beam (control). The slab test simulated traffic loading by passing a moving wheel of variable load and speed over a slab of asphalt concrete beam compacted on a timber base. The study showed although there was debonding between overlay and timber bases (existing pavement), the polymer modified binder, nonwoven geotextiles, and geogrid interlayers had given 2.5, 5.0 and 10 times increase in life, respectively, in comparison with control beam.

Figure 2.10. Caltabiano (1990) beam test arrangement.

Sanders (2001) have carried out laboratory tests to evaluate the effectiveness of geogrids and nonwoven geotextiles on cracking and rutting control. Similar beams to those used by Caltabiano (1990) were used, but placed over a rubber foundation. A 10 mm pre-crack was cut in the base of the asphalt beam. The arrangement is shown in Figure 2.11a. Extensive series of tests were carried out at 20°C and 5Hz, on roller-compacted beams containing various reinforcement types. Figure 2.11b gives the results in terms of the rate of crack growth for each specimen type tested. Results show that steel geogrid performed well, with a FER up to 3 over the unreinforced specimens. The polymeric geogrid was almost effective as the steel geogrid, while the fiberglass geogrid performed slightly inferior. It was noticed that a reduction in crack propagation rate due to the presence of the reinforcement appeared even before the crack reached the level of the geogrid. It was suggested that this may be due to the geogrid effectively preventing localized permanent deformation.

Figure 2.11. Sanders (2001) test results: (a) fatigue test system; (b) results of crack height and number of load applications.

A research conducted by Bühler (2007) with asphalt beams fatigue test was conducted with pressures up to 560 kPa (25 Hz) load applied in the share mode (Figure 2.12a). In this case, failure was considered as the instant in which a crack becomes visible in the central region of the beam (deformation accelerated in the plot area – Figure 2.12b). Results showed the polyvinyl alcohol (PVA) and polyester geogrids have achieved a FER up to 5.1 when in asphalt layer, compared to high stiffness fiberglass geogrids with a FER maximum of 2.7. According to Penman and Hook (2008), results based on extensive laboratory testing and surveys conducted on many project sites show that, in general, the inclusion of a fiberglass geogrid within the asphalt overlay can reduce the rate of reflective crack propagation by a factor of 2 to 3 times.

Figure 2.12. Bühler (2007) laboratory tests: (a) fatigue test system for asphalt beam; (b) effect of reinforcement results.

Cleveland et al. (2002) conducted a research to inform the relative effectiveness of geosynthetic materials in reducing or delaying reflective cracking in HMA using the Texas Transportation Institute (TTI) overlay tester developed by Germann and Lytton (1979) shown in Figure 2.13. Six asphalt beams (three replicates) were reinforced with geosynthetic material (two fiberglass geogrid composites, two polyester geogrid composites, a fiberglass geogrid, and a polypropylene nonwoven geotextile) with the seventh as the unreinforced beam (control). Failure was defined as the condition in which a continuous crack propagated up each side of the beam and completely across the top of the sample. Figure 2.13b shows an asphalt beam tested to failure with the crack patterns drawn. Geogrid composites have superior performance, achieving a FER up to 16, while polyester composites and fiberglass geogrids had a FER of 4 and 2, respectively. The TTI overlay testers had been also successfully used by Button and Lytton (1987) and Pickett and Lytton (1983).

Figure 2.13. Cleveland et al. (2002) laboratory tests: (a) schematic diagram of compacted test beam and TTI overlay tester; (b) sample tested to failure with crack location.

Additionally, Bischoff and Toepel (2003) and Scullion and von-Holdt (2004) have shown that fiberglass geogrids may deteriorate faster under the action of traffic load, particularly when differential vertical movements of cracks walls (shear mode) is the main factor of reflective cracking. This process can be explained by the short lifetime of glass fibers when subjected to fatigue, becoming brittle after some load cycles.

A research conducted by Montestruque et al. (2012) in Brazil at the University of Sao Paulo evaluated the fatigue behavior (shear mode) of polyester and fiberglass geogrids for the use of pavement rehabilitation. The results showed a superior performance of the polyester grid in comparison to fiberglass material, which was not recommended by the authors since it becomes brittle after some load cycles. The fiberglass geogrid broke after 21,000 cycles, while polyester geogrid did not break after 160,000 cycles.

A study held by Khodaii et al. (2009) was directed at assessing the effect of geogrids reinforcing asphalt overlays with increasing joint/crack in existing pavements, using different reinforcement positions. The geogrid used was 100% polyester with tensile strength of 50 kN/m and 12% strain in machine direction. The asphalt mixture was placed on top of two discontinuous blocks (simulating an existing asphalt layer and a concrete layer) with a 10, 15 or 20 mm gap cut 2/3 the depth from the top. Four types of specimens were prepared: (I) unreinforced, (II) geogrid placed on the concrete block, (III) geogrid embedded in one third depth of asphalt concrete from bottom, and (IV) geogrid embedded in the middle of the asphalt overlay. Figure 2.14a illustrates a test specimen under load. The test was conducted until the vertical crack length reached the full specimen overlay depth (75mm). For concrete layer pavement, tests results showed FER values of 8.1 (10 mm gap/one-third), 6.9 (10 mm gap/middle) and 1.9 (10 mm gap/bottom), 6.2 (15 mm gap/one-third) and 5.4 (20 mm gap/one-third). The same behavior occurred for asphalt existing pavement, FER values of 6.8 (10 mm gap/one-third), 5.1 (10 mm gap/middle), 2.9 (10 mm gap/bottom), 4.4 (15 mm gap/one-third) and 2.4 (20 mm gap/one-third). Figure 2.14b presents an example of permanent deformation over fatigue life for overlays with concrete block base and 10 mm gap. In general, samples with reinforcement embedded in middle and one-third depth of overlay lasted longer than when embedded at bottom, while accumulating less permanent deformation.

(a)

(b)

Figure 2.14. Khodaii et al. (2009) laboratory tests: (a) test specimen under load; (b) permanent deformation over fatigue life for overlays - concrete block and 10 mm gap.

Sobhan et al. (2004) states that the reinforcement effectiveness factor gradually increased with increasing embedment factor suggesting that the geogrid is more beneficial when placed within the bottom-half of the overlay. Moreover, Kuo and Hso (2003) conducted numerical studies showing that placing the geogrid at one third depth of asphalt overlay thickness from the bottom with old concrete pavement had the minimum tensile strain above geogrid and, therefore, had a maximum fatigue life compared with the specimens with geogrid at the bottom or in the middle of the asphalt overlay. However, this study showed placing the geogrid in the middle of asphalt overlay was the best placement for retarding the reflection cracking compared with the specimens with geogrid placed at the bottom of overlay.

In Brazil, Obando (2012) conducted flexural strength tests in order to evaluate crack reflection of two prismatic beams of dense hot mix asphalt reinforced geogrid between them. An initial crack was induced by cutting the inferior part of the lower beam, and a vertical cyclic loading was applied with a metallic plate in the center of top layer. Failure was defined when vertical displacements at the surface reached 25 mm. Figure 2.15 presents asphalt beam tests and geogrid installation. Results have shown that the use of geogrids may decrease in 15 times the vertical surface displacements. Also, it was noticed that the intensity of the cracks have decreased and crack reflection to the upper layer was prevented by the geogrid inclusion.

Figure 2.15. Obando (2012) laboratory tests: (a) unreinforced beam during flexural strength test; (b) geogrid installation during compaction process; (c) geogrid-reinforced beam.

Canestrari et al. (2013) conducted tests in order to investigate the impact of grid reinforcement at the interface of asphalt layers. Flexural tests (repeated loading four-point bending) were carried out on double-layered asphalt concrete (AC) specimens, with reinforced and un-reinforced interface using a carbon fiber/ fiber-glass geogrid pre-coated with bitumen and a fiber-glass reinforced polymer geogrid (cover of a thermosetting epoxy resin - vinylester). Figure 2.16a presents the scheme of four point bending test. The geogrid have improved the permanent deformation resistance of the double-layered systems with respect to the unreinforced specimens which reached collapse before the planned test conclusion (36,000 cycles). The presence of the reinforcement have reduced the load fraction carried by the AC double-layer and therefore reducing damage accumulation of the AC mixture, as shown in Figure 2.16b.

(a)

(b)

Figure 2.16. Canestrari et al. (2013) laboratory tests: scheme of four point bending test; (b) permanent deformation behavior at 1.5 kN load level.

2.4.2 Small-scale wheel tracking tests

Small-scale trafficking devices are mainly used for laboratory evaluation of HMA layers () to investigate fatigue, reflective cracking, rutting, moisture susceptibility and stripping in asphalt layers by means of a small loaded wheel device rolling repeatedly over a prepared asphalt concrete slab. These less expensive tools, however, often do not fully simulate field conditions and can generate inaccurate results due to the unconventional loading conditions, such as the use of steel wheels or bidirectional trafficking. However, the advent of some reduced-scale accelerated trafficking devices made possible to have a ‘trade-off’ between the cost and the capability of applying realistic trafficking conditions (Chehab and Tang, 2012).

Improved performance against reflective cracking was investigated by Austin and Gilchrist (1996) using a series of asphalt slabs loaded by a moving wheel (3 kN load) over a rubber support (Figure 2.17a), developed by the Nottingham University research group (Brown and Brodrick 1981). Asphalt beams (1.0 m long x 0.2 m wide x 0.08m thick) were tack coated to a plywood support with a 10 mm pre-crack (Figure 2.17b). The installation of the geogrid or geogrid composite into the slabs had improved the crack resistance of the slab under the moving wheel load in comparison to the control slabs, which became severely cracked. For the unreinforced slab, crack growth continued rapidly until it was completely cracked through by 3,300 passes. For the geogrid-reinforced slab, the crack propagated through the slab at 1,300 passes. For the geogrid composite reinforced slab, the slab cracked through at 25000 passes. In this last case, trafficking continued to 30,000 passes, with still only a very fine crack visible in the asphalt layer and a value of 2,500 micro strain recorded. This behavior had also been observed by Brown et al. (1985), as well as the fact that even when a crack propagates through a geogrid-reinforced beam it still retains its structural integrity. According to the authors, the addition of nonwoven geotextile to the geogrid to form a composite have improved reflective cracking resistance.

Figure 2.17. Brown and Brodrick (1981) laboratory tests: (a) wheel tracking device; (b) slab configuration.

One of the goals of the research conducted by Sobhan and Tandon (2003) was to evaluate the progressive damage and the development of cracks in unreinforced and geogrid-reinforced specimens during various phases of the fatigue life using a wheel tracking device. Figure 2.18 shows typical behavior for an unreinforced and a geogrid-reinforced specimen at 0%, 20%, 40%, 60%, 80% and 100% of the fatigue life when subjected to cyclic loading amplitude of 444 N. For the unreinforced specimen, the formation of reflection cracks during the early stages and additional secondary cracks near failure stages were clearly observed, as shown in Figure 2.18a. For reinforced specimens (Figure 2.18b), it was found the presence of geogrids either restricted or diverted the cracks, noticeably delaying the propagation of reflection cracking.

Figure 2.18. Sobhan and Tandon (2003) progression of reflection cracks results: (a) unreinforced; (b) geogrid-reinforced. Note: the numbers 1, 4 and 6 correspond to 0%, 60% and 100% of fatigue life.

Gallego and Prieto (2006) conducted tests on a small-scale wheel reflective cracking device shown in Figure 2.19a. The wheel that conveys a vertical pressure of 0.65 MPa is passed 43 times/min on a hydraulic system that introduces horizontal displacements that simulate the crack opening. The fracture criterion was the emergence of a vertical step between the crack faces of 0.2 mm (relative vertical movement between edges) at a crack opening velocity of 0.6 mm/h. The trafficking device support system consists of four trapezoidal frames and two chassis. Both chassis are set 10 mm apart, simulating the joint or crack distance. Tests were run

on two asphalt slabs with and without geogrid reinforcement (Figure 2.19b). The overlay with the geogrid had 1.5 to 1.7 times better strength than the same unreinforced asphalt slab. In terms of cracking development, a single reflective crack appeared for the unreinforced specimen, while two cracks with multiple ramifications happened for reinforced specimen (Figure 2.19c), showing the same behavior observed by Montestruque et al. (2004).

Figure 2.19. Gallego and Prieto (2006) laboratory tests: (a) wheel tracking device; (b) specimen preparation; (c) crack propagation for unreinforced and geogrid-reinforced specimens.

In 2007, the same equipment was used by Prieto et al. (2007) to investigate the potential benefits of using three different geosynthetic products. Five tests were conducted on two unreinforced specimens (1 and 2), a polypropylene nonwoven geotextile (3), a 50 kN/m fiberglass geogrid composite (4), and a 100 kN/m fiberglass geogrid composite (5). Results were based on reflective cracking strength (min). Type 1 and 2 specimens have demonstrated cracks reflecting after 300 min, while types 3, 4 and 5 after 450, 600 and 500 min, respectively. Type 5 specimen was expected to possess greater crack reflection strength because of the double strength value; however, this behavior was not evidenced.

Recently, a laboratory research was carried out by Montestruque et al. (2012), aiming to study the pattern cracking propagation in a proposed interlayer system. The Laboratoire des Ponts et Chaussées (LCPC) simulator, typically used for rutting tests (Figure 2.20a) was adapted to perform the “Wheel reflective cracking test”. The composite system (Figure 20b) is

a technical and economical alternative for rehabilitation of cracked pavements. Three overlay systems were tested: (1) conventional solution (60 mm HMA), (2) 20 mm Strata Reflective Crack Relief System (highly elastic, impermeable hot mix interlayer) plus 40 mm Stone Matrix Asphalt (SMA) and (3) similar to the second but with the addition of polyester geogrid composite between Strata and SMA. A loaded wheel with a pneumatic tire is rolled back and forward on the specimen surface. The test was finished when the crack reached the top surface of both faces. The factor of efficiency was obtained as $FE = N_{(System)} / N_{(Conventional)}$, where N is the number of cycles. The $FE_{(strata/conventional)}$ presented a fatigue life increase up to 2.8, while the $FE_{(grid\ system/conventional)}$ reached values up to 18.9. The results showed three different pattern of reflective cracking. In the conventional case, there was a quick crack propagation from bottom to top. In the second case, there was delay in crack growth due to dissipation of crack energy. In the last case, crack growth was first delayed due to Strata and then interrupted due to the geogrid composite action. According to this research, the stress relief-reinforcement interlayer is a potential composite solution to reflective cracking problems.

Figure 2.20. Montestruque et al. (2012) laboratory tests: (a) wheel tracking device; (b) proposed interlayer system.

Assessments using a small-scale wheel tracking device were also studied by Delbono et al. (2012) to investigate alternatives to prevent and delay reflective cracking (Figure 2.21). Stages of cracking propagation during 10,000 cycles were registered with a load wheel of 0.7 kN. Asphalt slabs were formed as follows: (1) concrete + HMA; (2) concrete + PP geocomposite + HMA; (3) concrete + asphalt sand + PET geogrid + HMA and (4) concrete + asphalt sand + HMA. Thus, two respective pairs of structures could be compared. The systems where the slab rests consist in two mobile supports in the edges, with an ethylene vinyl acetate (EVA) rubber between them to allow vertical movement. For the first comparison, the time

until crack apparition lead to a FER of 2.5, while for the second comparison, the test did not yield enough evidence to quantify a difference behavior.

Figure 2.21. Delbono et al. (2012) test study: (a) wheel tracking device; (b) slab configuration.

A laboratory simulation of load-induced reflective cracking was carried out by Yu et al. (2013) using the Hamburg wheel tracking device (Figure 2.22a) to study the development of reflective cracking under repetitive dynamic wheel load. The test ran at loading rate of 52 cycles/min with a load pressure of 0.7 MPa. The criterion for damage was set as the occurrence of cracks at the bottom of surface layer for all the specimens (Figure 2.22b). The shortcoming of this approach was the subjective results (visual observation) and manually recorded number of load repetitions to damage. A fiberglass-polyester interlayer (Figure 2.22b) showed a mean value of 39000 cycles to retard reflective cracking, while the control specimen had crack reflected after 7400 cycles. According to the authors, the fiberglass-polyester has a high resistance to tensile stress and a low elongation rate, revealing that tensile stress and strain were dominant in the reinforced specimens, with a FER of 5.3.

Figure 2.22. Yu et al. (2013) laboratory test: (a) Hamburg wheel tracking device; (b) schematic diagram of the test; (c) fiberglass polyester interlayer.

Recently, a one-third scale model mobile load simulator (MMLS3) testing equipment located at Penn State's Civil Infrastructure Testing and Evaluation Laboratory (CITEL) was used in a research conducted by Solaimanian (2013) to investigate the asphalt overlay for signs of distress and cracking. The MMLS3, shown in Figure 2.23a, is a wheel tracking device that applies unidirectional trafficking to the pavement in a controlled laboratory environment or on full-scale pavements in the field. The tests, included a control setup and an experimental setup with a layer of geogrid composite at the interface of the concrete slab with the asphalt layer (Figure 2.23b). The control setup exhibited top-down cracking at the vicinity of the joint (Figure 2.23c) after experiencing 20,000 trafficking cycles, while the experimental setup with geogrid showed top-down cracking at similar position after 150,000 cycles (Figure 2.23d). Crack width for both setups was almost the same (approximately 0.3 to 0.4 mm). The geogrid composite have significantly improved the top-down cracking resistance of the asphalt overlay. In terms of plastic deformation, the control setup have exhibited a rutting average of 18.6 mm for 400,000 trafficking cycles, while the geogrid setup have showed 2.45 mm rutting for the same trafficking cycle (Figure 2.23d).

Figure 2.23. Solaimanian (2013) laboratory test: (a) MMLS3 wheel tracking device; (b) experimental setup; (c) control setup results; (d) geogrid composite setup results.

2.4.3 Accelerated pavement testing and field investigations

Large scale tests can be carried out in different ways: (1) building a pavement and monitoring under real load traffic; (2) building trial sections as part of a new road or as part of a maintenance treatment and monitoring performance under real load traffic; (3) building trial sections in the field and monitoring performance under accelerated traffic and (4) building trial sections in the laboratory for use with accelerated loading. The last two options have been intensively embraced due to time-saving, preventing wastage of resources (Sanders, 2001). According to Graziani et al. (2014), an improvement in the existing design and testing approaches of geosynthetic-reinforced asphalt overlays can be obtained considering the actual response of the geosynthetic-reinforced system under vehicular loads.

Brown et al. (1985) had conducted a series of studies to look at the effectiveness of a polypropylene geogrid in improving the rutting resistance of pavements using an accelerated testing facility developed at the University of Nottingham (Brown and Brodrick, 1981). The large scale test facility (Figure 2.24) has a wheel movement controlled by the hydraulic motor designed to apply a load magnitude of up to 12 kN and maximum speed of 14.5 km/hr. In this research, the study concluded the inclusion of a geosynthetic in the asphalt layer have reduced the rut depth by 20 to 58%.

Figure 2.24. Pavement test facility (a) accelerated trafficking facility (Brown and Brodrick, 1981); (b) photograph of the equipment (Moses, 2011).

Austin and Gilchrist (1996) also conducted tests in the same accelerated trafficking facility (Brown and Brodrick, 1981) to compare the performance of reinforced pavements with a control pavement with respect to the control of surface deformation. The pavement was tested with the wheel running in single track mode at a wheel load of 6 kN and contact pressure of approximately 300 kPa. Testing was carried out on slabs made from various asphalt mixes and

loaded on a very stiff base to eliminate the effects of the lower pavement. Figure 2.25a presents the pavement section layout. According to this research, pavements reinforced with either a geogrid or a geogrid composite have showed considerably improved performance regarding surface deformation (Figure 2.25b). The reduction in rutting was found to be as high as 70%. Also, a comparison between unreinforced and reinforced slabs has showed that the reinforcement provided significant resistance in the lateral flow and, hence, the buildup of permanent deformation (Figure 2.25c). Dondi (1997) also reported that the geosynthetic interlayer reduces the displacements and rut depths under dynamic loading.

Figure 2.25. Austin and Gilchrist (1996) laboratory tests: (a) pavement layout; (b) average surface deformation; (c) transverse rut profile.

Sanders (2001) also reports results using the testing facility described by Brown and Brodrick, (1981). The pavement consisted of an 80 mm asphalt layer placed onto a layer of 600 mm by 600 mm paving flags over sand bedding, granular sub-base and a low-stiffness clay subgrade. The layer of flags was intended to represent a severely cracked existing pavement. Six different sections were constructed as shown in Figure 2.26a. Geosynthetics were placed at mid-depth in the asphalt layer. The wheel was loaded in two different paths, each of which traversed two reinforced sections and a control section. The paving flags ensured that each reinforced section included at least two transverse joints. A longitudinal joint was set at an offset of 100 mm from the center of the wheel path. The development of surface cracking was monitored throughout the test, as was the relative vertical movement between adjacent paving flags, at locations immediately adjacent to the wheel path. The results are plotted in Figure 2.26b as the number of wheel passes before the first appearance of reflective cracking on the surface versus the measured relative vertical movement between flags. It indicates that there was a fundamental difference between reinforced and unreinforced cases, and the pavement

lifetime have enhanced at least 2 times. Furthermore, this improvement was achieved with the geogrid placed approximately at the neutral axis of the asphalt layer, implying that the mechanism may have been the local inhibiting of crack formation rather than conventional reinforcement.

