Geosynthetics in pavements: North American contributions

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ABSTRACT: Geosynthetics have been used to reinforce the base layer of flexible pavement systems for the past thirty years. However, and in spite of the good field evidence that geosynthetic reinforcements can improve pavement performance, the specific conditions or mechanisms that enable and govern the reinforcement function are, at best, unclear as they have remained largely unmeasured. Overall, the selection of design parameters for geosynthetics has been complicated by the difficulty in associating their relevant properties to the improved pavement performance. Nonetheless, significant research has been conducted with the objectives of: (i) determining the governing mechanisms and relevant properties of geosynthetics that contribute to the enhanced performance of pavement systems, (ii) developing appropriate analytical, laboratory and field methods capable of quantifying the above properties for geosynthetics, and (iii) enabling the prediction of pavement performance depending on the various types of geosynthetics used. This paper summarizes research conducted to address these objectives with specific focus on contributions made in North America.

1 INTRODUCTION

The use of geosynthetic reinforcements in regions of an earth structure under tensile stresses helps inhibiting extension strains within the soil, thereby increasing the overall strength of the composite material (Jewell 1981, Palmeira 1987). Common uses of soil-geosynthetic reinforcement include the construction of roads, retaining walls, foundations and embankments. This paper focuses on the North America contributions towards the use of geosynthetic reinforcement in the base-course layer of flexible pavements (paved roads).

2 BACKGROUND

To highlight the significance regarding the use of geosynthetics in flexible pavements, this section provides an overview of flexible pavement design and of the functions of geosynthetics in pavement applications.

2.1 *Flexible pavements*

Flexible pavements can be defined as layered systems that include materials on top (where the contact stresses are high) that have improved qualities than those towards the bottom (where the contact stresses are smaller). Adherence to this principle makes possible the use of local materials and usually results in an economical design (Huang, 1993). A typical flexible pavement system includes four distinct layers: asphalt concrete, base course, subbase, and subgrade (Figure 1). The surface layer is typically asphalt concrete, which is a bituminous hot-mix aggregate (HMA) obtained from distillation of crude petroleum. The asphalt concrete is underlain by a layer of base course, typically consisting of 0.2 m to 0.3 m of unbound coarse aggregate. An optional subbase layer, which generally involves lower quality crushed aggregate, can be placed under the base course in order to reduce costs or to minimize capillary action under the pavement. The constructed layers are placed directly onto a prepared subgrade, which is generally graded and compacted natural insitu soil.

2.1.1 *Critical stresses*

Flexible pavements allow redistribution of traffic loads from the contact surface to the underlying layers. As the pavement flexes under the load, stresses are redistributed over a greater area than that of the tire-footprint. Figure 2 illustrates the stress redistribution under the wheel load. Design of flexible pavement pays particular attention to two critical locations within the pavement structure: (1) the horizontal tensile strain at the bottom of the asphalt layer, which should be minimized in order to prevent fatigue cracking, and (2) the vertical stress on the top of subgrade, which should be minimized in order to reduce permanent deformations. The allowable vertical stress on a given subgrade depends on the shear strength of the subgrade. The granular base in flexible pavements should be thick enough so that the vertical compressive subgrade stress is reduced to some limit value below the allowable distress level (Yoder and Witczak 1975).

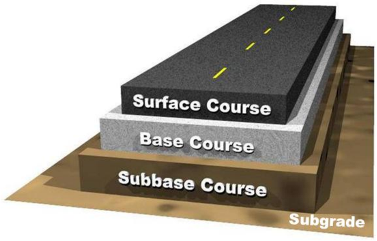

Figure 1: Cross-section of flexible pavement system (Muench 2006)

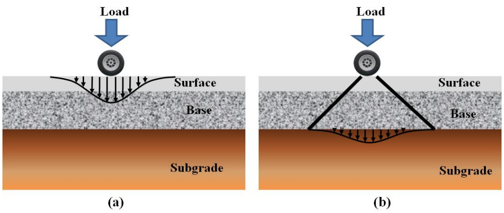

Figure 2: Stress distributions with depth in a flexible pavement (a) high stress area directly under wheel load; (b) reduced load at subgrade level

2.1.2 *Type of failure modes*

During its lifetime, a flexible pavement can experience two different types of failure modes: structural and functional. Structural failure leads to the collapse of the pavement, thereby making it incapable of sustaining the surface loads. Functional failure, on the other hand, renders the pavement incapable of carrying out its intended function, causing discomfort to passengers. Structural failure requires a complete rebuilding of the pavement whereas functional failure can be remediated by maintenance.

Pavement distress may occur due to either traffic or environmental loads. Traffic loads result from the repetition of wheel loads, which can cause either structural or functional failure. Environmental loads are induced by climatic conditions, such as variations in temperature or moisture in the subgrade, which can cause surface irregularities and structural weaknesses. Cycles of wetting and drying (or freezing and thawing) cause base course material to breakdown, generating fines in the subgrade and leading to crack development. Construction practices also affect pavement distress conditions. For example, the use of aggregates with excessive fines and inadequate inspection may lead to rapid pavement deterioration. Finally, pavement distress is also a function of maintenance or, more correctly, lack of maintenance (Yoder and Witczak 1975). For example, sealing cracks and joints at proper intervals and maintaining the shoulders help improve pavement performance. Ultimately, a pavement's intended longevity represents a calculated decision on the part of the engineer who has to balance increased initial construction costs against increased maintenance costs during the design process.

2.2 *Reinforcement function*

Typical functions of geosynthetics used in the construction of roadways include reinforcement, separation, filtration, lateral drainage and sealing (Koerner 2005). Geosynthetics used for separation minimize intrusion of subgrade soil into the aggregate base or sub-base. The potential for the mixing of soil layers occurs when the base course is compacted over the subgrade during construction and also during operation of traffic. Additionally, a geosynthetic can perform a filtration function by restricting the movement of soil particles while allowing water to move from the subgrade soil to the coarser adjacent base. In addition, the in-plane drainage function of a geosynthetic can provide lateral movement of water within the plane of the geosynthetic. Finally, geosynthetics can be used to mitigate the propagation of cracks by sealing the asphalt layer when used in the overlay of the pavement.

This paper focuses on the reinforcement function of geosynthetics in flexible pavements. Reinforcement is the synergistic improvement of the pavement created by the introduction of a geosynthetic into a pavement layer. While the function of reinforcement has often been accomplished using geogrids, geotextiles have also been used as reinforcement inclusions in transportation applications (Bueno *et al.* 2005, Benjamin *et al.* 2007). The stresses over the subgrade are higher in unreinforced flexible pavements than in geosynthetic-reinforced pavement, as shown schematically in Figure 3. The geosynthetic reinforcement has generally been placed at the interface between the base and sub-base layers or the interface between the sub-base and subgrade layers or within the base course layer of the flexible pavement.

The improved performance of the pavement due to geosynthetic reinforcement has been attributed to three main mechanisms, as follows: (1) lateral restraint, (2) increased bearing capacity, and (3) the tensioned membrane effect (Giroud and Noiray 1981, Giroud *et al.* 1984, Perkins and Ismeik 1997, Holtz *et al.* 1998). These three mechanisms are illustrated in Figure 4. The rationale behind these three mechanisms was originally based on observations and analyses done for unpaved roads. The relevance of these mechanisms for flexible pavements is discussed next.

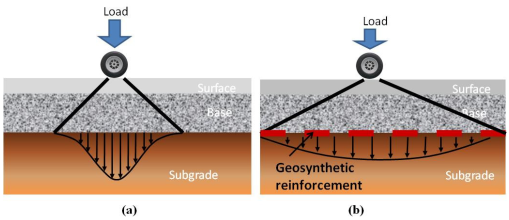

Figure 3: Relative load magnitudes at subgrade layer level for (a) unreinforced flexible pavement and (b) geosynthetic-reinforced flexible pavement

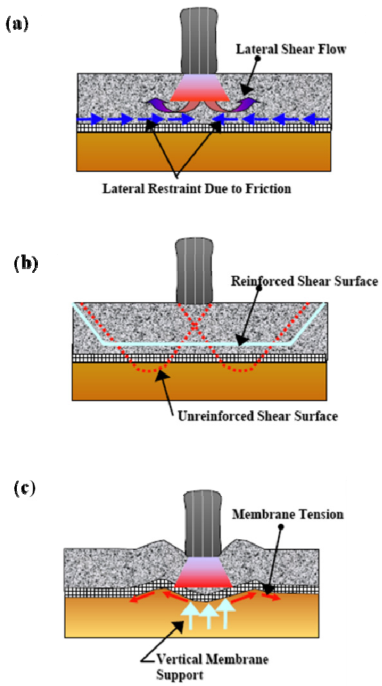

Figure 4: Reinforcement mechanisms induced by geosynthetics (Holtz et al. 1998): (a) Lateral restraint; (b) Increased bearing capacity; and (c) Membrane support

2.2.1 *Lateral Restraint*

The primary mechanism associated with the reinforcement function for flexible pavements (Figure 4a) is lateral restraint or confinement (Bender and Barenberg 1978). The name is misleading as lateral restraint develops through interfacial friction between the geosynthetic and the aggregate, thus the mechanism is one of a shear-resisting interface (Perkins 1999). When an aggregate layer is subjected to traffic loading, the aggregate tends to move laterally unless it is restrained by the subgrade or geosynthetic reinforcement. Interaction between the base aggregate 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. A geotextile with good frictional capabilities can also provide tensile resistance to lateral aggregate movement.