Figure 2.26. Sanders (2001) laboratory tests: (a) pavement test sections; (b) relative vertical deflection versus wheel passes before the first appearance of reflective cracking.

A research conducted by Grabowski and Pozarycki (2008) investigated the fatigue crack propagation of large dimension asphalt pavement samples reinforced with geogrid and geogrid composite. The large dimension samples were cut in 1.00 m width from the test section of the asphalt pavement structure (Figure 2.27a). The asphalt pavement structure consisted of the following layers: 7 cm base course, 6 cm binder course and 5 cm wearing course. The reinforcement was installed between binder and wearing courses. AC layers were laid on the mechanically stabilized base course 20 cm thick (Figure 2.27b). Laboratory tests had been carried out in terms of multiple repeated loading and constant temperature. The evaluation of fatigue crack propagation was based on the analysis of the reinforcement efficiency index expressed by a function of the load cycles number and energy absorbed in the volume of the tested samples. Significant increase of energy absorbing capacity in samples reinforced with both geogrid and geogrid composite had been noticed when compared to the unreinforced samples. The investigation for the reinforced sample with geogrid resulted in significantly smaller fatigue crack propagation resistance against the sample reinforced with geogrid composite (Figure 2.27b).

(a)

(b)

Figure 2.27. Grabowski and Pozarycki (2008) test results: (a) testing arrangement for fatigue crack propagation; (b) test results.

Wasage et al. (2004) have reported rutting resistance evaluation of geosynthetics reinforced asphalt pavement by means of laboratory large scale tests. The tests have measured the surface rut depth and base deformation of pavement sections when loaded by a moving wheel simulating a truck wheel load. Test specimens were fabricated consisting of 300 mm residual soil (CBR value of 1–2%), 300 mm granular aggregates base course and 50 mm of asphalt mix (porosity of 20%). The moving wheel covered a distance of about 620 mm in a reciprocating motion with a wheel pressure of 700 kPa and a wheel speed of 0.3 km/h. A geogrid duplicate test (referred to as ST) and two nonwoven geotextiles (TS-1 and TS-2) were placed at the interface of the base and surface courses. Figure 2.28a compares rut depth results of control specimens and specimens with geogrid, while Figure 2.28b presents the transverse surface profiles for all the specimen types tested. It is evident that the reinforcement function of the geogrid was effective in arresting rut depth development as well as resistance in the lateral flow. For both the longitudinal and the transverse surface profiles, the control and the TS-1 specimens showed the highest deformations, whereas geogrid specimens (ST) showed the least deformations. Fuji et al. (1993) conducted field tests and concluded that thick asphalt

overlay reinforced with geogrid did also decrease surface deflections, and two layers of geogrid are even more effective than a single layer.

Figure 2.28. Wasage et al. (2004) test results: (a) surface deformation; (b) transverse rut profile.

Siriwardane et al. (2010) presents the results of an investigation on the effectiveness of geogrids as a reinforcements of asphalt layers in flexible pavements. Flexible pavement sections (with and without fiberglass geogrids) were constructed and tested in the laboratory as a part of the laboratory-scale experimental study. Sections were instrumented with pressure cells, displacement gages, and strain gages. Test sections were subjected to 1,000,000 load applications at a frequency of 1.2 Hz. Laboratory test section consisted of hot mix asphalt layer, granular base layer, and subgrade soil (Figure 2.29a). Figure 2.29b shows the cumulative displacements for reinforced and unreinforced thick asphalt sections (152 mm). Vertical displacements have decreased with the inclusion of the geogrid within the asphalt layer with an improvement of approximately 38%. Observations showed the inclusion of a geosynthetic reinforcement spreads the circular load over larger area in the lower layers of the pavement section reducing vertical stresses.

Regarding field quantification of pavement performance, Herbst et al. (1993) have presented an interesting set of data from an experimental research in Austria, where the comparative benefits of geogrids and nonwoven geotextiles could be directly assessed. The section with 60 mm of asphalt over a polypropylene geogrid on a 30 mm asphalt leveling course showed no cracking after nearly three years, while 7% of the cracks on the equivalent geotextile reinforced section had appeared on the surface. However, where the geogrid was used without the 30mm leveling course, 6% of cracks had appeared. A corresponding section with steel geogrid (also without leveling course) allowed 27% of cracks to appear, and even more where

no prior sealing treatment had been carried out. Huhnholz (1996) also presented direct evidence that a polymer geogrid gave a life enhancement factor of at least 3. On the A23 road in Germany, a severely cracked and rutted pavement was planned out to a depth of 100 mm and the geogrid fixed prior to 100 mm of new asphalt being laid. After 7 years, no cracking had been reported where a previous similar unreinforced treatment had developed reflective cracking within two years.

Figure 2.29. Siriwardane et al. (2010) laboratory test: (a) pavement structure layout; (b) cumulative displacements.

Darling and Wooltencroft (2000) have evaluated the performance of a fiberglass geogrid in delaying reflective cracking. Performance of the test sections was monitored during 4 years using visual surveys. For the geogrid sections, 18.3% of the transverse cracks have reflected to the surface after four years of service. In the longitudinal direction, 1.82% of the cracks have reflected to the surface, compared to 74.3% in the control sections. In the second section, 42.9% of the transverse cracks reflected to the surface in the control section after 4 years of service compared to 18.8% in the sections treated with geogrid. In the longitudinal direction, 14.2% of the cracks reflected in the control section compared to 1.4% in the reinforced section. Ellis et al. (2002) also conducted trials at a number of military airfields in the UK to assess the performance of different anti-reflection cracking systems, including geogrid reinforcements. The performances of test sections have been compared to control areas constructed with conventional asphalt overlay. After 7 years in service, fewer reflection cracks had occurred in test sections incorporating a polyester geogrid or a fiberglass geogrid under 100 mm of asphalt overlay, yet an associated control had extensively cracked.

In Brazil, a research conducted by Fritzen (2005), evaluated a rehabilitated conventional overlay in comparison with a reinforcing polyester geogrid composite using a heavy vehicle traffic simulator shown in Figure 2.30. The total overlay thickness was 70 mm (20 mm stress relief + geogrid + 50 mm HMA). In the conventional section, cracks came out after 19200 cycles, propagating in the entire thickness of the asphalt overlay. In the section with geogrid (Figure 2.30b), cracks had only appeared at the surface after 220,500 cycles. Although there was a short difference on cracking propagation time, extracted specimens have revealed no cracking propagation above the geogrid. Top-down cracks (Figure 2.30c) were attributed to a possible oxidation of the HMA.

Figure 2.30. Fritzen (2005) field tests: (a) HVS traffic simulator; (b) geogrid composite application; (c) top-down cracks.

Laurinavičius and Oginskas (2006) conducted an experimental research on geosynthetic-reinforced asphalt overlays in order to identify the impact of reinforcement on the rheological characteristics and strains of the AC layer. A layer of asphalt concrete was reinforced with geogrids (3 types) laid between the first and the second layer of asphalt concrete (Figure 2.31). The measurements have been based on the plate bearing test. After 5 months, the rut depths have increased from 1.0 to 1.4 mm with geosynthetic materials and 3 mm without reinforcement. It was shown that rutting depth depends on modulus of elasticity of asphalt concrete, which depends on the type of geosynthetic material used. Another field research was taken in Brazil by Bühler (2007) to confront geogrid-reinforced asphalt overlays with a control section in order to understand rutting resistance. The FER was assumed as 10 mm rutting. The benefit achieved by the geogrid section was 1.41 higher in comparison with the control section.

Figure 2.31. Laurinavičius and Oginskas (2006) experimental test: (a) pavement layout; (b) experimental section.

Penman and Hook (2008) describes how fiberglass geogrids and geogrid composites have successfully been used as interlayers over the last 25 years in order to delay the propagation of reflective cracks in asphalt pavements. According to the authors, there are more than 100 examples of projects where this type of product has been used to reinforce overlays for airfield runways, taxiways and aprons. Based on extensive surveys conducted on many project sites, it has been shown that the inclusion of a geogrid within a pavement overlay will reduce the rate of reflective cracking propagation by a factor of 2 to 3 times. This leads to an extension of the pavement service life and reduced after-care maintenance costs. In a typical airport pavement, these benefits are likely to yield life cycle cost saving of 20 and 30%.

Correia et al. (2013) have reported a successful Brazilian case with the use of a geogrid composite on a wearing surface of a timber bridge deck. In 2006, four bridges were built for vehicular access within the Campus of the University of Sao Paulo at Sao Carlos. These bridges were constructed and studied as part of an Emergency Timber Bridges Program for the State of Sao Paulo (Calil Jr. 2007). Asphalt surface was used in all bridges. Two of these bridges were built as mixed wood-concrete deck bridges with round wood beams and concrete deck. A third structure was built as pre-stressed box-beam bridge with composed plywood beams. In the fourth bridge (prestressed box-beam bridge, with glued-laminated timber beams), a geogrid composite was installed during asphalt surface construction (30 mm HMA + geogrid composite + 20 mm of stress absorbing membrane interlayer-SAMI) (Figure 2.32a). A field survey was conducted 6 years later, demonstrating no cracking problems at the bridge where the geogrid was installed (Figure 2.32b). Also interesting, longitudinal cracks and cracks on the bridge approach were also not evidenced, differently from the other three bridges (Figure 2.32c). The study have demonstrated potential benefits of including a geosynthetic on both minimizing

cracks appearance on the deck surface of timber bridges and as a reinforcement element on the bridge approach embankment, reducing vertical displacements.

Figure 2.32. Correia et al. (2013) field study: (a) geogrid installation; (b) reinforced wearing surface without cracks; (c) crack in the bridge approach of unreinforced surface.

2.5 Stress-strain response of geogrid reinforcement in pavement layers

As reported by Tang et al. (2013), direct measurements of the strains developed in the geogrid provide both means of examining the effectiveness of the geogrid in reinforcing the flexible pavement and a unique opportunity to investigate geogrid tensile behavior under cyclic moving traffic loading. The effectiveness of a geogrid in reinforcing a pavement structure is not well understood. One of the questions that remain is the extent to which a geogrid is engaged and mobilized throughout its service use.

According to Perkins (1999), a prerequisite to analyzing the reinforcement mechanisms in geosynthetic reinforcing pavement layers is the development of a study on the strain distribution in the geosynthetic. Haas et al. (1988) and Miura et al. (1990) presented data demonstrating such trends for paved roadways using geogrid reinforcement while Fannin and Sigurdsson (1996) showed the same for an unpaved road using a geotextile. Haas et al. (1988) also showed that vertical stress on the subgrade was less when reinforcement was present. While these studies have provided data that aids in illustrating the mechanisms described above, additional data is needed to understand the mechanisms by which geosynthetic reinforce flexible pavements. This information is needed to proceed with the development of numerical models that predict reinforced pavement response.

Perkins (1999) conducted large-scale experiments on reinforced and unreinforced pavement sections using large-scale experiments, as illustrates Figure 2.33a. Two biaxial geogrids and one woven geotextile were used in the test sections. Geosynthetic strains were measured using bonded resistance foil strain gauges glued to the ribs of the geogrid and glued

to the geotextile with a heavy epoxy. Foil strain gauges were placed to measure radial and circumferential strain at a number of radius from the load-plate centerline. Figure 2.33b illustrates a profile of permanent geosynthetic radial strain with radial distance from load for 7500 load cycles. These results illustrate a pattern of strain development in the area beneath the load plate where the greatest lateral movement of the base occurs. An evidence of lateral base restraint is the development of significant geosynthetic tensile strain. Figure 2.33c shows that permanent geosynthetic strain accumulates under relatively constant dynamic strain for each load cycle. Constant dynamic geosynthetic strain implies that the dynamic load induced in the geosynthetics is also nearly constant. The dynamic strains suggest that the dynamic load induced in the geosynthetics is significant. Examination of permanent and dynamic strain for the first load cycle shows that the reinforcement is mobilized immediately upon load application.

Figure 2.33. Perkins (1999) test: (a) schematic experimental layout; (b) permanent strain in the geosynthetic versus distance from the centerline; (c) dynamic radial strain in the geosynthetic versus load cycle number.

A full-scale dynamic loading was adopted to study geosynthetic-reinforced asphalt pavement by Ling and Liu (2001). A geogrid was installed in the bottom of the AC layer, along the asphalt-subgrade interface, to function as tensile reinforcement. The load was applied to the surface of the AC layer using a rigid rectangular footing under plane strain conditions. Figure 2.34 illustrates the experimental setup. The strains along the machine direction of the geogrid specimen were monitored at five different locations using strain gauges, including one below the center of the footing and the rest about 6-cm apart from each other, and their restraining effects on the layered system were compared. Two different types of reinforcements were used, a polypropylene geogrid (A) and a less stiff polyester geogrid (B). Five cycles of cyclic loading (unloading and reloading) were applied with three stress levels of 218 kPa, 436 and 872 kPa. The strain distributions along the geogrid, at the start and end of each loading step, are shown in Figure 2.35a, as the maximum and minimum values. Dynamic loading tests with the load applied at a frequency of 2 Hz were also conducted having load amplitudes of 50, 100, 150, and 180 kPa. Figure 2.35b shows the strains recorded in the geogrids over time at different stress levels. The restraining effect of geogrid was reflected by the strains developed in the geogrid around the vicinity of the loading area. The settlement over the loading area of reinforced pavement was reduced when compared with unreinforced pavement. The improvement was more significant for dynamic loading. Results showed that polyester geogrid allowed smaller strains to develop compared with the polypropylene geogrid. According to the authors, the results can be attributed to a more effective interlocking between polypropylene geogrid and asphalt material, allowing load transfer.

Figure 2.34. Ling and Liu (2001) test: (a) experimental setup; (b) strain gauges details.

Figure 2.35. Geogrid strain measurements during Ling and Liu (2001) tests: (a) distribution of strains along geogrid – cyclic mode; (b) Strains in Geogrid – dynamic mode.

Chen et al. (2009) conducted a study to investigate the strain distribution along the geogrid as base reinforcement in pavement on weak subgrade under cyclic plate load testing. Two different geogrids, GG1 (biaxial) and GG2 (triaxial) were used. The tests were conducted inside a test box of dimensions of $2.0 \times 2.0 \times 1.7 \text{ m}^3$ using a servo-hydraulic actuator. A 40-kN load at a frequency of 0.77 Hz was applied through a 305-mm-diameter steel plate. Foil strain gages were used to measure strains along the centerline of the geogrids for different load cycles. Figure 2.36 presents the results. The triaxial geogrid GG2 performed similar to the biaxial geogrid GG1. The data show that significant permanent tensile strains were developed in geogrids, which are believed to restrain the lateral movement of the base course aggregate. The measured tensile strain was greatest at the point beneath the center of the loading plate, where greatest lateral movements of the base courses were expected to occur. Bondable foil strain gages, which experience both accumulation of plastic strains and relatively high dynamic strain level were not suitable for continuous monitoring of strain development for geogrid in cyclic loading tests with high numbers of loading cycles.

Figure 2.36. Chen et al. (2009) results of permanent strain distributed along the centerline of the geogrid results: (a) GG1 geogrid; (b) GG2 geogrid.

Tang et al. (2013) presents results of a laboratory study on various geogrid products embedded in laboratory pavement sections subjected to cyclic moving wheel loads by using the one-third scale model mobile load simulator - MMLS3 (Figure 2.23a). Three sections were reinforced by the different geogrids placed at the base–subgrade interface, while, as a control, one section was left unreinforced. Inflation pressure was set of 690 kPa up to 10^5 load repetitions. During the APT, strains that developed in the geogrids were measured at intervals of loading applications by strain gauges installed in pairs on the upper and lower surfaces of the geogrid ribs. Permanent and dynamic responses to moving wheel load were recorded. According to the results presented in Figure 2.37a, considerable strains were developed in geogrid ribs in the direction transverse to traffic loading. It was observed that geogrid ribs in the longitudinal direction of traffic loading were not mobilized. The tensile forces developed in the geogrids (Figure 2.37b) were derived from measured strains and the tensile capacity at small displacements in pullout tests. The tensile forces mobilized in the geogrids showed a clear correlation with the reinforced pavement performance in resisting the subgrade deformation (Figure 2.37c). Figure 2.38a shows a snapshot of the measurements from strain gauges on a geogrid in response to the moving wheel load over a period of 2 s, during which all four wheels passed the strain gauge location once. Figure 2.38b presents the accumulated strain measured from gauges attached on the upper and lower surfaces of the geogrid rib in transverse direction (TD), indicating that the geogrid rib's flexural deflection has significant effects on tensile strain measurements.

Figure 2.37. Tang et al. (2013) laboratory results: (a) geogrid strain in the transverse direction; (b) geogrid tensile force in transverse direction; (c) accumulation of permanent deformation of subgrade.

Figure 2.38. Tang et al. (2013) results on the effects of flexural deflection of ribs on strain measurements for Grid C: (a) dynamic responses to moving wheel load and (b) permanent strains in transverse direction.

While some these studies have provided data that aids in illustrating the reinforcement mechanisms taking place in aggregate base layers, additional data is needed to understand the mechanisms by which geosynthetics reinforce asphalt overlays. Nguyen et al. (2013) have presented a solution of instrumentation of a glass fiber grid by strain gages to monitor their performance in the asphalt layer of a new pavement. A piece of glass fiber grid ($1\text{m} \times 1\text{m}$) was instrumented with two longitudinal and two transverse strain gages, glued directly on the mesh of the grid, positioned at the under the center of dual and one wheel. The strain gages used are Kyowa KFG-5-350-C1-11, 10mm length. This grid was placed on top of a thin layer of

bituminous mix, 1 cm thick and covered by tack coat, and 7 cm of dense-graded asphalt mixture was laid. The pavement structures were built on the existing subgrade of the test track, composed by sand with 10% of fines and a granular base of 30 cm. The full-scale experiment was carried out on the pavement fatigue carrousel of IFSTTAR in Nantes, France. Figure 2.39 presents the shape of the signals for 53,000 loads. Figure 2.40a presents the gages installed on the geogrid and maximums of longitudinal strains for two wheels position, for the two strain gages glued on the grid and for a classical pavement strain gage, placed in the bituminous layer, at the same level. The geogrid did not seem to reduce the level of longitudinal strain, compared with the response of the unreinforced pavement (Figure 2.40b). This research has demonstrated the possibility to measure strains of the fiber glass grid under traffic loading, although indicating no significant strain reduction in longitudinal direction due to the grid reinforcement in a new pavement. Results of transverse strains could not be compared with those obtained with a classical gage placed in the unreinforced structure because the transversal strain gage was broken during construction.

Figure 2.39. Nguyen et al. (2013) typical results of longitudinal and transverse strain gages under the center of the load: (a) two wheel; (b) one wheel.

Figure 2.40. Nguyen et al. (2013) study: (a) geogrid with two longitudinal and two transverse strain gages; (b) maximum longitudinal strains in the structure and on the geogrid.

Graziani et al. (2014) have recently conducted a study on the structural response of geogrid-reinforced bituminous pavement based on stress-strain measurements carried out using pressure cells and asphalt strain gauges (ASGs) installed inside instrumented pavement sections. Pavement section was constructed to obtain actual response of geogrid-reinforced systems under vehicular loads by installing a fiber glass polymeric geogrid (FP) and a carbon fiber/ fiber glass geogrid (CF) inside a double-layered asphalt surfacing along an in-service road. CF geogrid has an ultimate tensile strength of 111 kN/m and elongation at rupture of 3-4.5%, while FP has a tensile strength of 211 kN/m and elongation at rupture of 3%, both on longitudinal direction. The pavement response to falling weight deflectometer (FWD) and real-scale truck loads was measured. Before construction of the instrumented pavement section the existing asphalt surfacing (90 mm) was milled. The new double-layered asphalt surfacing was composed of a 40 mm thick lower layer and a 50 mm thick upper layer (Figure 2.41a). The same asphalt concrete mix was used for both layers. SBS polymer-modified tack coat emulsion was applied on the surface of the lower layer with a rate of 0.25 kg/m² of residual binder. Figure 2.41b illustrates the instrumentation and geogrid installation.

Figure 2.41. Graziani et al. (2014) study: (a) instrumented area plain view and cross section; (b) ASG installation and paving on CF geogrid.

The FWD results of Graziani et al. (2014) results have shown an overall 65% reduction in average peak strains inside the CF section with respect to the unreinforced section. Controversially, the FP section did not show any reduction compared to the unreinforced case. According to real-scale tests, the same behavior was observed for CF section; however, the presence of the FP geogrid have caused an increase of the measured longitudinal strains (Figure 2.42). As a result, the more flexible CF geogrid showed better compatibility with the AC double-layered surfacing, reducing the overall strains compared to the unreinforced case, potentially increasing the long-term pavement performance. Particularly, the reinforcing effect was not observed in FP geogrid, mainly attributed to a debonding effect (reported by Canestrari et al. 2013), probably related to the high torsional rigidity and thickness of FP geogrid.

Figure 2.42. Graziani et al. (2014) results of measured and simulated horizontal asphalt strains: (a) unreinforced section; (b) CF section; (c) FP section.

Although some studies have been conducted on geosynthetics in asphalt layers to evaluate pavement structural response, few studies have investigated the tensile strain response in the geosynthetic and its mobilized tension when subject to traffic loading conditions. Hence, the reinforcement benefit to the pavement in this application needs to be more thoroughly investigated, as well as the associated reinforcement mechanisms. Knowledge of using geosynthetics in pavements layers has focused on the use of reinforcements for aggregate base and subgrade layers, in which mechanisms cannot be directly associated to those in reinforced asphalt layers. Thus, additional data is needed to understand the mechanisms by which geosynthetics reinforce asphalt overlays, as well as the structural benefits provided by this technique. According to Sobhan et al. (2005), more comprehensive testing and analytical modeling are needed to better predict the field performance of geogrid reinforcing asphalt overlays. Pavement stress-strain analysis is an ideal tool for analytical modelling of pavement behavior and thus, constitutes an integral part of pavement design and performance evaluation.

3 DEVELOPMENT OF AN ACCELERATED PAVEMENT TESTING FACILITY

3.1 Overview of the Accelerated Pavement Testing facility

An Accelerated Pavement Testing facility is defined by Metcalf (1996) as “*the controlled application of wheel loading to a prototype or actual, layered, structural pavement system to determine pavement response and performance under a controlled, accelerated, accumulation of damage in a compressed time period. The acceleration of damage is achieved by increased repetitions, modified loading conditions, imposed climatic conditions, or a combination of these factors. Full-scale construction by conventional plant and processes is necessary to model real world conditions.*” With the use of Accelerated Pavement Testing (APT) facilities, the effect of millions of traffic loads can be compressed into just few weeks of testing, representing the effect of several years of in-service pavement traffic. Studies using APTs can be used to provide a better understanding of pavement modeling, viability analyses of new materials, new construction techniques and also to perform economic analyses of different solutions. In this way, a variety of pavement structures can be tested.

The Accelerated Pavement Testing facility developed for this research consists of a large steel testing box (for the construction of pavement sections) and a wheel tracking device. Figure 3.1 shows the final configuration of the APT facility at the University of Sao Paulo.

Figure 3.1. Overview of the Accelerated Pavement Testing Facility.

The testing box was constructed at the Laboratory of Geosynthetics at the University of Sao Paulo in the early 2000's. The dimensions of this box are 1.8 m (length), 1.6 m (width) and 1.80 m (height). The box was built with steel reinforced walls to withstand high lateral soil stresses with minimum lateral deformations, thereby achieving plane strain state in the structure. Details of the testing box construction can be found in Viana (2003). As a reaction frame, an I-beam was fixed on the walls of the box, which completes the load transfer system of the wheel tracking device. Figure 3.2 illustrates the testing box and the wheel tracking device.

Figure 3.2. Accelerated Pavement Testing Facility: (a) testing box; (b) wheel tracking device.

3.2 Wheel tracking device

The wheel tracking device was developed by Prototypus Ltd. at Sao Carlos – SP. The equipment was designed to apply two loading type modes: stationary repeated wheel loading and cyclic moving wheel loading. The wheel tracking device consists of a load frame (type I-beam), along which a loading carriage system moves forward and backward. The free rolling single truck wheel is mounted in this loading carriage and the load is applied by means of a hydraulic jack. A load cell (AEPH - Model RPPB – 15000 kg) is attached between the loading carriage and the hydraulic jack to measure the applied load.

The loading carriage system moves in a steel sliding gate rack attached to the reaction frame guides, which guides the movements in the horizontal direction. The loading carriage is moved by an electrical motor. The wheel involves a rubber tire (Trelleborg T- 800) of 546mm diameter by 154 mm wide (inflation pressure capability of 1 MPa) mounted on a steel wheel.

Steel springs are used as a softener against any abrupt movement of the loading carriage system. Figure 3.3 illustrates a wheel tracking device schematic view and its components.

Figure 3.3. Wheel tracking device schematic view and components.

A pavement section had to be constructed in the testing box in order to first test the wheel tracking device. However, before its final version was ready, the wheel tracking device had undergone several adjustments. Changes were necessary in order to obtain the equipment best possible configuration. Cross braces were installed to increase the wheel tracking device capability to withstand movements. To ensure stable movement, horizontal gate wheels were installed in the load carriage. Different types of sliding gate racks and gate wheels were tested up to the final configuration of C60 steel materials, as they require less maintenance. Figure 3.4 illustrates the final arrangement of the wheel tracking device components.

Figure 3.4. Wheel tracking device components.

3.3 Wheel loading system

The loading system is operated by a hydraulic system composed of a hydraulic pump motor, hydraulic jack, directional valves and high pressure hydraulic pipes. An electrical motor is used to position the loading carriage and to provide moving wheel loads (speed and direction movements). Figure 3.5 shows the hydraulic operational system components.

Figure 3.5. Components of the hydraulic operational system.

The hydraulic pump motor (WEG 4 CV - 120 liter oil reservoir from AMC Ltd) is responsible for fluid circulation and for keeping pressure at the hydraulic jack (piston rod of 32-mm diameter and a 150-mm stroke length from AMC Ltd.) and the electric motor (Orbit Hydraulic Motor BMP-100). The functions of the hydraulic pipes and directional valves are to transport the fluid and to regulate the applied pressure and flow control. A manometer sets the required pressure and is controlled by a manometer valve. Loads up to 21 kN can be applied in this equipment.

An in-house software for data acquisition was developed using Labview® to control the load-time history applied load to the wheel. The data acquisition system was used to collect data from the instruments installed in the pavement sections (Chapter 4). The software was programmed to induce the target load-time history during the test for both cyclic moving and stationary wheel loadings.

In this research, a 700-kPa tire contact pressure was used for the tests. Tire inflation pressure also plays an important role in the tire-pavement interaction process. Traditional methods of pavement analysis assumed that contact pressure is the same as the tire inflation pressure, uniformly distributed over a circular contact area. Therefore, in order to assure the required pressure condition, the tire inflation pressure was set to 700 kPa.

In order to correctly estimate a tire contact pressure of 700 kPa in the pavement sections, a footprint measurement was used to capture the tire contact pressure area. In addition, a pressure cell was used to verify the exact magnitude of the applied load. Measurements indicated that a 700-kPa contact tire pressure was achieved with the application of a 16-kN load. Figure 3.6 illustrates the tire inflation pressure process, a pressure cell for applied wheel load verification and the foot-print measurement.

Figure 3.6. Tire contact pressure verification.

3.3.1 Cyclic moving wheel load

In the cyclic moving wheel load mode, the load is applied in a single direction and uniformly distributed in the wheel path. The wheel is lifted off the pavement surface at the end of each load cycle and suspended on the main frame on its return. By loading in one direction, the wheel tracking is able to simulate real-world traffic conditions.

The wheel passes over the pavement test section only once during each revolution around the track. Also, it triggers micro switches (KW11-7-1 AC 250V/16A) that register the number of wheel loading applications and control the loading carriage movement (Labview® program). Figure 3.7 illustrates the moving wheel loading path and the sensors.

As shown in Figure 3.7, micro switches indicated as number 1, 4 and 6 are responsible for informing the loading carriage to return (unloaded), for stopping and moving forward (loaded), respectively. The system is also provided with two pairs of micro switches, indicated as sensor 2 and 3 and sensors 5 and 6, used to register the forward and backward speed of the loading carriage. In the case of faster or slower movements, these sensors are also programmed to correct the following revolution time. The control system was implemented because there was no oil viscosity control during the test.

Figure 3.7. Moving wheel loading path and the sensors.

When the wheel hydraulic load is applied and the load cell registers the target load, a micro switch installed in the hydraulic jack is triggered. The loading carriage then applies a constant contact pressure on the pavement surface up to its return. The path effective length of the moving wheel (at constant speed) covers a distance of 1.0 m. The wheel speed, when in contact with the asphalt surface, reaches 3.6 km/h. Figure 3.8 shows the cyclic moving wheel, the loading path and the sensors. The cyclic moving wheel load with bi-directional load application can be also programmed in this APT. Detailed instructions for the test setup are presented in Appendix A.

In the cyclic moving wheel load mode (Figure 3.8a), the wheel load pulse has a linear load increase from 0 to 16 kN over a 0.1 second rise time. Following, the load is held constant during 1.0 second. Then the load decreases to zero over a period of 0.1 second. Finally, a 1.2 second period load before the load cycle is repeated, resulting in a total of 2.4 seconds with load pulse frequency of 0.4 Hz. About 1400 wheel passes per hour can be accomplished in this loading mode. Figure 3.8b shows the corresponding input loading mode and the load cell response, and a precise match between the target and actual measured loads can be noticed. Figure 3.8c presents the applied load pressure maintenance during tests.

(a)

(b)

(c)

Figure 3.8. Cyclic moving wheel load mode: (a) input load pulse; (b) load cell response; (c) load maintenance during tests.

3.3.2 Stationary repeated wheel loading

In the stationary repeated wheel loading mode, the wheel load pulse lasts for 0.1 second and the corresponding resting time lasts for 0.9 second, resulting in a load pulse frequency of 1.0 Hz. Figure 3.9 shows the corresponding input loading mode and the load cell response, and a precise match between the target and actual measured loads can be noticed.

A stationary non-repeated wheel loading mode can also be applied in this APT facility, with a constant or increasing load. However, this function needs to be programmed. In this research, the stationary loading mode was used, and is available for future tests.

(a)

(b)

Figure 3.9. Stationary repeated wheel loading mode: (a) input load pulse; (b) load cell response.

3.4 APT automation and data acquisition system

The APT facility automation is accomplished by a programmable logic controller system (PLC) connected to the hydraulic pump and a *LabVIEW* program that controls the equipment. The data acquisition system used in this research is the System i5000 from Vishay Micro-Measurements. It is a complete data acquisition system that accepts inputs from strain gages, thermocouples, displacement sensors, load cells, and other transducers (built-in bridge completion for 120, 350, and 1000-ohm strain gages). The system offers readings up to 100 Hz. The load cell is connected to the data acquisition system.

A data acquisition board (National Instruments) sends micro switches and valves information to the data acquisition system, so it is able to perform the required loading method: cyclic moving wheel loading mode or stationary wheel loading mode. Figure 3.10 illustrates an APT automation and the data acquisition layout.

Table 3.1 summarizes the Accelerated Pavement Testing main specifications. Similar APT specifications can be found in NCHRP SYNTHESIS 433 TRB (2012).

Figure 3.10. APT automation and data acquisition layout.

Table 3.1. Accelerated Pavement Testing specifications

Parameter	Selection
Mobile/Fixed	Fixed to the testing box - Automated
Linear/nonlinear	Linear
Uni-directional/bi-directional	Both (bi-directional is programmable)
Number of axles	1
Own power	Hydraulic
Load range	0 to 21 kN for one tire
Tire details (type, size, max inflation, load capacity)	Truck (Trelleborg T-800, 546 mm diameter, 154 mm width, 1 MPa, 1885 kg). Adjustable for different tires.
Tire wander options	No options
Suspension	No suspension
Temperature control options	Room temperature
Speed range	3,6 km/h (Moving wheel loading mode). Stationary loading mode is programmable.
Dedicated operational data collection on APT device during operation?	Yes

4 EXPERIMENTAL TESTING PROGRAM

In the Experimental Program, pavement materials and the geosynthetic products used in this research are described. Pavement section construction process, instrumentation layout, testing matrix and laboratory investigations are also presented herein.

4.1 Pavement materials

The pavement materials used in this research to construct the pavement sections consist of subgrade soil, aggregate base material, asphalt concrete mix, asphalt emulsion as tack coat and a geogrid. The sections below provide the general properties of these materials.

4.1.1 Subgrade soil

The soil described as “Subgrade soil” was obtained at SP-215 Roadway (station 152+600 km) in Sao Carlos, Sao Paulo. The soil was classified as MH and A-7-5 according to the Unified Soil Classification System (USCS) and AASHTO classification system, respectively. The soil was also classified as non-lateritic (NG’) according to the MCT classification (Nogami and Villibor, 1981). The soil was transported to the laboratory and air dried before the construction of the sections and testing. Figure 4.1 illustrates the aspect of the subgrade soil.

(a)

(b)

Figure 4.1. Subgrade soil: (a) soil aspect; (b) spreading and drying process.

In order to evaluate the geotechnical properties of the subgrade material, several index tests were conducted. Figure 4.2 presents the grain size distribution curve and the subgrade material compaction curve (standard Proctor). Table 4.1 summarizes the geotechnical properties. California Bearing Ratio (CBR) tests were prepared with samples compacted at $0.95\gamma_{dmax}$ and optimum moisture content. Based on the results, the in-place CBR of 4.5% represents a relatively weak subgrade condition.

Figure 4.2. Subgrade soil: (a) particle size distribution curve; (b) compaction curve.

Table 4.1. Properties of the Subgrade soil

Test	Index Parameter	Value	Specification
Soil Classification	USCS	MH	ASTM D 2488
	AASHTO	A-7-5	AASHTO M145-87
Specific Gravity	Specific Gravity, G_s	2.9	ASTM D 854
	D_{10} , mm	-	ABNT NBR 7181
	D_{30} , mm	0.002	ABNT NBR 7181
	D_{60} , mm	0.02	ABNT NBR 7181
	Uniformity coefficient, C_u	9.6	
	Coefficient of gradation, C_c	0.10	
Particle size analysis	Liquid limit, LL (%)	63	ABNT NBR 6459
	Plastic limit, PL (%)	40	ABNT NBR 7180
	Plasticity index, PI (%)	23	
Standard Proctor Compaction	Optimum water content, w_{opt} (%)	28.8	ABNT NBR 7182
	Maximum dry unit weight, γ_{dmax} (kN/m³)	14.9	ABNT NBR 7182
	Degree of Saturation, S_r (%)	85.5	
California Bearing Ratio	CBR (%)	4.5	DER/SP-M53-71
	Expansion (%)	3.4	

The repeated load triaxial test is specified for determining the resilient modulus according to AASHTO T 307 (1999). The test was conducted on subgrade soil specimens 1 and

2 compacted at $0.95\gamma_{dmax}$ and at optimum moisture content. Figure 4.3 presents the repeated load triaxial test set up and typical resilient modulus results for deviator stress.

(a)

(b)

Figure 4.3. Repeated load triaxial: (a) test set up; (b) results for deviator stress.

The resilient modulus (MR) of fine-grained cohesive soils is significantly influenced by the deviator stress. The resilient modulus decreases with the increase of the deviator stress, as shown in Figure 4.3b. The AASHTO design guide that suggests the resilient modulus of fine-grained soils can also be estimated as a prediction based on California Bearing Ratio: $MR \text{ (MPa)} = 17.6 \text{ (CBR)}^{0.64}$ (NCHRP 1-37A, 2004). This prediction would result in a MR of approximately 50 MPa.

Table 4.2. Basic statistical data of the resilient modulus model parameters k_i obtained from the test results of subgrade soils

	Models	R^2	k_1	k_2	k_3
AASHTO	$M_R = k_1 \cdot p_a \cdot \left(\frac{\theta}{p_a}\right)^{k_2} \cdot \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3}$	0.99	6345	-0.40	-2.79
Deviatoric Stress	$M_R = k_1 \cdot \sigma_d^{k_2}$	0.97	11319	-0.88	
Composite Model	$M_R = k_1 \cdot \sigma_3^{k_2} \cdot \sigma_d^{k_3}$	0.99	17651	0.03	-1.03

4.1.2 Aggregate base

The aggregate material is a crushed stone aggregate base, meeting DER/SP-ET-DE-P00/008 (2005) specification for crushed stone course type C. It is one of the most used types of aggregate bases in Brazil. The basalt quarry is located in the city of Sao Carlos (SP). The material is classified as A-1-a according to AASHTO classification system, respectively. Figure 4.4 shows the grain size distribution curve of the aggregate base material and the compaction curve (Modified Proctor). Table 4.3 presents the laboratory test results on this material.

Figure 4.4. Aggregate base: (a) particle size distribution curve; (b) compaction curve.

Table 4.3. Properties of the aggregate base material

Test	Index Parameter	Value	Specification
Classification	AASHTO	A-1-a	AASHTO M145-87
Specific Gravity	Specific Gravity, G_s	2.6	ABNT NBR 6508
	Uniformity coefficient, C_u	46.85	
	Coefficient of gradation, C_c	4.8	
Particle size analysis	Plasticity index, PI (%)	NP	
Modified Proctor Compaction	Optimum water content, w_{opt} (%)	6.2	ABNT NBR 7182
	Maximum dry unit weight, γ_{dmax} (kN/m³)	23.9	ABNT NBR 7182
California Bearing Ratio	CBR (%)	> 80	DER/SP-M53-71
	Expansion	-	
Resilient Modulus	Resilient Modulus, M_r (MPa)	300	AASHTO T 307

4.1.3 Asphalt concrete

The hot mix asphalt material was obtained from a local plant in Sao Carlos (SP), meeting the DER/SP-ET-DE-P00/027 (2005) specification. The asphalt mix is composed of a 9.5 mm (3/8") dense-fine-graded mixture with an asphalt binder content of 5.4% (Bitumen Penetration Grade 30/45), which is indicated for all traffic conditions according to FHWA & NAPA Information Series 128 (2001). Figure 4.5 shows the grain size distribution curve for asphalt mix. Table 4.4 summarizes the properties of the asphalt binder used in the concrete asphalt mix.

Figure 4.5. Particle size distribution curve for asphalt mix.

Table 4.4. Properties of the asphalt binder type 30/45

Test	Unit	Values	Limits	Specification
Penetration	Dmm	35	30 - 45	ASTM D 5
Softening Point	°C	52.4	Min. 52	ASTM D 36
Ductility	cm	>130	Min. 60	ASTM D 113
Viscosity, Saybolt Furol 135 °C	cP	450	Min. 374	ASTM D 88
(SP21, 20 rpm)	150°C	cP	218	Min. 203
	177°C	cP	77	76 - 133
Thermal susceptibility index		-1.4	(- 1.5) – (+0.7)	ASTM D 36
Flash Point	°C	300	Min.235	ASTM D 92
RTFOT Penetration	%	63	Min. 60	ASTM D5
RTFOT Softening point	°C	3	Max 8	ASTM D 36
RTFOT Ductility	cm	>130	Min. 10	ASTM D 113
RTFOT Variation in mass	%	0.070	Max 0.5	ASTM D 2872
Solubility in Trichloroethylene, in mass	%	99.9	Min. 99.5	ASTM D 2042

4.1.4 Tack coat emulsion

Asphalt emulsion is a type of asphalt sealant usually applied during paving work repairs in order to provide adherence between layers. Asphalt emulsions have been recommended as sealant for applications involving paving geosynthetics in Brazil (DER-SP ET-DE-P00/043, 2006). These emulsions consist of asphalt cement in an emulsified water solution (63% of Bitumen Penetration Grade 30/45). A cationic aqueous rapid setting emulsion (CRS) was used in this research as the tack coat impregnation, in accordance with DER-SP ET-DE-P00/043 (2006) specifications. Table 4.5 shows the characteristics of the tack coat emulsion used.

Table 4.5. Properties of the emulsion tack coat used in this study

Test	Values	Specification
Viscosity Saybolt Furol at 50 °C (Pa.s)	21.0	ASTM D 7496
Sieve test (%)	0.1	ASTM D 6933
Identification of cationic property	Positive	ASTM D 7402
Residue by distillation (%)	63.0	ASTM D 6997
Demulsibility (%)	64.1	ASTM D 6936

4.2 Geogrid

A geogrid composite specifically designed for asphalt reinforcement was used in this research. The geogrid, manufactured from high-modulus polyvinyl alcohol (PVA), is oriented in two directions, forming a biaxial product with quadrangular apertures. An ultra-lightweight polypropylene (PP) nonwoven geotextile is attached on one side of the geogrid to facilitate the installation and to allow a continuous bonding. The material is pre-coated with bitumen in order to facilitate adherence to the asphalt layers. In this research, the geogrid composite is referred to as geogrid only, because its primary function is reinforcement. Geogrid tensile tests were conducted following the recommendations of the ASTM D 6637 (2011). Figure 4.6 presents the aspect of the geogrid and the results of the tensile strength tests performed according to machine direction (MD) and cross machine direction (CMD). Table 4.6 presents the index test results.

(a)

(b)

Figure 4.6. Geosynthetic reinforcement: (a) geogrid; (b) tensile test results.

Table 4.6. Specifications of the geogrid used in this research

Properties	Characteristics	Geogrid	
Mechanical properties of the geogrid	Tensile strength (kN/m)	at 2% of strain	MD 13.32
			CMD 17.72
		at 5% of strain	MD 36.28
			CMD 41.89
	ASTM D 6637	Ultimate	MD 40.51
			CMD 46.91
Physical properties of the geogrid	Spacing dimension (mm)		MD 40.00
			CMD 40.00
	Mass per Unit area (g/m ²)		240.0
Polymer of the geogrid	poly-vinyl alcohol		

4.3 Pavement layers construction

All test sections were constructed based on a typical flexible pavement structure with a subgrade soil, aggregate base course and an asphalt concrete layer. Figure 4.7 shows the typical pavement cross-section used in this study.

Figure 4.7. Typical pavement cross-section.

The subgrade soil had been previously air dried and was sieved in #4 sieve (4.2 mm) before compaction. Approximately 250 kg of the air-dried soil was stored in big bags in order to facilitate the soil preparation and transportation. The water content of the air-dried soil was corrected to the target moisture (Proctor tests) using a concrete mixer. The bags were moved to the testing box using a travelling crane. Figure 4.8 illustrates the subgrade soil preparation procedure used for the tests.

(a)

(b)

(c)

(d)

Figure 4.8. Subgrade layer preparation: (a) sieving process; (b) moisture correction; (c) transport to the testing box; (d) soil spreading.