2.2.2 *Increased bearing capacity*

As illustrated in Figure 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.

2.2.3 *Tensioned membrane effect*

The geosynthetic can also be assumed to act as a tensioned membrane, which supports the wheel loads (Figure 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 realize this effect. Higher deformations are required to mobilize the tension of the membrane for decreasing stiffness of the geosynthetic. In order for this type of reinforcement mechanism to be significant, there is a consensus that the subgrade CBR should be below 3 (Barksdale *et al.* 1989).

2.2.4 *Relevance of the Various Mechanisms*

The aforementioned mechanisms require different magnitudes of deformation in the pavement system to be mobilized. Since the early studies on geosynthetic reinforcement of base course layers focused on unpaved roads, significant rutting depths (in excess of 25 mm) may have been tolerable. The increased bearing capacity and tensioned membrane support mechanisms have been considered for paved roads. However, the deformation needed 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 the most for the improved performance of geosynthetic-reinforced pavements.

3 NORTH AMERICAN DESIGN METHODOLOGIES FOR GEOSYNTHETIC-REINFORCED FLEXIBLE PAVEMENTS

The design philosophy of flexible pavement systems was initiated by the Romans, evolving into the current design approaches. As mentioned, the design approach involves providing a protective layer over the subgrade that improves the serviceability under traffic and environmental loads. Figure 5 shows the evolution of road design methods in the US since the 1930s.

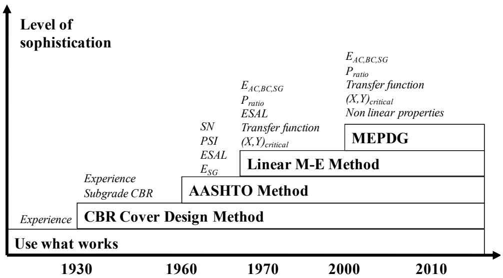

Figure 5: Evolution of pavement design methods (Reck 2009)

The Cover Based Design Method was developed after the great depression in the 1930s. It required a single input in terms of the California Bearing Ratio (CBR), but it also required use of significant engineering judgment. Subsequently, and after completion of the American Association of State Highway Officials (AASHO) Road Test in the 1960s, a series of design methods were proposed. They were more sophisticated than the Cover Based Method, requiring a greater number of design parameters as input. For example, in the 1970s, the linear mechanistic-empirical (M-E) design method was proposed by researchers from South Africa. Since the early 1990s, the focus in the US has shifted to M-E design methods that incorporate features from purely empirical methods to sophisticated non-linear finite element methods. Attempts have been made to incorporate the use of geosynthetic reinforcements into AASHO and M-E design methods. The advantages and limitations of these approaches for designing geosynthetic-reinforced flexible pavements are discussed next.

3.1 *AASHTO Method*

The American Association of State Highway and Transportation Officials (AASHTO) guide for design of pavement structures is one of the most widely used methods for flexible pavement design in North America (AASHTO 1993). The AASHTO method uses empirical equations developed from the AASHO road tests, which were conducted in the late 1950s. The method considers the pavement as a multi-layer elastic system with an overall structural number (*SN*) that reflects the total pavement thickness and its resiliency to repeated traffic loading. The required *SN* for a project is selected such that the pavement will support anticipated traffic loads and experience a loss in serviceability no greater than established by project requirements. The *SN* is determined using a nomograph that solves the following equation:

$$\log W_{18} = Z_R \times S_O + 9.36 \times \log(SN+1) - 0.2 + \frac{\log \frac{\Delta PSI}{2.7}}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \log M_R - 8.07 \quad (1)$$

where W_{18} is the anticipated cumulative 18-kip Equivalent Single-Axle Loads (ESALs) over the design life of the pavements, Z_R is the standard normal deviate for reliability level, S_O is the overall standard deviation, Δ_{PSI} is the allowable loss in serviceability, and M_R is the resilient modulus (stiffness) of the underlying subgrade. Once the required overall *SN* has been determined, the individual layers can be designed according through a series of iterations using the following equation:

$$SN = (a \times d)_{hma} + (a \times d \times m)_{base} + (a \times d \times m)_{subbase} \quad (2)$$

where *a* is the coefficient of relative strength, *d* is the thickness in inches of each layer, and *m* is the modifier accounting for moisture characteristics of the pavement.

The purposes of using geosynthetics as reinforcement in flexible pavements have been: (1) to extend a pavement's life-span, or (2) to enable the construction of a pavement with a reduced quantity of base course material without sacrificing pavement performance. Early design approaches for reinforced flexible pavements focused at modifying Equations 1 and 2 to reflect the benefit achieved by the addition of geosynthetics. These improvements to the pavement system provided by geosynthetic reinforcement have been measured in terms of the Traffic Benefit Ratio (TBR) and the Base Course Reduction (BCR).