The subgrade layers were constructed in the testing box in 50-mm thick lifts, compacted manually using a drop hammer. An experimental layer was initially compacted in order to define the number of compaction blows and the drop height required to reach the specified compaction parameters ($w=28.8\%$ and $\gamma_d = 14.9 \text{ kN/m}^3$). To obtain 50-mm thick lifts, three compaction passes (drop manual hammer of 15 kg and drop height of 600 mm) were found to be necessary. During subgrade construction, bulk-density measurements and oven-dried moisture content readings were taken every 200 mm. The total subgrade thickness was set of minimum 1.0 m for pavement sections. Figure 4.9 illustrates the subgrade layer construction process, and the compaction control measurements.

(a)

(b)

(c)

(d)

Figure 4.9. Subgrade layer construction: (a) compaction; (b) leveling; (c) compaction control; (d) final layer aspect.

It should be noted that the subgrade soil was completely removed after each test and reconstructed to assure the same conditions in all tests (void ratio and moisture). A nonwoven geotextile was used as separation between the subgrade soil and the aggregate base material to avoid an intermixing of between these layers. The nonwoven geotextile is a 100% polyester with cross-machine direction strength of 7 kN/m at elongation (strains) greater than 50%.

The base course material was also prepared using a concrete mixer to reach the target water content (optimum moisture of 6.2 %). The aggregate base was compacted in 100-mm lifts using a vibratory plate compactor. The layers were compacted at 95% of the modified Proctor (Figure 4.4b). The bulk-density measurements were taken at the end of the compaction process. The maximum base thickness was set to 200 mm for pavement test sections. Figure 4.10 shows the base course construction process.

(a)

(b)

(c)

(d)

Figure 4.10. Base course construction: (a) moisture correction and mixing; (b) aggregate transport; (c) spreading; (d) compaction.

The AC layer was placed in lifts corresponding to a minimum of 50 mm and compacted with a vibratory plate. The HMA was typically placed at 125°C to 130°C. The asphalt layer was allowed to cure for a minimum of a day prior to loading. The asphalt surface was painted white to better visualize markers and possible crack development during test. Figure 4.11 illustrates the AC layer construction process.

(a)

(b)

(c)

(d)

Figure 4.11. Asphalt concrete layer construction: (a) transportation to the testing box; (b) leveling; (c) compaction; (d) surface aspect with white painting.

4.4 Asphalt overlay construction – Unreinforced sections

The asphalt overlay was constructed in the pavement sections in order to simulate a pavement rehabilitation system. In the second stage of tests, the asphalt overlay was installed over a previously tested asphalt layer (deteriorated surface). This new asphalt concrete layer (overlay) was placed in 60-mm thick lifts and compacted with a vibratory plate. The same compaction procedure used for the previous asphalt layer was followed. However, a surface leveling process was performed to repair deteriorated asphalt surface before the asphalt overlay compaction and also to install asphalt strain gage sensors. After this procedure, the surface to be overlaid received a thin tack coat of asphalt emulsion to facilitate a uniform adherence of the overlay. The CRS emulsion was applied at a rate of approximately 0.6 l/m^2 , according to DER-SP ET-DE-P00/043 (2006) when using geogrids. For both unreinforced and reinforced asphalt overlay construction process, the same tack coat rate was used. The new asphalt layer was placed after the tack coat breakage. Figure 4.12 illustrates the asphalt overlay construction process. The asphalt surface was also painted white to better visualize markers and possible crack development during the tests.

Figure 4.12. Asphalt overlay construction: (a) deteriorated surface; (b) leveling; (c) tack coat emulsion application; (d) new asphalt surface aspect.

4.5 Asphalt overlay construction – Geogrid-reinforced sections

For geogrid-reinforced test sections, the installation of the geogrid was conducted after the emulsion breakage, following the same process used for unreinforced tests. The geogrid was placed directly onto the tack coat, with the geotextile fabric facing down and the geogrid facing up. The geogrid was installed tight, without any folds or creases. The geogrid was attached to the box edge using clamping fixture (nails attached to the asphalt layer).

The paving process was done carefully over the geogrid-covered surface to minimize displacements. The new AC layer was also placed in 60-mm thick lifts and compacted with a vibratory plate. Figure 4.13 presents the geogrid installation procedure and the final aspect of the asphalt overlay. The asphalt surface was later painted white to better visualize markers and possible crack development during tests.

Figure 4.13. Geogrid installation: (a) geogrid installation; (b) asphalt mix over geogrid; (c) compaction; (d) asphalt surface aspect.

After each test was completed, 100-mm diameter asphalt cores were extracted from the outside of the wheel traffic path in order to define the compaction characteristics of asphalt layers, such as density, air void, indirect tensile strength and resilient modulus.

4.6 Assembly of the equipment

The assembly (and disassembly) of the equipment to the testing box was conducted using a travelling crane available in the Laboratory of Geosynthetics of USP. The lateral pieces of the wheel tracking device were attached to the testing box with the use of steel screws. An external support was used to place the equipment when not running tests. Figure 4.14 illustrates the assembly and disassembly of the equipment.

Figure 4.14. Assembly of the equipment: (a) movement using travelling crane; (b) fixing system; (c) APT ready for tests; (d) equipment external support.

4.7 Pavement test sections

In order to compare the results from a reinforced pavement section with an unreinforced one, two identical pavement sections were constructed and referred as Control sections. The Control sections were tested in the first stage of the tests. Figure 4.15 presents the layout of the Control sections. The first stage of tests was performed during a period of time starting immediately after the first 50 mm of asphalt concrete were placed and ending when the number of cycles (N) reached 10^5 cycles (medium to high traffic volume) or when the rut depth exceeded 25 mm (failure condition). The moving wheel loading mode (previously described in Chapter 3) was used in all test sections. Figure 4.16 presents the moving wheel loading mode and an example of the wheel pressure of 700 kPa registered throughout a test.

Figure 4.15. Pavement layout of the first stage of Test 1 and Test 2.

Figure 4.16. Moving wheel loading mode used in pavement: (a) dynamic input pulse; (b) example of the wheel load pressure registered throughout a test.

The second stage of tests was performed after the Control sections had been first loaded. In the second stage, a 60-mm asphalt overlay was placed over the previously degraded asphalt layer. The testing procedure with the asphalt overlay was subjected to the same traffic condition (10^5), load magnitude and frequency (Figure 4.16) as the previous HMA layer. The resurfaced sections included one test with a geogrid as reinforcement in the asphalt layer (Baseline test) and another without reinforcement (Unreinforced test), according to Figure 4.17. Figure 4.18 shows the flow chart of the testing procedure for both the first and second stages of tests.

Figure 4.17. Pavement layout of the second stage of: (a) Test 1 - Unreinforced; (b) Test 2 - Baseline.

A third test layout was proposed as the concept of using the geogrid as reinforcement of a single new asphalt layer. In this test, the geogrid was installed between two new asphalt layers. The first AC layer was compacted with a thickness of 50 mm, followed by the geogrid installation. A second AC layer was compacted with 60 mm of thickness, assuring the geogrid installation in the zone of tension (below neutral axes). At the same time, this pavement structure would have the same total AC layer thickness of Test 1 and Test 2. For this test, only one stage of 10^5 load cycles was performed. Figure 4.19 presents the pavement section layout of Test 3 (Single reinforced HMA layer).

Figure 4.18 - Flow chart of APT tests using wheel tracking machine for pavement sections.

Figure 4.19. Pavement layout of Test 3 – Single reinforced HMA layer.

A fourth test was carried out following the same testing routine as the geogrid-reinforced asphalt overlay test (Test 2), with the difference that a reduced base course thickness was used. Instead of the 200-mm thick as in the previous tests, a 100-mm thick base course was

constructed. This modification intended to simulate a less rigid pavement structure in comparison to other tests. A control section was constructed in the first stage of the test, and then a second asphalt layer was constructed with a geogrid reinforcing the asphalt overlay. The same 10^5 load frequency and failure condition were adopted for each stage of test. Figure 4.20 presents the pavement section layouts of Test 4 (Reduced Base). Table 4.7 presents the testing matrix for all pavement test sections studied in this research and the terminology used to describe each section test and stage.

Figure 4.20. Pavement layout of Test 4: (a) first stage; (b) second stage.

Table 4.7. Test sections and notation used

Test	Section	Stages	Geogrid installation	Notation	Data of test
1	Unreinforced	1° - Control asphalt layer	-	1A	Mar. – Apr./2012
		2° - Unreinforced asphalt overlay	-	1B	May – Jun./2012
2	Baseline reinforced	1° - Control asphalt layer	-	2A	Ago. – Sept./2011
		2° - Geogrid-reinforced asphalt overlay	Between lower and upper asphalt layers	2B	Oct. – Nov./2011
3	Single reinforced HMA	Single reinforced asphalt layer	Between two new asphalt layers	3	Out./12– Jan./2013
4	Reduced Base	1° - Control asphalt layer	-	4A	Ago. – Nov./2013
		2° - Geogrid-reinforced asphalt overlay	Between lower and upper asphalt layers	4B	May – Ago./2014

Table 4.8 presents the main properties of the AC layer for pavement test sections. Average values of thickness, TMD (theoretical maximum specific gravity) (ASTM D 2041) and air void values were determined from cores extracted from each section outside the wheel path after the loading was completed. The coefficient of variation (CV) is also given. Figure 4.21 shows the asphalt core extraction process for asphalt layer characterization. The asphalt concrete in these sections presents high air voids due to the compaction process. However, all the layers presented similar air void percentage, which is attributed to the relatively low compaction effort used in this study.

Table 4.8. As-constructed asphalt concrete layer properties for test sections

Test section	Asphalt layer	Thickness (mm)	TMD (kN/m ²)	Air Voids (%)
1A	Layer 1	53,16 (CV = 2.87%)	2.676	9.84 (CV = 3.5%)
1B	Layer 2	61.08 (CV = 2.67%)	2.676	10.20 (CV = 5.4%)
2A	Layer 1	52,12 (CV = 3.54%)	2.677	10.42 (CV = 3.0%)
2B	Layer 2	62.00 (CV = 3.16%)	2.677	11.41 (CV = 5.26%)
3	Layer 1	53,5 (CV = 3.54%)	2.672	9.9 (CV = 4.0%)
	Layer 2	62.04 (CV = 2.50%)	2.672	10.0 (CV = 4.4%)
4A	Layer 1	52,5 (CV = 3.54%)	2.672	9.80 (CV = 2.0%)
4B	Layer 2	60.00 (CV = 3.90%)	2.672	9.0 (CV = 5.4%)

Figure 4.21. Asphalt layer characterization (a) core extraction; (b) asphalt core of the entire layer; (c) asphalt cores for tests.

Table 4.9 presents the results of indirect tensile strength (DNER-ME 138/94) tests performed on asphalt concrete cores and resilient modulus (DNER-ME 133/94). Tests were performed at room temperature (25 °C). Results show an important reliability on indirect tensile strength and resilient modulus results obtained in layers 1 and in layers 2 for all tests, providing evidenced that the construction and materials are very consistent in all tests. This means that any difference in pavement response can be due to the reinforcement inclusion. Additionally, the fatigue response of asphalt paving mixtures was analyzed for the applicability of the test results, for use in modeling pavement performance. It is of great importance to emphasize this type of fatigue test as being the most used in mechanistic design approaches of flexible pavements in Brazil. Figure 4.22 shows the indirect tensile strength and fatigue tests. Figure 4.23 presents the fatigue response for asphalt concrete mixes (layer 1) used in this research in Tests 1, 2 and 3.

Table 4.9. Indirect tensile strength values and resilient Modulus of asphalt concrete

Test section	Asphalt layer	Indirect tensile strength (MPa)	MR (MPa)
1A	Layer 1	1.09	3730 (CV =24.05%)
1B	Layer 2	0.968	4977 (CV =14.0%)
2A	Layer 1	1.04	3595 (CV =14.99%)
2B	Layer 2	0.876	4600 (CV =15.90%)
3	Layer 1	1.12	3581 (CV =18.66%)
	Layer 2	0.98	4660 (CV =13.74%)

(a)

(b)

Figure 4.22. Asphalt concrete mechanical characterization: (a) indirect tensile strength test; (b) fatigue test.

Figure 4.23. Fatigue response of asphalt paving mixtures used in this research.

4.8 Instrumentation

An instrumentation program was designed to monitor pavement sections in order to quantify the mechanical response of the pavement materials under dynamic loading. Instrumentation has been categorized into sensors for measuring: asphalt surface deflection, stress and strain in the asphalt layer, and stress in the base course and subgrade layers. The data acquisition system was configured to record information on the full-time history of response of described load cycles (Figure 4.16). Figure 4.24 presents the instrumentation layout of a typical pavement section. All pavement sections were constructed using the same instrumentation layout.

(a)

(b)

Figure 4.24. Layout of instrumentation: (a) transverse section - front view; (b) longitudinal section.

Figure 4.25. Transverse instrumentation centerline and asphalt surface area of analysis.

4.8.1 Displacement transducers

A total of eight Linear displacement sensor (LDS) were used to monitor asphalt surface deformation, with full-scale ranging from 50.8 mm (Vishay – Model HS50) and 100 mm (Kyowa – Model DT-A). The LDSs allowed determination of the transverse rutting profiles (plastic and elastic deflections) as a function of the number of loading wheel passes in the

frequency of 100 Hz. The LDSs were placed from the vicinity of the load center, where the largest surface deformations occur, to the box edge. Additional permanent deflection profiles were measured by using a perfilometer in the vicinity of the load area after 1, 10, 100, 500, 1,000, 2,500 and 5,000 passes, and after intervals of 10,000 wheel passes. The surface profiles were determined by first taking profile measurements before pavement loading (pass level 0). The reference was taken at the same level. A typical LDS arrangement used in the test sections is shown in Figure 4.26. A detail of the permanent displacement measurement is also presented in the figure.

Figure 4.26. LDSs used in the pavement test sections: (a) arrangement; (b) detail of permanent displacement measurement.

4.8.2 Asphalt strain gages

The asphalt strain gages used to measure asphalt concrete strain in this research were ASG-152 H-type (CTL Group). This strain gage has a physical range of $\pm 2000 \mu\epsilon$, temperature up to 200°C and sensitivity of $0.06 \text{ N}/\mu\epsilon$. The gage is a 350-ohm full bridge strain gage and is individually calibrated. The lead-in wires connected to the gages were covered with a nylon sheath to protect them from the effects of temperature and abrasion. Figure 4.27a presents the asphalt strain gages. For the sensors installed at the bottom of the first asphalt layer, a shallow cavity was dug on the surface of the granular base and leveled using a 25-mm layer of sand-asphalt emulsion mix. This procedure prevents damage from the aggregate particles, as described by Garg e Hayhoe (2001), Timm *et al.* (2004), Gopalakrishnan and Thompson (2006) and Graziani *et al.* (2014). Figure 4.27b presents the positions of the sensors at the granular base surface. The ASGs were oriented both transversely and longitudinally to the wheel track.

Before paving operations, a small amount of asphalt concrete was placed over the sensors and hand compacted. This was done in order to provide additional protection against subsequent rolling/vibratory impact. For the sensors to be installed between the existing and new asphalt layers, shallow cavities were created on the surface of the existing asphalt layer in order to obtain proper space for the installation of ASGs (Figure 4.27c). A thin layer of sand-asphalt emulsion mix was used to tack the ASGs in their correct positions and alignments inside the prepared cavities. The sensors were protected by filling and leveling the cavities with a small amount of asphalt concrete (4.27d). A light compaction was used to minimize the movement of the sensors during paving. This procedure was simultaneously done with the asphalt surface leveling (4.12b). Figure 4.28 presents asphalt strain gages placed directly below the wheel path in asphalt layers 1 (existing) and 2 (new). An inspection of the sensors carried out at the end of the construction revealed that all ASGs had survived the working operations.

Figure 4.27. Asphalt strain gage: (a) ASG 152 H-type; (b) positioning; (c) cavities; (d) cavities leveling.

Figure 4.28. Positions of the asphalt strain gages placed directly below the wheel path.

4.8.3 Pressure transducers

Pressure transducers (Earth pressure cells) by Kyowa were used to monitor vertical dynamic stress in the pavement layers. Pressure transducers were installed in the interface base course-asphalt layer (BEC A - 1 MPa range), at the middle of the base course (BEC A - 500 kPa range) and at the top of the subgrade (BEC A - 200 kPa range). For the reinforced sections, transducers were placed over the geosynthetic reinforcement. At the subgrade and base course, a pre-hole filled with sand was adopted to ensure the same conditions used during the calibration process of the transducers. At the asphalt layer, the same protection process used for asphalt gages was adopted, except for the pressure transducer, which was wrapped with temperature heat tape. Light compaction was used to minimize the movement of the sensors during paving. Figure 4.29 presents the installation of pressure transducers. Inspection of the sensor revealed that all pressure cells survived the working operations.

Figure 4.29. Pressure transducers: (a) subgrade layer; (b) base course; (c) temperature protection; (d) asphalt concrete mix over the new asphalt layer.

4.8.4 Horizontal displacement measurement system

The displacement that occurred in the geogrid when in asphalt layers and subject to traffic load was an important input parameter to understand the geogrid mobilized tension (at low strain levels). Displacements along the geogrid ribs were then monitored using tell-tale system. The tell-tale system allowed displacement measurements through stainless wires attached to the geogrid on one side and connected to displacement sensors (LX-PA from Unimeasure) on the other side. The displacement sensors (0-70 mm), which consisted of a cable and spring assembly, were then aligned to the wires of the tell-tales connected to the geogrid. The displacement readings involve the voltage generated due to movement of the cable changes. Figure 4.30 shows the displacement measurement system.

Figure 4.30. Displacement measurement system: (a) LX-PAs; (b) LX-PA and wire connection; (c) support system for attaching sensors outside the testing box.

Displacements along the geogrid were monitored with tell-tales at 15 different points distributed along the transverse rib of the geogrid (cross-section centerline) as shown in Figure 4.31. The distances of the measurement points along the geogrid were set to 48 mm and used to calculate strains. The strain distribution was defined by assuming a uniform strain between two consecutive points. Then, the average tensile strain response mobilized in the geogrid between these points (transverse direction) was calculated based on direct measurements of internal horizontal displacements. Due to the symmetry of the model, only one side of the cross-section was monitored. Wires were fixed to the geogrid rib by nodes and bonded with epoxy to it, as shows Figure 4.32. The wires were protected with Teflon tubes (greased inside) to assure temperature protection and frictionless movement.

(b)

Figure 4.31. Tell-tale position along the geogrid: (a) cross-section; (b) top view.

(a)

(b)

Figure 4.32. Wire fixing in the geogrid: (a) node; (b) tube protection and bonding detail.

For comparison purposes, the unreinforced test section interlayer was monitored using the same system. However, a 60 mm strip of nonwoven geotextile monitored using the tell-tale system was used since no reinforcement was present in this section. The geotextile used was a 100% polyester nonwoven (NWGT) of 7 kN ultimate tensile strength. A strip was chosen to enable asphalt strains measurements at the asphalt interlayer without developing significant

tension within the pavement structure. Displacements along the geotextile strip were monitored with tell-tales at 15 different points (cross-section centerline) as shown in Figure 4.33. The same 48-mm distance of the measurement points along the strip was used to calculate strains.

Figure 4.33. Position of the tell-tales at the unreinforced interlayer: (a) cross-section; (b) strip of nonwoven geotextile; (c) wires fixing.

5 PRESENTATION OF THE EXPERIMENTAL RESULTS

The data from accelerated pavement tests conducted in this research are presented in this chapter. Discussion of the results in this chapter includes the evaluation of each pavement section, presenting asphalt surface permanent and elastic displacements, dynamic vertical stresses in pavement layers and asphalt concrete strains developed during tests. Comparative analyses between pavement sections will also be presented in Chapter 6 “Discussion of the Results”. Detailed analyses of displacement profiles along the geogrid and the calculations of strains are also presented in Chapter 6.

5.1. Testing Matrix

The testing matrix of this research is presented in Table 5.1, comprising the 4 tests conducted in this study. In the first stage of Tests 1 and 2, control sections (as referred by a flexible pavement structure composed by asphalt layer, aggregate base and subgrade layer) were tested. In the second stage of these tests, an asphalt overlay was applied to pavement structures. The second stage of Test 1 was conducted without geosynthetic reinforcement; whereas the second stage of Test 2 presented a PVA geogrid as overlay reinforcement. Test 2 was defined as the baseline test since a conventional geogrid-reinforced asphalt overlay process was simulated. Test 3 was conducted in only one stage of test having a geogrid reinforcing the interfaces of newly installed asphalt layers. Test 4 was also conducted in two stages of tests (control and overlay section), however with reduced base course thickness in comparison to previous tests. This configuration for tests was chosen to allow future comparisons of these results.

Table 5.1 – Testing matrix

Test	Layer	Layer description	Geogrid installation
1- Unreinforced section	1A	Control asphalt layer	-
	1B	Unreinforced asphalt overlay	-
2- Baseline reinforced section	2A	Control asphalt layer	-
	2B	Geogrid-reinforced asphalt overlay	Between lower and upper asphalt layers
3- Single reinforced HMA section	3	Single reinforced asphalt layer	Between two new asphalt layers
4- Reduced Base reinforced section	4A	Control asphalt layer	-
	4B	2° - Geogrid-reinforced asphalt overlay	Between lower and upper asphalt layers

5.2 Test 1 – Unreinforced Section

This section presents the results of Test 1 conducted using APT tests. This test did not have geosynthetic reinforcements. The first stage of this test involved a flexible pavement structure composed by asphalt layer, base course and subgrade layer. This control section (Test 1A) was subjected to 10^5 load cycles in the moving wheel load mode. In the second stage of Test 1, an asphalt overlay was constructed over the previously tested asphalt surface in order to simulate a pavement rehabilitation project. The unreinforced asphalt overlay section (Test 1B) was also loaded with 10^5 cycles.

5.2.1 Rut depth measurements

Analysis of the permanent vertical displacements were conducted in two ways: measured rutting profiles and accumulated surface vertical displacements. First, rut depth measurements along transverse centerline were averaged for each pass level as a function of the number of wheel load passes. Figure 5.1 shows the measured rutting profiles as function of the number of load applications for sections 1A and 1B, for pass levels of 10,000, 30,000, 50,000, 80,000 and 100,000. Results of Figure 5.1a show that before 10,000 load cycles, an increasing rate of deformation was evidenced, probably attributed to AC layer densification. Air voids in the AC layer of 9.84% were recorded in this section (Table 4.8), mainly influenced by the asphalt layer compaction achieved during asphalt layer construction. Heave was identified as the areas of the pavement surface that showed elevations higher than adjacent areas. Figure 5.1b also presents an initial densification phase before 10,000 load cycles, with approximately the same compaction condition of the asphalt surface layer (10.20% of air voids). However, due the comparatively high stiffness of this pavement section, smaller levels of rut depths were noted after testing, in comparison to the rut depth values measured during the first stage of the test (before overlay placement). The presence of heave displacements adjacent to ruts could also be evidenced in section 1B, as presented in a final rutting profile for both pavement sections (Figure 5.2). Is important to note that for these rut depth values, the deformation within the pavement structure may be located within the asphalt layers and aggregate base, and more evenly distributed through the depth of the pavement.

(a)

(b)

Figure 5.1. Rut depth measurements along transverse centerline of Test 1: (a) section 1A; (b) section 1B.

Figure 5.2. Final rut profile along transverse centerline of Test 1.

Figure 5.3 shows rutting as accumulated surface vertical displacements at the centerline plotted as a function of the number of wheel load passes, exhibiting maximum rut depth measurements of each test. Maximum value of 6.1 mm rut depth was observed for section 1A (Figure 5.3a), resulting in few micro cracks alongside wheel path. These fissures appeared after 80,000 cycles appearing in longitudinal direction parallel to the wheel path. Figure 5.4 illustrates the rutted surface and microcracking occurring during Test 1A. One of the possible reasons for the development of cracks in this location is the high magnitudes of rut depths. As presented in Figure 5.3b, a maximum rut depth of approximately 4.9 mm was verified in the second stage of the test (Test 1B), with no visible cracks in the asphalt surface.

Figure 5.3. Accumulated surface vertical displacements at the centerline of Test 1: (a) section 1A; (b) section 1B.

Figure 5.4. Asphalt surface of section 1A: (a) rutted wheel path; (b) microcracking at the edge of the wheel path during test.

5.2.2 Elastic surface deflections

Pavement surface deflections provides information that can be used for the structural evaluation of new or in-service flexible and rigid pavements ASTM D 4965 03 (2008). Deflections are measured with sensors that monitor the vertical movement of a pavement surface due to the load and can be used in back calculation methods to determine pavement structural layer stiffness and the subgrade resilient modulus. Thus, many characteristics of flexible pavements can be determined by measuring their deflection in response to traffic load. The pavement response is known as the “deflection basin”. Figure 5.5 presents examples of LDS total readings used to obtain information of elastic deflection results. Figure 5.6 shows the elastic deflection basin of sections 1A and 1B for load cycles 500, 1000, 10,000, 50,000 and 100,000. Considering the deflection basin evolution presented in Figure 5.6, results showed a decrease with increasing number of wheel load for both control and overlay pavement sections.

Figure 5.5. Example of LDS readings during tests: (a) 173 mm from the wheel location; (b) 323 mm from the wheel location (c) 473 mm from the wheel location; (d) 623 mm from the wheel location.

Figure 5.6. Deflection basins for different load cycles of Test 1: (a) section 1A; (b) section 1B.

Figure 5.7 shows the final elastic deflection basin for sections 1A and 1B at the end of 10^5 load cycles. It is observed a substantially larger deflection close to the point of loading for the control section in comparison to that obtained during the second stage of the test, although showing similar curve shape. This behavior represents an expected curve for a good stress distribution pavement section, since deflections decrease rapidly with distance from the point of loading. LDSs readings away from the wheel load show maintenance of very small elastic deflection values (close to the edge of the box). Figure 5.8 presents maximum cyclic elastic surface deflections measured during tests (close to wheel load application). As indicated in Figure 5.8, maximum elastic deflections have slightly varied with the number of load cycles, showing a decrease trend at the end of the Test 1A, while reaching a plateau at 0.35 mm during Test 1B. The presence of an asphalt overlay resulted in a reduction of maximum elastic deflections of approximately 22%.

Figure 5.7. Final deflection basins at 10^5 load cycles of Test 1: (a) section 1A; (b) section 1B.

Figure 5.8. Maximum cyclic elastic surface deflections in Test 1.

5.2.3 Dynamic stress response

Instrumentation also allowed the evaluation of stresses induced during loading wheel passes (dynamic stress response of the test sections). The dynamic vertical stress response within the middle of the base course and at the top of the subgrade are presented in Figure 5.9 for section 1A (after 50,000 load cycles – half of the test). As indicated in Figure 5.9a, the pressure transducer installed in the middle of the base course presented maximum stress of 90 kPa. This value was maintained during the test. The pressure transducers installed at the top of the subgrade showed an average peak stresses of 47 kPa (Figure 5.9b). The pressure transducer installed at the bottom of the asphalt layer did not give reliable results, and a survey after the test showed that an aggregate particle had damaged the pressure transducer.

(a)

(b)

Figure 5.9. Typical results of dynamic vertical stresses of section 1A: (a) middle of the base course; (b) top of the subgrade.

The results presented in the second stage of the test (1B) are presented as typical dynamic stress response at the bottom of asphalt layer 1, middle base course and at the top of the subgrade, showing an example of registered vertical stresses during this test (Figure 5.10). The pressure transducer installed in the asphalt layer 1 presents an average peak value of 152 kPa (Figure 5.10a) during the test. Base course pressure transducer results showed an average peak value of 50 kPa up to the end of the test, while 40 kPa were recorded in the subgrade. The pressure transducer installed between asphalt layers (at the bottom of asphalt layer 2) has not given reliable results, as observed in section 1A. As expected, for the second stage of test, the presence of an asphalt overlay reduced the dynamic vertical stresses in 47 and 50% for base course and subgrade layers, respectively.

(a)

Figure 5.10. Typical results of dynamic vertical stresses of section 1B: (a) bottom of asphalt layer 1; (b) middle of the base course; (c) top of the subgrade.

5.2.4 Asphalt concrete strain response

Asphalt Strain Gages (ASGs) were installed in both the longitudinal and transverse directions of the wheel path to measure load-induced AC tensile strains during traffic testing. ASGs were installed along a longitudinal alignment, 350 mm apart on either side of the instrumentation centerline, underneath the asphalt layer of the section 1A. This position was chosen in order to evaluate the horizontal tensile strains, in both longitudinal (L) and transverse (T) directions. An example of elastic signals recorded by an ASG during a test is presented in Figure 5.11.

Figure 5.11. Asphalt strain response during a test: (a) total load cycles; (b) signals recorded.

Figure 5.12 presents typical results of the ASGs installed in the bottom of asphalt layer of section 1A. The figure shows the results obtained after 50,000 load cycles (i.e. time zero in the figure is 50,000 cycles). Vertical scales in the plots are different for a better visualization of the recorded signals. As observed in Figure 5.12a, longitudinal strains recorded by the ASG L showed the expected compression of comparatively small amplitude (when wheel was approaching) and then a comparatively extension (peak strain). The transverse strain recorded by the ASG T showed the expected tension-only response, with the highest level of elastic strains. Maximum elastic strains measured by ASG L were recorded as $500 \mu\epsilon$ and the readings of the ASG T were in the range of $700 \mu\epsilon$.

Figure 5.12. Typical response of ASGs in section 1A: (a) ASG L; (b) ASG T.

For the second stage of the test (1B), the ASGs installed in the bottom of the existing asphalt layer remained installed, and two other ASGs were installed in the interface of the lower layer and upper layer (60 mm below surface). The ASGs installed in the bottom of asphalt layer

1 (lower layer) will be now referred as ASG 1 and at the bottom of asphalt layer 2 (ASG 2). Thus, the total of 4 sensors will be referred as ASG 1L, ASG 1T, ASG 2L, ASG 2T.

Figure 5.13 presents the results of ASG installed in the bottom of asphalt layer 1 of section 1B, representing an average peak tensile strains results after 50,000 load cycles. Maximum horizontal tensile strains measured by ASG 1L were recorded as 410 $\mu\epsilon$ and the readings of ASG 1T were maximum 270 $\mu\epsilon$. Comparing these results with the results from the previous test (before applying overlay), registers of maximum elastic strains have decreased from 480 to 410 $\mu\epsilon$ for the longitudinal ASG and 700 to 270 $\mu\epsilon$ for transverse ASG. This behavior is explained by the inclusion of an asphalt overlay in the pavement sections, leading to smaller levels of horizontal tensile strains in the lower asphalt layer. The sensors installed in asphalt layer 2 (asphalt overlay) have shown elastic strains results of 280 $\mu\epsilon$ for ASG 2L and 200 $\mu\epsilon$ for ASG 2T (Figure 5.14). Vertical scales in the plots are different for a better visualization of the signals recorded during cycles.

Figure 5.13. Typical response of ASGs in section 1B: (a) ASG 1L; (b) ASG 1T.

As observed in Figures 5.13a and Figure 5.14a, longitudinal strains recorded at positions 1L and 2L showed a compression amplitude followed by an extension (peak strain). Transverse strains recorded at the position 1T showed the expected tension-only response, while ASG 2T showed possible alignment problem during installation, with a peak tensile strain followed by a comparatively smaller second peak. Results show that the longitudinal elastic strains in the asphalt layer are greater than the transverse strains in the second stage of the test. Also, results of ASG 1L and ASG 1T showed the same pattern as in the first stage of the test.

Figure 5.14. Typical response of ASGs in section 1B: (a) ASG 2L; (b) ASG 2T.

Figure 5.15 shows accumulated plastic strains recorded by the ASG installed in section 1B. The highest plastic strains were obtained for longitudinal ASGs and for transverse ASG 2T installed in the upper layer.

Figure 5.15. Plastic strains response in asphalt concrete layers of section 1B.

The use of ASGs demonstrated the ability to measure with confidence the load-induced asphalt concrete tensile strains under 700 kPa of traffic loading. The results of asphalt strain gages presented here for section 1B have shown longitudinal elastic strains greater than transverse strains. Evaluation of the sensors carried out at the end of construction revealed that all sensors survived the working conditions of this test.

5.2.5 Internal displacements

The strain distribution within reinforcement layers was monitored to allow the determination of the magnitude and location of the reinforcement maximum peak strains. In order to compare the strain distribution of the geogrid-reinforced pavement sections with that of an unreinforced section, the same tell-tales system was used. A strip of NWGT was instrumented with 15 extensometers, as previously described in Chapter 4, and installed between the existing and new asphalt layers of section 1B, transversely to wheel load path. Thus, internal horizontal displacements in the asphalt concrete layer could be evaluated.

Figure 5.16 presents the results of permanent displacements registered by the extensometers during Test 1B. The assessment of extensometers results indicates, as expected, the internal displacements increases with increasing number of load cycles. All LDSs showed positive (tensile) readings during the test. LDS 1 and LDS 2 are installed under the wheel load (symmetrically in the same position in relation to the wheel centerline), as indicated in Figure 4.33a. The following sensors, LDS 3 to LDS 15 are installed in one side of the transverse centerline of the testing box, with LDS 15 close to the edge of the box (Figure 4.33a).

Figure 5.16. Permanent displacements data with the number of load cycles of Test 1.

As presented shown in Figure 5.16, a comparatively high displacement level was obtained for the LDSs nearby the wheel load (LDS 1 to LDS 6). Maximum and equivalent evolution of horizontal internal displacements obtained in LDSs 1 and 2 showed values of 0.55 mm of accumulated displacements. Results of horizontal displacement profiles, elastic

displacements and the calculation of strains along the interface of existing and new asphalt layers will be presented in the “Discussion of the Results” Chapter.

5.3 Test 2 – Baseline Reinforced Section

This test was chosen as the baseline test since a conventional asphalt overlay reinforcement with the use of a geogrid was simulated in this test. In the first stage of test, a flexible pavement structure composed by an asphalt layer, a base course and a subgrade layer, was subjected to 10^5 wheel load cycles (Test 2A). In the second stage, a geogrid reinforcement was installed over the previously tested asphalt layer and an asphalt overlay was constructed (Test 2B). Section 2B was also loaded with 10^5 cycles in the moving wheel load mode. However, since this test was conducted during the initial phase of the research, a comparatively limited instrumentation was available in the Laboratory. For section 2A, no asphalt strain gages were installed in the asphalt layer. In the second stage, the asphalt strain gages were available; however, only 5 out of 15 horizontal displacement sensors were installed in the geogrid due to a problem with the data acquisition system. Therefore, geogrid displacement measurements for this test will not be presented.

5.3.1 Rut depth measurements

Figure 5.17 shows measured rutting profiles with the number of load applications for Test 2, for pass levels of 10,000, 30,000, 50,000, 80,000 and 100,000. According to Figure 5.17a, an initial significant densification phase (before 10,000 load cycles) was evidenced in both stages of Test 2, similarly to the response observed in Test 1. Section 2A presented air voids in the AC layer of 10.42% (Table 4.8), also attributed to the asphalt layer compaction achieved during asphalt layer construction. Heaves alongside wheel path were also prominent in this section.

Figure 5.17. Rut depth measurements along transverse centerline of Test 2: (a) section 2A; (b) section 2B.

Figure 5.18 illustrates the rutted surface of section 2A. Figure 5.17b presents the second stage of the test, in which a geogrid was installed in the asphalt layer, also showing an initial densification phase before 10,000 load cycles, with air voids of approximately 11.4%. Final rutting profile for both pavement sections is presented in Figure 5.19. As expected, smaller levels of rut depths were noted in the overlay section due to pavement increased stiffness, in comparison to the first stage of the test. Additionally, heaves were also reduced. The accumulated surface vertical displacements for both stages of the test are plotted as a function of the number of wheel load passes in Figure 5.20. The overall rut depth for the control section at the end of the test was approximately 6.4 mm (Figure 5.20a). Figure 5.20b, shows maximum rut depth of approximately 3.1 mm for section 2B. No sign of cracking was detected in Test 2.

(a)

(b)

Figure 5.18. Asphalt surface of section 2A: (a) rutted wheel path; (b) detail of rutting.

Figure 5.19. Final rut profile along transverse centerline of Test 2.

(a)

(b)

Figure 5.20. Accumulated surface vertical displacements at the centerline of Test 2: (a) section 2A; (b) section 2B.

5.3.2 Elastic surface deflection

Figure 5.21 shows the deflection basin of first and second stages of Test 2 at 500, 1000, 10,000, 50,000 and 100,000 load cycles. The results showed a decrease in elastic deflections with increasing number of wheel load cycles for section 2B (Figure 5.21b), while the evolution of elastic displacements of the control section (2A) have varied with the time (Figure 5.21a). Figure 5.22 shows the final deflection basin of both tests for 10^5 load cycles.

Figure 5.21. Deflection basin for different load cycles of Test 2: (a) section 2A; (b) section 2B.

Figure 5.22. Final deflection basins at 10^5 load cycles of Test 2: (a) section 2A; (b) section 2B.

As observed in Figure 5.22, deflections decrease rapidly with distance from the point of loading for section 2A when compared to the overlay section 2B, showing the behavior of a thick pavement. Very small elastic deflection values (close to the edge of the box) were noted for both sections (approximately 0.05 mm). Figure 5.23 presents maximum cyclic elastic deflections (LDSs close to the wheel load point) measured during test sections 2A and 2B.

Maximum elastic deflections have varied with the number of load cycles in section 2A, showing a decrease trend at 10^5 load cycles. For the second stage of the test, a tendency of slight decrease can be observed, achieving 0.15 mm at the end of the test. The presence of an asphalt overlay reinforced with geogrid have reduced maximum elastic deflections in approximately 68% in comparison to the first stage of test.

Figure 5.23. Maximum cyclic elastic surface deflections in Test 2.

5.3.4 Dynamic stresses response

Dynamic vertical stress response at pavement layers were also investigated in the first and second stages of Test 2. Pressure transducers typical results installed in the middle of the base course and at the top of subgrade are presented in Figure 5.24 for section 2A (after 50,000 load cycles). Middle of the base course pressure transducer indicated results of average peak values of 95 kPa during this test (Figure 5.24a). The pressure transducers installed at the top of the subgrade presented results of an average peak values of 35 kPa (Figure 5.24b).

(a)

(b)

Figure 5.24. Typical results of dynamic vertical stresses of section 2A; (a) middle of the base course; (b) top of the subgrade.

The pressure transducer installed at the bottom of the asphalt layer have responded to wheel load, but with increasing and decreasing stress response. Figure 5.25 presents marks in the wheel path where pressure transducer was installed. It is believed that the impact of the wheel load have affected the pressure transducer results. Due to the questionable data collected in this stage of Test 2, the analysis of dynamic responses at the bottom of the asphalt layer are not presented.

Figure 5.25. Detail of the pressure transducer installed at the bottom of the asphalt layer in section 2A.

Figure 5.26 shows an example of the average peak vertical stresses registered during Test 2B at the bottom of asphalt layer 1, middle of the base course and at the top of the subgrade, respectively. Dynamic vertical stress response at the bottom of asphalt layer 1 (lower layer), middle of the base course and at the top of the subgrade are presented for section 2B. In the second stage of the test, the pressure transducer installed in the bottom of asphalt layer 1 started to response appropriately, presenting 104 kPa during the test. Base course pressure transducer results showed an average peak of 42 kPa and the subgrade pressure transducer have presented results of an average peak of 20 kPa during the test.

(b)

(c)

Figure 5.26. Typical dynamic vertical stresses of section 2A: (a) bottom of asphalt layer 1; (b) middle of the base course; (c) top of the subgrade.

5.3.5 Asphalt concrete strain response

Asphalt concrete strain response in section 2A will not be presented here since no asphalt strain gages were available in this test. In the second stage of the test (section 2B), ASGs were installed in the upper asphalt layer. Figure 5.27 presents typical results of ASGs (after 50,000 load cycles) installed in the bottom of the upper asphalt layer in of section 2B, in both longitudinal and transverse directions. The average peak tensile strains measured by ASG 2L was 125 $\mu\epsilon$, whereas the readings of peak strains of ASG 2T were averaged in 115 $\mu\epsilon$.

Figure 5.27. Typical response of ASGs in section 2B: (a) ASG L; (b) ASG T.

As observed in Figure 5.27a, when wheel load passed over the instrumented area, longitudinal elastic strains recorded at position 2L showed an initial compression of reduced amplitude and then an extension when the tire footprint reached the ASG position (peak). At the same time, the transverse strains recorded at position 2T showed a high peak compression and a second small tensile peak, explained by the alignment of the sensors during installation. Figure 5.28 presents the accumulated plastic strains recorded by the ASGs, showing comparable plastic strains evolution during the test.

Figure 5.28. Plastic strains response in asphalt concrete layers of section 2B.

In general, results of ASGs presented in section 2B have shown longitudinal tensile strains greater than the tensile transverse strains. A sensor survey carried out at the end of construction revealed that all sensors survived the working conditions of this test. Figure 5.29 illustrates the ASGs being removed from asphalt layer and their final aspect after the test.

Figure 5.29. Detail of the ASGs: (a) removal process; (b) aspect of the sensor after the test.

5.4 Test 3 - Single reinforced HMA section

5.4.1 Rut depth measurements

Test 3 (Single reinforced HMA layer) was conducted in only one stage of test having a geogrid reinforcing the interfaces of newly installed asphalt sub layer (50 mm) and an upper layer (60 mm). Results of permanent vertical displacements along wheel path transverse centerline are presented in Figure 5.30. Figure 5.30a presents rutted profiles for different pass levels (10,000, 30000, 50,000, 80,000 and 100,000) and Figure 5.30b represents the final rutted profile (10^5 cycles in the moving wheel load mode).

(b)

Figure 5.30. Rut depth measurements along transverse centerline of Test 3: (a) different load cycles; (b) final rutting profile.

Figure 5.31 presents rutting evolution and final aspect of the asphalt surface of the Test 3 section. Results in Figure 5.31 a show an increasing initial rate of deformation up to 10,000 load cycles, also evidenced in all test sections. In terms of asphalt layer compaction conditions, air voids of lower and upper layers were approximately 10% (very similar to the values observed in other tests). The aspect of asphalt surface at the end of the test showed minimal rutting (maximum 1.7 mm) and heaves alongside the wheel path (0.67 mm). No cracks were observed in this test. This behavior was expected for this test since the pavement section had a thick asphalt layer (110 mm, reinforced with a geogrid).

(a)

(b)

Figure 5.31. Rutting evolution of Test 3: (a) accumulated surface vertical displacements at the centerline; (b) asphalt surface final aspect.

5.4.2 Elastic surface deflections

Pavement surface deflections were measured during Test 3 and results were plotted with the distance from the applied wheel load in Figure 5.32. Figure 5.32a shows the deflection basin evolution at 500, 1000, 10,000, 50,000 and 100,000 load cycles, exhibiting a decrease in elastic deflection levels with increasing number of wheel load cycles. Figure 5.32b shows the final deflection basin of this test for 10^5 load cycles. The curve presented in elastic displacement's data represents the expected result of a thick pavement, since the slope of deflection basin shows a constant decrease of elastic deflections with distance from the point of loading. Maximum elastic deflections results (close to the wheel load point), presented in Figure 5.33, showed a decrease trend and stabilization at the end of the test, achieving to 0.09 mm at the end of the test.

Figure 5.32. Deflection basins of Test 3: (a) different load cycles; (b) 10^5 load cycles.

Figure 5.33. Maximum cyclic elastic surface deflections in Test 3.

5.4.3 Dynamic stress response

Figure 5.34 presents typical dynamic stress response of pressure transducers installed in the bottom of asphalt layers 1 and 2, middle of the base course and at the top of the subgrade of Test 3. Results are presented as an average of peak vertical stresses registered during the test (after 80,000 load cycles). Pressure transducers installed in the asphalt layers 1 (lower layer) and 2 (upper layer) have registered 75 and 160 kPa of stress peaks during the test (Figures 5.34a and 5.34b). In this test, pressure transducer installed in the upper layer presented reliable results, different from the observed in Test 1 and Test 2. Base course pressure transducer results showed an average peak of 35 kPa (Figure 5.34c). The top of the subgrade pressure transducer presented results of an average peak of 15 kPa during the test (Figure 5.34d). Vertical scales in the plots were set differently for a better visualization of the signals recorded.

Figure 5.34. Typical dynamic vertical stresses of Test 3: (a) bottom of asphalt layer 2; (b) bottom of asphalt layer 1; (c) middle of the base course; (d) top of the subgrade.

5.4.4 Asphalt concrete strain response

In Test 3, all ASGs were installed during the same period in both lower and upper asphalt layers since this test was conducted in only one stage. Thus, results of asphalt concrete strains will be presented as ASG 1L and ASG 1T (lower layer) and ASG 2L and ASG 2T (upper layer). Figure 5.35 presents ASGs results at the bottom of asphalt layer 1, considering an average peak results after 50,000 load cycles. Average peak tensile strains measured by ASG 1L were recorded as approximately 110 $\mu\epsilon$ and the readings of ASG 1T as 100 $\mu\epsilon$.

Figure 5.35. Typical response of ASGs in Test 3: (a) ASG 1L; (b) ASG 1T.

Sensors installed in asphalt layer 2 have shown results around 100 $\mu\epsilon$ in both longitudinal and transverse directions (Figure 5.36). As observed in Figures 5.36a, longitudinal strains recorded at positions 2L showed a compression when wheel load was approaching and then an extension (peak), as observed in most of the ASGs results in longitudinal direction. Figure 5.35a presented only tensile behavior for ASG 1L. Transverse strains recorded at positions 1T and 2T showed a single tensile peak (Figure 5.35b and Figure 5.36b). In general, longitudinal elastic strains in the AC layer were greater than the transverse strains. Figure 5.37 illustrates the accumulated plastic strains recorded by the ASGs installed in this section. The highest plastic strain levels were noted for longitudinal strain gages.

Figure 5.36. Typical response of ASGs in Test 3: (a) ASG 2L; (b) ASG 2T.

Figure 5.37. Plastic strains response in asphalt concrete layers of Test 3.

5.4.5 Displacements along geogrid

In order to monitor strain distribution within the geogrid reinforcement in the asphalt layers, a transverse section of the geogrid was instrumented with the use of extensometers and the tell-tails system (previously described in Chapter 4). Horizontal displacements along the geogrid were monitored during the test in 15 attached points and are presented in Figure 5.38. The assessment of displacements distribution results in Figure 5.38 indicates horizontal displacement's magnitude along the geogrid increasing with increasing number of load cycles. According to these results, all LDSs showed positive (tensile) readings during the test. LDSs installed under the wheel load (LDS 1 and LDS 2), symmetrically in the same position (Figure 4.33a), showed the highest level of displacements during the test (maximum 0.22 mm). LDSs distant from the wheel load showed the minor displacement levels. Results of plastic and elastic

horizontal displacement and the calculation of strains along the geogrid will be presented in Chapter 6 “Discussion of the Results”.

Figure 5.38. Permanent displacements data with the number of load cycles of Test 3.

5.5 Test 4 – Reduced base reinforced section

5.5.1 Rut depth measurements

Figure 5.39 shows rutting magnitude with the number of load applications for sections 4A and 4B of Test 4 (Reduced base) in different pass levels during the test. Test 4B ran until 70,000 load cycles due to the need to conduct an equipment maintenance (fixing the connection between the piston rod of the hydraulic cylinder and the wheel support). It can be seen in Figure 5.39a that control section (4A) exhibits a relatively high rate of rutting and heaves over time in comparison to other control sections (1A and 1B). This behavior is attributed to pavement reduced stiffness. An increasing initial rate of deformation up to 2,000 load cycles was verified with heaves alongside wheel path up to 4 mm. Also, the severe rutted surface resulted in microcracking at the edge of the wheel path, being an indirect indication of the high air voids in the asphalt layer (9.8%). Figure 5.40 presents the aspect of the rutted surface and microcracking at section 4A. Microcracking have appeared after 20,000 cycles, with no reflective cracking.

(a)

(b)

Figure 5.39. Rut depth measurements along transverse centerline of Test 4: (a) section 4A; (b) section 4B.

(a)

(b)

Figure 5.40. Asphalt surface of section 4A: (a) wheel path; (b) rutted surface and microcracking.

The second stage of the test (4B) also presented results of an initial densification phase before 2,000 load cycles (Figure 5.39b). However, due the greater stiffness of the reinforced overlay section, smaller levels of rut depths and heaves were noted, in comparison to the first stage of the test (4A). Also, microcracking appeared alongside wheel path after 80000 load cycles (Figure 5.40a), comparable to the observed in Test 1.

Figure 5.41 shows the accumulated surface vertical displacements with the number of wheel passes for both sections of Test 4. Maximum rut depth at centerline were recorded as 9.9 mm in control section, while for the second stage of the test, 5.5 mm were noted at 70,000 load cycles. Both higher initial densification and maximum observed rut depth levels were expected for in Test 4 due to the pavement reduced overall stiffness in comparison to other tests.

Figure 5.41. Accumulated surface vertical displacements at the centerline of Test 4: (a) section 4A; (b) section 4B.

5.5.2 Elastic Surface deflections

Figure 5.42 presents the deflection basin of both stages of Test 4 at 500, 1000, 10,000, 50,000 and 100,000 load cycles. Results presented a decrease in elastic deflections levels with increasing number of wheel load cycles for both sections. Figure 5.43 shows the final deflection basin of sections 4A and 4B.

Figure 5.42. Deflection basins for different load cycles of Test 4: (a) section 4A; (b) section 4B.

Figure 5.43. Final deflection basins at 70,000 load cycles of Test 4.

Figure 5.43 presents deflection results decreasing rapidly with distance from the point of loading for section 4A when compared to the overlay section 4B. An influence of the subgrade layer was also verified in the curve shape of control section. Maximum cyclic elastic deflections registered in both tests are presented in Figure 5.44. These deflections have decreased with the number of load cycles in section 4A. For the second stage of the test, a plateau trend was visualized, achieving 0.20 mm at the end of the test. The presence of an asphalt overlay reinforced with geogrid have reduced maximum elastic deflections in approximately 58% in comparison to control section.

Figure 5.44. Maximum cyclic elastic surface deflections in Test 4.

5.5.3 Dynamic stress response

Figure 5.45 presents a typical dynamic stress response for pressure transducers installed in the bottom of asphalt layer, middle of the base course and the top of the subgrade section 4A (after 50000 load cycles). Results are presented as an average of peak vertical stresses registered during this test. The pressure transducer installed in the asphalt layer presented results ranging from 100 a 320 kPa during the test. Base course pressure transducers results presented an average peak of 150 kPa. The subgrade pressure transducer have presented an average peak of 103 kPa during this test. Vertical scales in the plots are different for a better visualization of the signals recorded. Higher values of pressure transducers were registered in this test in comparison to the other tests due to the base course layer reduced thickness. Figure 5.46 presents average peak vertical stresses results registered during section 4B test at the bottom of asphalt layer 1 and at the top of the subgrade. The pressure transducers installed in the bottom of asphalt layer 2 and at the base aggregate course did not respond to traffic load.

Results show that the overlay has decreased dynamic stress levels of 320 kPa (Figure 5.45) at the bottom of asphalt layer 1 to 170 kPa (Figure 5.46). Subgrade pressure transducer presented average peak of 75 kPa during the test, while in the first stage of the test (before asphalt overlay), 103 kPa had been registered. Using an interpolation of results presented in both stages of Test 4 in asphalt and subgrade layers, a pressure transducer in the base course would have registered approximately 135 kPa.

(a)

(b)

(c)

Figure 5.45. Typical results of dynamic vertical stresses of section 4A: (a) bottom of asphalt layer; (b) middle of the base course; (c) top of the subgrade.

(a)

(b)

Figure 5.46. Typical dynamic vertical stresses of section 4B: (a) bottom of asphalt layer 1; (b) top of the subgrade.

5.5.4 Asphalt concrete strain response

Figure 5.47 shows the results of ASGs installed at the bottom of the asphalt layer of section 4A, considering an average peak results after 50,000 load cycles. The tensile strain results measured by ASG L shows values of approximately $760 \mu\epsilon$ as shown in Figures 5.47a, with a compression signal when wheel was approaching and then an extension (peak), as expected for this response. Figure 5.47b presents a single tensile peak of $1000 \mu\epsilon$ for ASG T. Vertical scales in the plots are different for a better visualization of the signals recorded. Figure 5.48 illustrates the accumulated plastic strains recorded by the ASGs installed in section 4A. The highest plastic and elastic strains were noted in the transverse strain gage.

Figure 5.47. Typical response of ASGs in section 4A: (a) ASG L; (b) ASG T.

Figure 5.48. Plastic strains response in asphalt concrete layer of section 4A.

In the second stage of Test 4, ASGs were installed in both existing and new asphalt layers. Figure 5.49 presents typical results of ASGs installed in the bottom of asphalt layer in, in both longitudinal and transverse directions, (after 50,000 load cycles). The average peak horizontal tensile strains measured by ASG 1L was 370 $\mu\epsilon$ and 380 $\mu\epsilon$ for ASG 1T. Figure 5.50 presents the results of ASGs installed in asphalt layer 2, presenting overall results of peak elastic strains levels of 300 $\mu\epsilon$ for ASG 2L and 330 $\mu\epsilon$ for ASG 2T.

Figure 5.49. Typical response of ASGs in section 4B: (a) ASG 1L; (b) ASG 1T.

Figure 5.50. Typical response of ASGs in section 4B: (a) ASG 2L; (b) ASG 2T.

Longitudinal strains recorded in both lower and upper layers showed a compression followed by an extension (tensile peak), while only tensile behavior was observed in transverse ASGs. In comparison to the first stage of the test, results demonstrated reduced horizontal tensile strains in asphalt layer 1 in at least 2 times. In this test, transverse elastic strains were greater than longitudinal elastic strains. Figure 5.51 illustrates the accumulated plastic strains recorded by the ASGs installed in section 4B. The highest plastic strain levels were noted in transverse and longitudinal strain gages installed in the upper layer.

Figure 5.51. Plastic strains response in asphalt concrete layers of section 4B.

5.5.5 Displacement along geogrid

Horizontal displacements along the geogrid monitored during Test 4 are presented in Figure 5.52. As observed in previous horizontal displacement analysis in Test 3, results indicates horizontal displacement's magnitude along the geogrid increasing with increasing number of load cycles, with all extensometers showing positive (tensile) readings during the test. LDSs under the wheel load (LDS 1 and LDS 2), symmetrically in the same position (Figure 4.33a), showed the highest level of displacements during the test (maximum 0.30 mm). The strains diminished at a distance away from the wheel load. Results of plastic and elastic horizontal displacement and the calculation of strains along the geogrid will be presented in Chapter 6 “Discussion of the Results”.

Figure 5.52. Permanent displacements data with the number of load cycles of Test 4.

6 ANALYSIS OF THE EXPERIMENTAL RESULTS

The focus of this chapter is the analysis on the performance of geogrid-reinforced pavement sections, particularly in comparison to unreinforced sections evaluated in the APT testing program. This chapter also contains the analysis of geogrid displacements and strains distribution under traffic loading, compared to the unreinforced asphalt layer interface.

6.1 Comparison between unreinforced and geogrid-reinforced asphalt overlay section

The pavement sections are compared in this chapter. Specifically, the performance of a geogrid-reinforced asphalt overlay with an unreinforced asphalt overlay section (Test 2 and Test 1). The performance of the two identical control sections (tested in the first stage of each test) will be compared in order to allow the comparisons of performances of overlay sections in the second stage of the tests (after overlay application).

The development of permanent deformations (rutting) on the pavement surface due to permanent vertical displacements was measured for both control sections (1A and 2A). Measured rutting profiles of 10^4 load cycles (after densification period) and 10^5 (final profile) of both experimental sections are compared in Figure 6.1. Similar permanent deformation responses can be observed for both sections, including maximum rut depths and heave alongside wheel path. This provides evidence of the consistency of the construction process as well as of the testing procedures adopted in this investigation.

Figure 6.1. Rut depth measurements along transverse centerline of sections 1A and 2A: (a) 10^4 load cycles; (b) 10^5 load cycles.

Figure 6.2a presents the accumulated surface vertical displacements at the centerline of control sections. A slight difference is seen in rate of rutting over time, with maximum rut depth of 6.4 and 6.1 mm, respectively for sections 1A and 2A, as well as equivalent heaves alongside wheel path. Overall, the differences are deemed negligible. Figure 6.2b shows a comparison of final deflection basin analysis, illustrating the similarity of pavement structure's stiffness.

Figure 6.2. Control section results (Test 1A and Test 2A): (a) accumulated surface vertical displacements at the centerline; (b) final deflection basin.

The data and analysis presented here certainly indicate that the results of control sections (1A and 2A) proved excellent repeatability of overall rutting and elastic deflection behavior, providing confidence in using the quality control measures. Since the same construction process and materials were used in these pavement sections, with experimental variables minimized as much as possible, it is therefore suitable to compare the performance of the second stage of Tests 1 and 2 (overlay sections).

Figure 6.3 presents the overall performance of accumulated vertical displacements of Tests 1 and 2. The development of rutting on pavement surface was plotted for the first stage of test (1A and 2A) and second stage (1B and 2B). The asphalt overlay has increased the pavement stiffness in both sections, leading to a reduction in the rate of rutting in the second stage of the test. However, inspection of results obtained during the second stages of the tests reveals that the most significant rutting over time was observed in the unreinforced section (when compared to reinforced overlay section 2B). The development of permanent deformation could not be evidenced to have reached final values. Figure 6.4 shows measured rut profile along transverse centerline and relative vertical displacements during tests of reinforced and unreinforced sections. It is clear that the reinforcement function of the geogrid was effective in

reducing maximum rut depth development during the test, as well as providing heaves resistance.

Figure 6.3. Accumulated surface vertical displacements at the centerline of Test 1 and Test 2.

Figure 6.4. Comparison of sections 1B and 2B: (a) rutting profiles at 10^5 load cycles; (b) relative vertical displacements.

As seen in Figure 6.4a, significant reduced heaves adjacent to ruts could be evidenced in section 2B. At the end of the test, the inclusion of the geogrid in the asphalt layer had promoted approximately 36% reduction in maximum rut depths. Similar results of rutting quantification in geogrid-reinforced asphalt overlays were reported by Brown et al. (1985). Specifically, they reported that the inclusion of a polymeric geogrid in asphalt layer has reduced rut depths by 20 to 58%. Montestruque et al. (2004) have showed results of plastic deformation in asphalt beams reinforced with polyester geogrid with a reduction between 30 and 36%. An

experimental study conducted by Laurinavičius and Oginskas (2006) with different types of geogrids have reported over 50% less rutting.

In order to minimize the effect of variation in the air voids content of asphalt layers, maximum rut depths results were plotted as measured relative vertical displacements versus the number of wheel passes after the first 10,000 load cycles (excessive deformation period). Figure 6.4b shows relative vertical displacements analysis of sections 1B and 2B. Comparing the second stage of tests (overlay sections), results indicated that there was a fundamental difference between geogrid-reinforced and unreinforced sections, enhancing pavement lifetime. The geogrid-reinforced section has shown minor relative vertical displacements after 10,000 passes.

The traffic benefit ratio (TBR) is defined as the number of cycles to reach a particular permanent surface deformation for a reinforced test section, divided by the number of cycles to reach this same deformation in an unreinforced test section. Figure 6.5 shows the TBR plotted against permanent surface of unreinforced (1B) and geogrid-reinforced (2B) sections. Based on maximum rut depth measured at the end of Test 2B (3.1 mm rut depth), TBRs up to 4.4 were obtained in comparison to unreinforced overlay section. Thus, TBR results have indicated increase in pavement's life improvement during the test of over four times greater when using geogrid asphalt reinforcement.

Figure 6.5. Traffic benefit ratio of Test 2 in comparison with Test 1.

Figure 6.6 shows a comparison of the final deflection basin of reinforced and unreinforced pavement sections (1B and 2B). Results illustrated an increase in pavement structure's stiffness of reinforced section observed by the deflation basin curve shape, since

deflections have decreased gradually with distance from the point loading. The effect of geogrids on reducing the elastic displacements occurred most significantly around the vicinity are of the wheel load. Figure 6.7 presents an overall comparison of maximum elastic displacements (close to the point loading) during Tests 1 and 2. Although both overlay sections show comparatively smaller deflections than the control layers at the end of the tests, the inclusion of a geogrid in the asphalt layer (section 2B) provided a reduction in pavement deflections in at least 2.5 times, enhancing pavement structural response. It is clear that the stabilization of elastic displacements have occurred in both sections in the same period; however, a significant reduction is noted in the section 2B, which is attributed to the geogrid contribution.

Figure 6.6. Final deflection basin of Test 1 and Test 2.

Figure 6.7. Maximum elastic deflections during Test 1 and Test 2.

Typical dynamic responses as observed using the pressure transducers installed in pavement layers of sections 2A and 2B are compared in Figure 6.8 (after 50,000 load cycles). Stress response from APT tests showed a marked difference between reinforced and unreinforced sections. An average of 32% reduction in dynamic vertical stresses at the bottom of asphalt layer 1 was observed in the geogrid-reinforced section (Figure 6.8a), which led to a reduction of vertical strains underneath wheel load. Stress measurements at the base course confirmed this benefit, since a reduction of 29% was observed (Figure 6.8b). Dynamic vertical stresses induced at the top of the subgrade were reduced by 36% due to the presence of reinforcement in the asphalt layer. These results are consistent with those reported by Siriwardane et al. (2010), who concluded that a fiber glass geogrid installed within the asphalt layer spread a circular load over a larger area in the lower layers of a pavement section decreasing to vertical subgrade stresses. These results provide strong evidence that geogrids in asphalt layers act as a reinforcement, providing structural benefits to the pavement. Figure 6.9 shows an example of average peak dynamic stresses time-history throughout tests, the standard deviations results and coefficient of variation. The level of deviation is negligible considering the level of strains acting in pavement structure. The results show that pavement layer stress measurements in this research were made with a very high degree of repeatability and reliability.

(a)

(b)

(c)

Figure 6.8. Typical dynamic vertical stress response of sections 1B and 2B: (a) at the bottom of asphalt layer 1; (b) at middle of the base course; (c) at the top of the subgrade.

(a)

Figure 6.9. Dynamic stress response of sections 1B and 2B as a function of wheel load passes: (a) at the bottom of asphalt layer 1; (b) at the top of the subgrade.

Table 6.1 presented results of average peak horizontal tensile strains recorded by the ASGs during Test 1 and Test 2. The tensile strain values reported in Table 6.1 highlights more than 50% reduction in longitudinal strains in the upper asphalt layer of geogrid-reinforced section with respect to unreinforced section. Transverse strains analysis have shown a reduction of 42% after the geogrid inclusion in the pavement structure. The reduction in transverse strains in the asphalt layer show the potential of this technique in reducing fatigue cracking during pavement service life time.

Table 6.1. Average peak horizontal tensile strains for Tests 1 and 2

Test/ Section	Longitudinal strains ($\mu\epsilon$)	Transverse strains ($\mu\epsilon$)
	ASG 2L	ASG 2T
1B	280	200
2B	125	115

Figure 6.10 provides information on the plastic strain response of AC layers of reinforced and unreinforced sections, showing the reduction in accumulated plastic strains at the upper layer due to the presence of the geogrid, mainly in transverse direction.

Figure 6.10. Plastic strains response in asphalt concrete layer of sections 1B and 2B: (a) ASG 2L; (b) ASG 2T.

Therefore, it is reasonable to assume that most of the differences in the performance of pavement sections are due to the intrinsic properties of the polymeric geogrid composite and its interaction characteristics with asphalt layers. Based on rutting measurements, the lifespan of a geogrid-reinforced asphalt overlay was found to be over four times longer than that of an unreinforced asphalt overlay. The results show that the polymeric geogrid reinforcement has effectively reduced the shear deformation and heaves of the asphalt surface course. Measurements obtained from APT test sections showed a significant difference in the stresses measured over the subgrade between the geogrid-reinforced and unreinforced sections. Specifically, an average reduction of 32% in the dynamic vertical stress at the bottom of the asphalt layer and 29% in the base course were observed when using geogrid reinforcement.

Concerning the reduction in the dynamic vertical stresses at the top of the subgrade, an average of 36% reduction was obtained. The reinforcement mechanism of PVA geogrid within the asphalt layer could be quantified by the instrumentation placed in the test sections. These mechanisms show a reduction in permanent vertical deformation (rutting), elastic deflections and a greater reduction of the vertical stresses in the pavement layers. Additionally, the reduction over 50% in the longitudinal elastic strains in the asphalt layer of the geogrid-reinforced section and the reduction of 42% in transverse elastic strains (upper layer) show the potential of this technique in reducing fatigue cracking during the pavement lifespan. Differences between reinforced and unreinforced sections tend to be more pronounced for dynamic stress/strains than for plastic strain response, thus contributing to less cumulative fatigue damage.

6.2 Comparison between single geogrid-reinforced HMA layer section and overlay sections

The comparison of pavement sections presented in this chapter assesses the structural benefits of overlay sections compared to a geogrid reinforcing new asphalt layer section. Results will be compared using data from the second stage of Test 1 (Unreinforced section) and Test 2 (Baseline reinforced section) with Test 3 (Single reinforced HMA layer), in which a geogrid reinforcing newly asphalt layers was tested.

The development of rutting on overlay sections were plotted for unreinforced section (1B) and geogrid-reinforced section (2B) and compared to Test 3. All three pavement sections have the same asphalt layer thickness (110 mm), as well as the same base course and subgrade characteristics. Figure 6.11 presents the performance of accumulated vertical displacements of overlay sections and geogrid reinforcing new asphalt layer section.

Figure 6.11. Accumulated surface vertical displacements at the centerline of Test 1, Test 2 and Test 3.

Comparing the overlay pavement sections with a reinforced new asphalt layer in Figure 6.11, a decrease in rate of rutting over time was observed. This result was somewhat expected since Test 3 section had not been previously tested, not having accumulated permanent deformation in the pavement structure. However, it is notable a more significant reduction in pavement rutting using this technique, in comparison to a conventional overlay system. The Test 3 section rutting evolution curve presented in Figure 6.11 is consistent to a rutting evolution of a pavement with adequate structural design in which the deformation occurs primarily through densification period, and the curve tends asymptotically towards the end of the test, stabilizing plastic deformations. The same behavior was observed for geogrid-

reinforced asphalt overlay section (Test 2B), however with larger rut depths. Results show that the unreinforced section represented a pavement with inadequate structural and material design specifications in which plastic flow dominates. The deformation trends observed in Tests 2 and 3, in comparison to Test 1, highlight the differences on the influence of permanent deformation mechanisms. The same trend in rutting evolution was found by Siriwardane et al. (2010) with a new asphalt layer of a flexible pavement reinforced with a fiberglass geogrid (Figure 2.29).

Figure 6.12 shows rut depth measurements along transverse centerline at the end of the tests 1, 2 and 3. Test 3 showed a market different in comparison to unreinforced section of Test 1B. Heaves on surface slightly occurred in both geogrid-reinforced sections, showing that the inclusion of a polymeric geogrid in this pavement structure have provided resistance to heaves. Given the significance of asphalt air-void volume values on overall behavior of all sections, it is believed reinforced sections would have performed better if had AC layer air-void volume values closer to the expected field compaction values of 4%.

Figure 6.12. Rut depth measurements along transverse centerline of Test 1B, Test 2B and Test 3.

For a better comparison between permanent deformation performance in Tests 2 and 3, the effect of air void contents in AC layers were minimized by rut depths results plotted as measured relative vertical displacements (Figure 6.13), after 10,000 load cycles. Comparing both geogrid-reinforced sections, results indicated that there was an equivalent behavior between relative vertical displacements presented in these sections. Geogrid-reinforced sections have shown similar minor relative vertical displacements after 10,000 passes, likely attributable to the stabilization effects from the geogrid. Thus, quite similar behavior of both geogrid-reinforced sections could be evidenced in this analysis, with superior response for Test 3.

Figure 6.13. Relative vertical displacements of Test 1B, Test 2B and Test 3.

Comparing final deflection basin and maximum measured elastic displacements during the tests, results in Figure 6.14 illustrate the difference in pavement stiffness of Tests 1, 2 and 3. Deflections have progressively decreased with distance from the point loading for Test 3 (Figure 6.14a), with levels of maximum elastic deflections approximately 70% reduced in comparison to unreinforced section (Figure 6.14b). Also observed in Test 3, the early stabilization of elastic deflections may positively imply an extension in fatigue life of the asphalt mix. In general deflection basin curves have been significantly affected by the inclusion of PVA geogrids in AC layers of Tests 2 and 3. Based on observations of the testing and data analysis, the measured rutting and deflection basin results provided important evidence of the effectiveness of a geogrid reinforcing a new asphalt layer in comparison to a conventional overlay technique in increasing pavement performance.

Figure 6.14. Elastic displacement results of Test 1B, Test 2B and Test 3: (a) deflection basin; (b) maximum elastic deflections.

Table 6.2 presented results of peak horizontal tensile strains recorded by the ASG during tests 1B, 2B and 3. The proposed pavement structure of a single geogrid reinforced HMA layer demonstrated results of horizontal tensile strains around 100 $\mu\epsilon$ (average peak values). This level of tensile strains may potentially reduce the initiation of fatigue cracking during pavement service life-time. As demonstrated by results presented in reinforced overlay section (2B), the average peak horizontal tensile strains presented in Test 3 section also reduced more than 50% in longitudinal strains in comparison to unreinforced section. Transverse strains analysis have shown a reduction of 50% in lower and upper layers, contributing to less cumulative fatigue damage in pavement structure. Graziani et al. (2014) also tested a new double-layered asphalt surfacing reinforced with geogrids, showing overall 65% reduction in peak strains inside the reinforced section, in comparison to an unreinforced section. In terms of plastic strain response (Figure 6.15), the levels of plastic strains in AC layers of Test 3 are very similar to the results presented in Test 2. This shows that the geogrids reduce plastic deformations in the asphalt layer of this flexible pavement structure. Results of ASGs installed in the lower asphalt layer of Test 2 will not be presented since no instrumentation was installed in the pavement section.

Table 6.2. Average peak horizontal tensile strains for Tests 1, 2 and 3

Test/ Section	Longitudinal strains ($\mu\epsilon$)		Transverse strains ($\mu\epsilon$)	
	ASG 1L	ASG 2L	ASG 1T	ASG 2T
1B	410	280	270	200
2B	-	125	-	115
3	110	100	100	100

(a)

(b)

Figure 6.15. Plastic strains response in asphalt concrete layers of Test 1, Test 2 and Test 3: (a) ASG 1L; (b) ASG 2L; (c) ASG 1T; (d) ASG 2T.

Table 6.3 presents an overall result of dynamic stress response obtained in pavement sections of Tests 1, 2 and 3. Results are consistent with those expected for pavement structures, with higher stress levels in the upper pavement layers. The presence of the geogrid in Tests 2 and 3 resulted in rapid dissipation of with depth in comparison to the unreinforced pavement structure. In Test 3, the most significant reduction was recorded by the subgrade pressure transducers, with 60% of the vertical compressive stresses registered in Test 1. The use of geogrids to reinforce new asphalt layers resulted in low stress levels in the base course and at the bottom of asphalt layer, with approximately 50% of the values registered in the unreinforced overlay section test. These stress results suggest that the use of a polymeric geogrid as reinforcement in the new asphalt layer effectively reduced the shear deformation and heave of the asphalt surface course. These results indicate that use of geogrids to reinforce an asphalt overlay present structural benefits to the pavement. When the load is applied to the surface of the pavement, a zone of tension develops at the lower section of the AC layer. The tensile stress acts in the asphalt concrete and is transferred to the geogrid as tensile force, minimizing the lateral movement and improving the compression and flexural stiffness of the pavement layers. The mobilized tensile forces in the geogrids explain the correlation with the reinforced pavement performance in resisting the pavement deformation. Additionally, a possible improvement in the long-term performance was evidenced with the results presented by Tests 2 and 3, especially regarding the potential of mitigating the development of fatigue cracks.

Table 6.3. Average peak dynamic vertical stresses for Tests 1, 2 and 3

Test/Section	Bottom of asphalt layer 2 (kPa)	Bottom of asphalt layer 1 (kPa)	Middle of the base course (kPa)	Top of the subgrade (kPa)
1B	-	152	60	30
2B	-	104	42	20
3	160	75	35	12

Findings of this research were based on tests with a PVA geogrid combined with an attached PP nonwoven geotextile. This specific geogrid provided adequate support for this particular pavement structure. However, additional research is needed to verify the potential use of geogrid reinforcement in asphalt layers, with different stiffness and materials types, tack coat types and rates of application. The observations and correlations obtained in this research are limited to the type of pavement structure and materials used.

A significant number of variables can be controlled in accelerated testing, including geogrid and soil types, base layer aggregate gradation and air voids of asphalt layers, among others. Nevertheless, the results in the present study are consistent with studies presented in the literature review “Chapter 2”, which mentions that the use of geogrids as asphalt layer reinforcement can provide structural benefits to flexible pavements.

Further studies could be beneficial to correlate geogrid index properties and their interaction characteristics with the asphalt layers or the pavement structure. Finally, it is important to observe that the success of this technology depends not only on the tensile properties of the geosynthetic, but also on its proper selection, positioning and installation in the asphalt layer.

6.3 Comparison between unreinforced section and geogrid-reinforced asphalt overlay section with reduced base course thickness

The pavement section in Test 4 (Reduced base) aims at evaluating the structural benefits of a geogrid-reinforced asphalt overlay pavement section with reduced base course thickness (100 mm) in comparison to an unreinforced asphalt overlay pavement section with 200 mm of the same base course, respectively Tests 4 and 1. These two tests involved a control section in the first stage of the test, before placement of asphalt overlay.

The development of permanent deformations (rutting) on the pavement surface due to permanent vertical displacements was measured for both control sections 1A and 4A. A comparison on the rutting evolution at the centerline for both control sections is presented in Figure 6.16a and the final rutting profiles for 10^5 cycles are compared in Figure 6.16b. The results in this figure show that rut profile of the reduced base course section (4A) is significant different in comparison to control section 1B, as expected for a pavement with reduced overall stiffness. Heave levels alongside the wheel path on the surface of section 4A showed twice of the values presented at section 1B. The difference in rate of rutting over time was noticed during tests, with maximum rut depth of 6.1 and 9.9 mm, respectively for sections 1A and 4A (Figure 6.16a). However, none of the sections presented stabilization of plastic deformations before 10^5 load cycles.

Figure 6.16. Comparison of rutting development of section 1A and 4A: (a) rutting evolution at the centerline; (b) final rutting profile at 10^5 load cycles.

Figure 6.17 illustrate similar deflection basin curves in the first stage of the tests. In the second stage, deflections have progressively decreased with distance from the point loading in section 4B (Figure 6.17a), showing the influence of a geogrid in increasing the overall pavement stiffness. The maximum elastic deflections have reduced approximately 40% in comparison to unreinforced section (Figure 6.17b). These results provide evidence of the effectiveness of geogrids to reinforce a pavement structure with 100 mm base course thickness. Test 4 ran until 70000 load cycles due to the need to conduct an equipment maintenance.

Figure 6.17. Elastic displacement results of Test 1 and Test 4: (a) deflection basin; (b) maximum elastic deflections.

Figure 6.18 presents the overall performance of accumulated vertical displacements of Tests 1 and 4 (first and second stage of test). The development of rutting on pavement surface was plotted for the first stage and second stage of Tests 1 and Test 4. The asphalt overlay reinforced with geogrid resulted in an increase in pavement stiffness in comparison to unreinforced overlay, even with half of the base course thickness, leading to a decrease in the rate of rutting in the second stage of the test. However, observing the second stage of test, no substantial difference on rutting over time was noted between sections, although it is evident that the reinforcement function of the geogrid was effective in reducing rut depth development when compared to previous damaged level of the pavement (before overlay). The initial densification (20000 load cycles) in the second stage of the test was also reduced in section 4B in comparison to the first stage (Figure 6.18).

Figure 6.18. Accumulated surface vertical displacements at the centerline of Test 1 and Test 4.

Figure 6.19 presents rutting profiles of both sections for 70000 load cycles in the second stage of the test and relative vertical displacements after 20000 load cycles. The reinforcement inclusion in Test 4 was capable of reducing rutting and heaves to the same levels presented in Test 1 (Figure 6.19a). Figure 6.19b shows the relative vertical displacements analysis for sections 1B and 4B, demonstrating similar evolution in terms of rate of running over time after overlay application. Figure 6.20 illustrates the aspect of the asphalt surface at the end of Test 4 showing no significant rutting and heaves along wheel path. Microcracking have appeared alongside wheel path after 60000 load cycles. An important lesson learned is that a geogrid reinforcing the asphalt overlay of a 100 mm thick base course can be as beneficial as an unreinforced asphalt overlay pavement with a base course twice as thick (Table 4.8).

Figure 6.19. Comparison of sections 1B and 4B: (a) rutting profile at 50,000 load cycles; (b) relative vertical displacements.

Figure 6.20. Asphalt surface of section 4B at the end of the test.

Results of dynamic stresses in the first stage of Test 4 presented the highest levels of stresses observed in pavement layers of all of the tests conducted in this study, attributed to pavement configuration. Results of 150 and 103 kPa respectively for base course and subgrade were registered in section 4A, compared to averages of 90 and 47 kPa in section 1A.

Figure 6.21 presents typical results of dynamic stresses of the second stage of Test 4, compared to the second stage of Test 1. Stress response measures from APT tests have shown similar average peak values between reinforced and unreinforced sections at the bottom of asphalt layer 1 (Figure 6.21a). At the top of the subgrade, results presented in Figure 6.21b showed higher induced stresses at section 4B in comparison to results of unreinforced section 1B.

Figure 6.21. Typical dynamic vertical stress response of sections 1B and 4B: (a) at the bottom of asphalt layer 1; (b) at the top of the subgrade.

The difference in dynamic stresses registered in the subgrade layers of these tests can be attributed to the reduced distance of stress dissipation in the base layer of section 4B in comparison to that of section 1B with base course twice as thick. It can be stated that a base layer with thickness twice of 4B section layer resulted in a stress reduction of approximately 50%, which is clear in Figure 6.21b. The pressure transducer installed in the bottom of asphalt layer 2 and at middle of the base course did not respond to traffic load during the test.

Table 6.4 presents results of average peak horizontal tensile strains recorded during test sections 1B and 4B. Results demonstrate the beneficial effect of a geogrid inclusion in this pavement structure since strains in longitudinal direction at lower asphalt layer have decreased in comparison to unreinforced section. At upper layer longitudinal strains have slightly increased in comparison to unreinforced section. In the transverse direction, both results presented 40% higher strain levels in section 4B. Figure 6.22 presents results of plastic strains developed in AC layers of both sections. In general, results showed very similar evolution of plastic strains in the reinforced and unreinforced asphalt layers.

Table 6.4. Average peak horizontal tensile strains for Tests 1 and 4

Test/Section	Longitudinal strains ($\mu\epsilon$)		Transverse strains ($\mu\epsilon$)	
	ASG 1L	ASG 2L	ASG 1T	ASG 2T
1B	410	280	270	200
4B	370	300	380	330

(a)

(b)

Figure 6.22. Plastic strains response in asphalt concrete layers of sections 1B and 4B: (a) ASG 1L; (b) ASG 2L; (c) ASG 1T; (d) ASG 2T.

The results presented in this chapter show the importance of investigation of pavement structural performance with the use of geogrids within asphalt overlays. Besides preventing fatigue or reflective cracking, geogrid reinforcing asphalt layers may be an option when designing flexible pavement structures to prevent rutting, heave and less induced stresses and strains in pavement layers. Based on rutting and deflection basin measurements, the lifetime of a geogrid-reinforced asphalt overlay was found to be comparable to an unreinforced asphalt overlay with a base course that is twice as thick.

All tests have shown that this particular polymeric geogrid composite reinforcement could effectively reduce the shear deformation of the asphalt surface course, considering the materials and compaction conditions studied in this research. The reinforcement mechanisms identified here include the reduction of horizontal tensile strains, reduction of vertical stresses and the reduction of dynamic and permanent vertical strains. However, further studies are required to show the performance for more than 10^5 load cycles. Also, additional research is needed to establish a threshold where the geogrids no longer influence the performance behavior. The use and specification of geogrids within asphalt overlays to improve pavement structural performance represent a significant cost effective opportunity to reduce the use of granular pavement materials, as well as extending pavement service life. The investigation presented in this work is expected to serve as a solid base for further works on pavement rehabilitation with the use of geosynthetics.

6.4 Analysis of displacement and strain distributions

In order to better understand the reinforcement mechanisms in asphalt overlays with the use of geosynthetics, the development of a strain distribution in geogrids reinforcing pavement sections will be evaluated. This information is necessary to gain understanding on the pavement response, which will then allow the development of methodologies to predict reinforced pavement responses. Thus, mechanisms of asphalt overlay reinforcement will be explored and the effects of a geosynthetic in tensile reinforcing asphalt overlays will be investigated. According to Perkins (1999), an evidence of lateral restraint is provided by the development of geosynthetic tensile strains. Thus, reinforcement displacement and strain distribution data from Tests 3 and 4 will be presented and compared to internal displacements recorded at the interface of old and new asphalt layers of Test 1, without geogrid reinforcement.

Figure 6.23 illustrates a profile of permanent horizontal displacements obtained during Test 1 (Unreinforced overlay) and Test 3 (Single reinforced HMA layer) for increasing number of load cycles. This investigation indicates, as presented in Chapter 5, internal displacements magnitude increasing with increasing number of load cycles for both sections. Using the same scales, results of reinforced sections show a slight increase up to the end of the test, compared to unreinforced section. All LDSs showed positive (tensile) readings during the tests 1 and 3. Much smaller displacements were recorded in reinforced section, which can be attributed to geogrid tensile property and the bond stiffness between geogrid and asphalt layers. Figure 6.24 illustrates the displacement distribution with distance from wheel loading, showing maximum displacements observed in the vicinity area of the wheel and reduction of displacements within the distance of the wheel load.

Figure 6.23. Permanent horizontal displacements data with the number of load cycles: (a) Test 1; (b) Test 3.

Figure 6.24. Displacement versus distance from the wheel load: (a) Test 1; (b) Test 3.

In general, horizontal internal displacements recorded at each load level in the reinforced section were smaller for each load cycle compared with unreinforced section. Maximum displacements in unreinforced section were approximately 0.55 mm, while the reinforced asphalt layer showed 0.22 mm of maximum values.

As previous described in Chapter 4, a uniform strain was defined between two points. Then, the average tensile strain response mobilized between two extensometers (transverse direction) is calculated based on direct measurements of internal horizontal displacements. Figure 6.25 illustrates a profile of permanent strains developed during Test 1 (Unreinforced overlay) and Test 3 (Single reinforced HMA layer) for increasing load cycles. The distributions of strains have presented a similar pattern to rutting profiles, showing only tensile strains (Figure 6.25).

Figure 6.25. Permanent strains data with the number of load cycles: (a) Test 1; (b) Test 3.

Within the asphalt layer interface of unreinforced section, maximum strains were developed in the range 2100 $\mu\epsilon$ (0.21% strain) in Test 1, as indicated in Figure 6.25a. In Test 3, geogrid reinforcement within the asphalt layers showed maximum total strains of 1000 $\mu\epsilon$ 0.1% (Figure 6.25b). The location of the maximum peak strains did not change with increasing number of load cycles for both sections, being more significant under the wheel load (zero reference in Figure 6.25).

The strain distributions along the geogrid and unreinforced asphalt interlayer are shown in Fig. 6.26. Results showed that the reinforcement mechanism for this application is the reduction of lateral movement of the asphalt layer by the development of shearing resistance with the geogrid. Results show the cumulative development of permanent strain directly beneath the load in the transverse direction of the path layer until the complete mobilization of the geogrid (after 30,000 cycles). Significant tensile strains were observed only in the vicinity of the wheel, until 300 mm from the load center.

Figure 6.26. Permanent strain versus distance from the wheel load: (a) Test 1; (b) Test 3.

Moreover, the unreinforced test sections exhibit much higher strains than the reinforced section, illustrating the ability of the geogrid to minimize lateral movement of the asphalt layer. In Test 3, the deformation was greatly restrained because of the presence of geogrid reinforcement. The restraining effect of geogrid was evidenced by the strains developed in the geogrid around the vicinity of the loading area. These are evidences that lateral restraining mechanisms effect on the performance of the HMA. After the tensile mobilization by the geogrid, no significant increase on the measured displacements were noticed. Also, no increase in plastic strains was detected. The opposite situation is seen for the unreinforced section, where the permanent displacements and strains maintain increasing up to the end of the test.

Strains measurement also allowed calculation of tensile forces developed within the PVA geogrid during the test. Considering unconfined tensile strength tests, (ASTM D 6637), plastic strains of $1000 \mu\epsilon$ (0.1%) show that dynamic loads as high as 1.0 kN/m were developed in the PVA geogrid in Test 3, corresponding to a tensile stiffness of 900 kN/m.

Confronting data of strains developed in the transverse section of the geogrid (under the wheel load) and at the interface of unreinforced section with transverse ASGs installed between AC layers (ASG 2T), similar trend in plastic strains evolution were recorded during Test 1 and Test 3 (Figure 6.27). These results illustrate comparable measurements, suggesting the potential use of tell-tails system in measuring plastic strains in geosynthetic reinforcement of asphalt layers.

Investigations on displacements recorded by the extensometers under the wheel load in Test 3s indicated elastic displacements occurring during the test. Figure 6.28 presents the displacements evolution (LDS 1 and LDS 2) at different times during Test 3. The vertical scale was plotted differently for a better visualization of recorded data. Figures 6.28a and 6.28c present elastic and plastic strains evolution at the beginning of the test, while Figure 6.28d shows no development of plastic strains at the end of the test, as previously shown. Figure 6.29 presents displacements amplitude at the beginning and at the end of the test, providing an example of the constant of plastic strains observed after 30,000 load cycles.

Figure 6.27. Permanent strains in geogrid (under the wheel load) and asphalt concrete layer strains: (a) Test 1; (b) Test 3.

Figure 6.28. Displacements data of Test 3: (a) displacements readings with wheel load (b) elastic displacements signal; (c) total displacements at the beginning of the test; (d) total displacements at the end of the test.

Figure 6.29. Displacements data of LDS1 and LDS 2 of Test 3: (a) beginning of the test; (b) end of the test.

Figure 6.30 presents the results of elastic strains developed at the end of Test 1 and Test 3 using results of LDSs 1 and 2 (under the wheel load). Based on displacements data, the elastic

strains of Test 3 could be predicted, presenting constant peak values of $100 \mu\epsilon$ (Figure 6.30a). The strain analysis based on other LDSs have recorded similar behavior in elastic strains during test. Nguyen et al. (2013) reported a study in which elastic strains along a fiber glass geogrid in the AC layer of a pavement structure were investigated. In this study, peak tensile strains in the geogrid in the range of $200 \mu\epsilon$ (longitudinal) and $300 \mu\epsilon$ (transverse) were found, however using strain gages.

In general, a reduction of lateral movement of the asphalt layer was observed through the development of shearing resistance with the geogrid (plastic behavior). After a certain period, constant plastic and elastic strains develop in the geogrid, proving the mechanism is not based on tensioned, membrane-type reinforcement, since plastic strains accumulation remained constant (Figure 6.30a). In contrast, plastic strains in Test 1 section continued increasing up to 10^5 load cycles (Figure 6.30b), with an average peak values of elastic strains of $180 \mu\epsilon$. Comparing data of elastic strains developed in the AC layer by ASGs 2T in both tests with strains measured using tell-tales, comparable results were recorded during Test 1 and Test 3 (Table 6.2), showing the potential of tell-tales technique in predicting reliable strains response.

Figure 6.30. Elastic strains developed under the wheel load at the end of: (a) Test 3; (b) Test 1.

Findings of this study show the importance of studying the reinforcement mechanism of asphalt overlays with the use of geogrids. Several studies have reported the use of very stiff geogrids in asphalt reinforcement; however, tensile stress mobilized in the studied polymeric geogrid was shown to be in very small. Unfortunately, displacement and strains distribution analysis could not be evaluated in Test 2.

Test 4 was also conducted with the transverse section of the geogrid monitored by tell-tails. Figure 6.31 presents the results of displacement and strains along the geogrid of Test 4.

The same pattern of displacement and strain distributions of Tests 1 and 3 were observed in Test 4, emphasizing the proposed investigation. Maximum plastic strains developed in Test 4 were in the range of 1250 $\mu\epsilon$ (0.125% strain), as indicated Figure 6.31b. Compared to unreinforced overlay section (Figure 6.26a), strains magnitudes are seen to be smaller in Test 4, even with reduced base course thickness.

Figure 6.31. Analysis of strains along the geogrid of Test 4: (a) displacement distribution; (b) strain distribution.

Figure 6.32 shows geogrid permanent strains data of Test 4. The development of plastic strains have reached a plateau after mobilization (Figure 6.32a), as also presented in Test 3 (Figure 6.25b). Plastic strains measurement allowed the calculation of tensile forces developed within the PVA geogrid during Test 4. Considering unconfined tensile strength tests, (ASTM D 6637), maximum plastic strains of 0.125% show that dynamic loads as high as 1.35 kN/m were developed in the PVA geogrid in Test 4, corresponding to a tensile stiffness of 1080 kN/m.

Comparable records of AC plastic strains using tell-tails and ASG 2T in Test 4B were also observed, as presented in Figure 6.32b. Ling and Liu (2001) presented similar results of plastic strains develop in a polymeric geogrid, installed between an asphalt layer and a base course, increasing with increasing load cycles, showing that effective bonding between the polypropylene geogrid and asphalt material allowed load transfer.

Figure 6.32. Permanent strains data of Test 4: (a) geogrid strains with the number of load cycles; (b) comparison with asphalt concrete layer.

In terms of elastic strains measured by the displacement analysis in geogrid during Test 4, constant elastic strains in the range of 200 $\mu\epsilon$ (peak values) were found, which is comparable to the results recorded at the interface of asphalt layers of Test 1 (180 $\mu\epsilon$). Comparing data of elastic strains developed in the AC layer of section 4B by ASG 2T with the strains measured using tell-tales, results of peak elastic strains of 200 and 330 $\mu\epsilon$ were respectively found, showing a comparable correlation of strains response.

Similar magnitude and location of the reinforcement maximum peak strains were also obtained in all the pavement sections tested in this study, even though sections had different reinforcement layout and different pavement structures. Figure 6.33 resumes the results of permanent strains versus distance from the wheel load centerline, at load cycle 70,000 for Tests 1, 3 and 4.

Figure 6.33. Permanent strain versus distance from the wheel load centerline, at load cycle 70,000.

This load-cycle level was chosen because, after this level, Test 4 was inoperable. These results illustrate a pattern of strain development similar to that shown in Perkins (1999) for geotextiles and geogrids reinforcements of base aggregate in flexible pavement structures. Chen et al. (2009) also conducted a study to investigate the strain distribution along the geogrid as base reinforcement in pavement on weak subgrade under cyclic plate load testing. Similar pattern in strains distribution along the geogrid were found, showing significant permanent tensile strains developed in geogrids under the wheel load. Liu and Ling (2001) also presented results of strain distribution in geogrids, however installed between asphalt and base course layer, showing the same pattern of strain distribution.

Results of cyclic moving wheel load tests using the proposed APT facility indicated that PVA geogrid reinforcement contributed to an increase in the stiffness and strength of the asphalt layer. Pavement rutting and elastic deflection were reduced when compared with an unreinforced section. A mobilized length of 350 mm was evidenced in geogrid-reinforced overlay sections, showing that the bonding between geogrid and asphalt layers and the stiffness of the geogrid ensured a satisfactory performance in section tests. Thus, the development of strains in the present study shows that tensile reinforcement in asphalt overlays can be credited to lateral restraining mechanism.

Additional research is needed to generalize these findings for other reinforcement materials and pavement structures. The displacement and strain profiles may change for other types of geogrids. The behavior of geogrid reinforcement depends not only on the polymer or type of geogrid, but also on their tensile stiffness, position in the asphalt layer, tack coat type and rate, as well as traffic load magnitude and pavement structure condition.

7 CONCLUSIONS

7.1 Summary and main conclusion

This research aimed at understanding the benefits on the performance of flexible pavements of using geogrids within asphalt overlays. The study also investigates the tensile-strain response of the geogrid under cyclic moving wheel loads. The generated experimental information allowed the understanding of the mechanism developed in geosynthetic-reinforced asphalt overlays. A new accelerated pavement test facility was developed for the purpose of examining the mechanical behavior of geosynthetic-reinforced asphalt overlay system. Displacements along the geogrid specimens were measured using a tell-tail system. Strains along the geogrid were calculated using measured displacements data. The APT facility consists of a large steel testing box, in which field-scale pavement layers could be constructed. Materials included subgrade soil, aggregate base, PVA geogrid composite and asphalt concrete. Pavement sections included geogrid-reinforced and unreinforced asphalt overlay sections, a pavement section in which a geogrid reinforces the new asphalt layer, and a geogrid-reinforced asphalt overlay with reduced base course thickness. Pavement system performance was assessed by applying a cyclic moving traffic wheel loads of 700 kPa to the pavement surface. A set of sensors was used to measure stresses, asphalt concrete strains, geogrid displacements, surface plastic and elastic displacements, and induced traffic loads. Based on the results obtained from this research, the following conclusions can be drawn:

- The accelerated pavement testing equipment, which was specifically designed and constructed as part of this study, was found to have attended the design requirements in both stationary and moving wheel load modes.
- The data analysis presented in this study indicate that the pavement control layers in the different pavement sections showed excellent repeatability after cyclic loading. Specifically, they showed similar overall rutting behavior, which provides confidence on the quality control measures adopted in the experimental components of this study.
- The overall results of pavements sections demonstrated that significant improvement in pavement performance results from the inclusion of a polymeric (PVA) geogrid reinforcement in the asphalt overlay. This includes decreased permanent surface deformation (rutting) and elastic deflections, reduced load-

induced vertical stresses in the pavement layers and smaller strains in asphalt concrete layers. The results showed that the polymeric geogrid reinforcement effectively reduced rutting and heave of the asphalt surface course (over four times longer). Differences between geogrid-reinforced and unreinforced sections were found to be more pronounced when compared to recoverable stress-strains than when evaluating permanent stress-strain response, thus contributing to less cumulative fatigue damage.

- Substantial improvement was obtained when using a PVA geogrid installed in a single geogrid-reinforced HMA layer of a pavement section, showing that this is potential solution for pavement reinforcement. Reductions over 50% were found in rut depth measurements, as well as longitudinal and transverse strains in lower and upper asphalt concrete layers in comparison to unreinforced overlay section. A superior distribution of traffic loads in base course and at the bottom of asphalt layer was evidenced and over 60% reduction of the vertical compressive stresses were registered in the subgrade layer. Also, approximately 70% reduction on maximum elastic deflections on asphalt surface were observed, implying in fatigue life extension of the asphalt mix. This technique has shown to provide long-term benefit and demanding less frequent maintenance to flexible pavements. This benefits can be reached without the need of reconstruction of aggregate base layers.
- The lifetime of a geogrid-reinforced asphalt overlay section with reduced base course thickness was found to be comparable to that of an unreinforced asphalt overlay with a double aggregate base course thickness, presenting similar behavior in terms of rate of running over time, deflection basin measurement and dynamic stress in pavement layers. Elastic AC strains slightly increased in the reduced base section in comparison to unreinforced section, with very similar behavior of plastic strains evolution in the asphalt concrete layers, although general improvement was appreciable. The use and specification of geogrids within asphalt overlays to improve pavement structural performance may represent a cost effective alternative to reduce the use of granular pavement materials, as well as extending pavement service life.
- The use of extensometers attached to the geogrid allowed approximate calculation of average displacements between any two adjacent points in the geogrid, as well as the distribution of strains along geogrid transverse rib. A comparison of an asphalt

interlayer without reinforcement could be assessed. Emphasis was placed on to monitoring points under the wheel load. The measurement of displacement along the geogrid provided understanding of the distribution of strains during traffic load. A typical trend of distribution could be evidenced.

- Cumulative development of permanent strains was observed directly beneath the load in the transverse direction of the path layer until the complete mobilization of the geogrid in Tests 3 and 4. After a certain period of test, plastic strains developed in the geogrid sections remained constant. In contrast, plastic strains in unreinforced asphalt interlayer continued increasing during the test. Significant tensile plastic strains were observed only in the vicinity of the wheel, until 350 mm from the load center. Unreinforced test sections exhibit much higher plastic strains than geogrid-reinforced test sections, illustrating the ability of the geogrid to minimize lateral movements of the asphalt layer. These results illustrate the lateral restraining mechanisms effect on the performance of the HMA layer, by the development of shearing resistance (plastic behavior) with the geogrid. Within the asphalt layers, maximum plastic strains reached 0.125% for geogrid-reinforced sections, while they exceeded 0.2% in unreinforced sections. A mobilized length of 350 mm was identified in geogrid-reinforced sections, showing that the bonding between geogrid and asphalt layers and the stiffness of the geogrid ensured a satisfactory performance in both tests.
- Benefits of geogrid inclusion in asphalt layers were observed to occur typically within the first applied load cycle. An investigations on displacements recorded by the extensometers indicated development of elastic displacements occurring in the geogrid during the tests. Elastic strains predicted using displacements measurements resulted in average peak values of 100 and 200 $\mu\epsilon$, respectively for geogrid reinforcing a new asphalt layer and geogrid-reinforced asphalt overlay with reduced base course. The unreinforced asphalt layer interface showed values of constant elastic strains of 180 $\mu\epsilon$. This values were, in general, very consistent to those monitored by the asphalt strain gages installed in the interface of asphalt layers.
- Several reinforcement mechanisms took place in the present study. Polymeric geogrid reinforcements have considerably reduced strains developed at the bottom of asphalt layers, as well as reduced vertical stresses in pavement lower layers. Resistance to rutting and lateral movement were clearly evidenced in the presented

study. These observations allow to conclude the geogrid in asphalt overlays act as a reinforcement, providing enhanced performance to flexible pavement structures.

The tests conducted in this research involved a geogrid composite made of PVA combined with an attached PP nonwoven geotextile. This specific geogrid provided adequate support for this particular pavement structure; however, extrapolation of the findings of this research to other materials or other conditions requires careful considerations. The observations and correlations obtained in this research are limited to the type of pavement structure and materials used in the experimental program.

7.2 Recommendations for future work

Recommended additional studies are to verify the potential use of geogrids within asphalt overlays to reinforce pavement structures include considering different geogrid stiffness and polymer types, geogrid position in the asphalt layer, tack coat rates and types, as well as traffic load magnitude and pavement structure condition. It is also suggested to conduct proof loading test for pavement layers to evaluate the stiffness of each layer, allowing the assessment of geogrid effect in pavement structure. Using experimental data presented in this study, parametric modelling can also be conducted to contribute to the determination of geosynthetic tensile properties that contribute to enhance the performance of pavement systems. These are valuable tools for an association of geosynthetic relevant properties and its interaction with pavement structure, which will lead to design methodologies to this application and a unified construction approach for geosynthetic-reinforced asphalt overlays.

7.3 Suggestions for equipment improvement

Simple improvements on the test setup can maximize the Accelerated Pavement Testing equipment automation. Automation can be improved by turning off the hydraulic pump automatically after the test. Thus, software will need reprograming. A temperature or oil viscosity control at the hydraulic pump would improve cycle revolution precision, mainly at the beginning of the test when the oil is cold. Additionally, the connection between the piston rod of the hydraulic cylinder and the wheel support should be optimized so that there will be no breakage or loss of connection in this area, which is due to the cyclic movement of the loading carriage.

Appendix A: Test setup

1. Check step by step setup Test setup and instructions.
2. Check tire calibration before test.
3. *LabView* 2011 and “APT Simulator” executable file must installed in the computer.
4. Power hydraulic pump to the energy and data acquisition board (connected to the load cell and data acquisition system – in the case of instrumentation sensors).
5. Plug data acquisition board USB to the computer. Pressure valves lights will be turned off.
6. Power hydraulic pump at the PLC.
7. Open “APT Simulator” software. A control panel opens (Figure A.1).
8. Choose type of loading mode (static or moving wheel load mode). Test begins. Figure A.2 shows the panel of static wheel load mode and Figure A.3 shows moving wheel load mode.
9. Type desired number of load cycles (white box). The recorded number of load cycles and the time of each cycle are indicated in the panel in milliseconds (ms).
10. “Sensors” and “Piston” indicated in the panel shows functioning of each sensor and eventual errors. “Limits”, “Corrections” and “Additional time” must be edited according to the programed revolution period.
11. The test ends up automatically, according to the programed total load cycles. To end up the test earlier than planned, type new total cycle again (white box). At the end of the test, an e-mail is sent to the informed address (Figure A.1), notifying the test is done and the loading carriage goes back to center.
12. Save data and turn off data acquisition system.
13. Power off data acquisition board and hydraulic pump. End of the test.
14. A manual control for valves and piston verification is also available as an executable file, in which valves and piston can be isolatable tested (Figure A.4).

It is recommended to recalibrate the equipment system if main pieces are removed (load cell, hydraulic jack, tire or gate wheels). The bottom of the reaction beam should always be greased to reduce friction between metal pieces. General maintenance is needed every 100,000 load cycles. Gate wheel pieces must be checked/exchanged every 500,000.

Pressure valve Pressure valve Pressure error

Dev1/port1/line1

CHOOSE TEST TYPE

Static wheel test

Moving wheel test

Data to send information and files

 @gmail.com Natalia Correia

Save files

C:\Users\Natalia\Desktop\DADOS

Developed by

www.prototypus.com.br

Date

29/5/2012 10:40:14 AM

Figure A1. APT Simulator software control panel.

Number of cycles **Cycles – Static wheel** **Time loaded (ms)** **Return (ms)**

100 1 2 600

Additional time (ms)

150

Piston **Piston** **Error**

Dev1/port1/line0

Sensor **Sensor** **Error**

Dev1/port0/line1

Figure A.2. Static wheel load mode control panel.

Number of cycles	Cycles – Moving wheel	Braking (ms)		
<input type="text" value="2000"/>	<input type="text" value="900"/>	<input type="text" value="70"/>		
Piston	Piston	Error		
<input type="text" value="Dev1/port1/line2"/>	☐	☐		
Sensor 1	Sensor 1	Error	Total time (ms)	
<input type="text" value="Dev1/port0/line2"/>	☐	☐	<input type="text" value="2351"/>	
Sensor 2	Sensor 2	Error	Time loaded (ms)	
<input type="text" value="Dev1/port0/line3"/>	☐	☐	<input type="text" value="1219"/>	
Sensor 3	Sensor 3	Error	Time return (ms)	
<input type="text" value="Dev1/port0/line4"/>	☐	☐	<input type="text" value="1122"/>	
Piston - Return	Piston - Return	Error	Limit S3-S2 (ms)	
<input type="text" value="Dev1/port1/line3"/>	☐	☐	<input type="text" value="70"/>	
Sensor 4	Sensor 4	Error	Correction (ms)	
<input type="text" value="Dev1/port0/line5"/>	☐	☐	<input type="text" value="10"/>	
Sensor 5	Sensor 5	Error	Time S3-S2 (ms)	
<input type="text" value="Dev1/port0/line6"/>	☐	☐	<input type="text" value="129"/>	
Sensor 6	Sensor 6	Error	Time S6-S5 (ms)	
<input type="text" value="Dev1/port0/line7"/>	☐	☐	<input type="text" value="106"/>	
<input type="button" value="Close"/>				

Figure A.3. Moving wheel load mode control panel.

On/Off	Static wheel - Valve	Error	Valve	<input type="button" value="Stop"/>
☐	<input type="text" value="Dev1/port1/line0"/>	☐	☐	
On/Off	Moving wheel - Valve	Error	Valve	<input type="button" value="Stop"/>
☐	<input type="text" value="Dev1/port1/line2"/>	☐	☐	
On/Off	Moving wheel –Return	Error	Valve	<input type="button" value="Stop"/>
☐	<input type="text" value="Dev1/port1/line3"/>	☐	☐	
On/Off	Pressure Valve	Error	Valve	<input type="button" value="Stop"/>
☐	<input type="text" value="Dev1/port1/line1"/>	☐	☐	
<input type="button" value="Close"/>				

Figure A.4. Manual control panel.

List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ABNT	Brazilian Technical Standards Association
AC	Asphalt concrete
APT	Accelerated Pavement Testing
ASG	Asphalt strain gauges
ASTM	American Society of Testing Materials
CBR	California bearing ratio
CF	Carbon fiber
CMD	Cross machine direction
CNPq	National Counsel of Technological and Scientific Development
CRS	Cationic rapid setting emulsion
DER	Department of Highways of Sao Paulo State
EVA	Ethylene vinyl acetate
FAPESP	Research Support of the State of Sao Paulo
FE	Factor of efficiency
FER	Factor of effectiveness of the reinforcement
FP	Fiber glass polymeric
FWD	Falling weight deflectometer
GG	Geogrid
GT	Geotextile
HMA	Hot mix asphalt
L	Longitudinal
LDS	Linear displacement sensor
MD	Machine direction
MR	Resilient Modulus
N	Number of cycles
NBR	Brazilian Standard
NWGT	Nonwoven geotextile
PET	Polyester
PLC	Programmable logic controller
PP	Polypropylene

PVA	Polyvinyl alcohol
SAMI	Stress absorbing membrane interlayer
SBS	Styrene Butadiene Styrene
SMA	Stone Matrix Asphalt
T	Transverse
TBR	Traffic benefit ratio
TMD	Theoretical maximum specific gravity
TTI	Texas Transportation Institute
TXDOT	Texas Department of Transportation
USP	University of Sao Paulo

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