The TBR is defined as the ratio between the number of load cycles on a reinforced section (N_R) to reach a defined failure state (a given rutting depth) and the number of load cycles on an unreinforced section (N_U) with the same geometry and material constituents that reaches the same defined failure state (Berg *et al.* 2000). Specifically, the TBR can be defined as:

$$TBR = \frac{N_R}{N_U} \tag{3}$$

Use of the TBR in pavement design leads to an extended pavement life defined by:

$$W_{18}\,(reinforced) = TBR * W_{18}\,(unreinforced) \tag{4}$$

The TBR is sometimes referred to as the traffic improvement factor (TIF), which is commonly used to relate the long-term performance of reinforced and unreinforced pavements. As shown in Figure 6, the TBR can also be used to calculate the number of traffic passes that a reinforced pavement can withstand as compared to an unreinforced pavement for a given rutting depth. For most geotextiles, the TBR value ranges from 1.5 and 10, and for geogrids between 1.5 to 70 (Shukla 2002).

The BCR is defined as the percent reduction in the base-course thickness due to an addition of geosynthetic reinforcement (T_R) in relation to the thickness of the flexible pavement with the same materials but without reinforcement (T_U), to reach the defined failure state. The BCR is defined as follows:

$$BCR = \frac{T_R}{T_U} \tag{5}$$

The BCR is sometimes referred to as the layer coefficient ratio (LCR). A modifier has been applied to the *SN* of the pavement, as follows:

$$SN = (a \times d)_{hma} + BCR.(a \times d \times m)_{base} + (a \times d \times m)_{subbase} \tag{6}$$

When designing a pavement using the BCR, the reduced depth of the base course can be estimated as follows:

$$d_{base,(R)} = \frac{SN_u - (a \times d)_{hma} - (a \times d \times m)_{subbase}}{BCR.(a \times m)_{base}} \tag{7}$$

where $d_{base,(R)}$ is the reduced base course thickness due to reinforcement and SN_u is the structural number corresponding to the equivalent W_{18} for the unreinforced pavement.

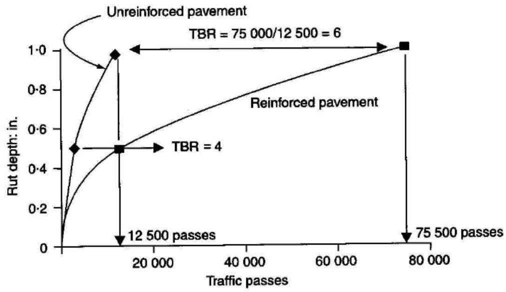

Figure 6: Typical TBR values for an unreinforced and reinforced pavement to reach a given rutting depth (Shukla 2002)

The BCR has been determined from laboratory and field tests. Anderson and Killeavy (1989) constructed test sections with different base course thicknesses. The study showed that geotextile-reinforced section with a 350 mm thick base layer performed similarly to an unreinforced section with a 450 mm thick base layer. Miura *et al.* (1990) reported the construction of field reinforced sections that contained a base course that was 50 mm thinner than that of unreinforced sections. The reinforced sections were observed to perform better than the control sections for all rutting depths. Also, at a site with a subgrade of CBR 8, Webster (1993) showed that a section containing a geogrid with a 150 mm-thick base showed a performance equivalent to that of an unreinforced section with a 250 mm-thick base. Thus, BCRs ranging from 20% to 40% have been reported in the literature, with greater percentage reduction for stronger subgrade materials.

The AASHTO design method is empirical in nature and does not directly consider the mechanics of the pavement structure, climatic effects, or changes in traffic loads and material properties over the design-life of the pavement. Extension of this design methodology to geosynthetic-reinforced pavements has been limited to the case of specific products, materials, geometries, failure criteria and loads used in test sections to quantify their values. Thus, this approach lacks desirable generality as experience cannot be easily transferred from one site to another.

This method has been unable to provide a consistent groundwork for performance comparison among various geosynthetics. In addition, it has been difficult to incorporate the BCR and TBR ratios into the design where the objective of the reinforcement is to provide both an increased pavement life and a reduced base course thickness. Although research conducted to date has supported some of the procedures, long-term performance information of projects designed using this method is not available in order to establish confidence limits.

3.2 *NCHRP Mechanistic-Empirical Method (2004)*

The National Cooperative Highway Research Program (NCHRP) has recently developed a guide for M-E design of new and rehabilitated pavement structures (NCHRP 2004). The method uses mechanistic principles and detailed input data to minimize design reliance on empirical observations and correlations that may be applicable for a specific project. The M-E method attempts to improve design reliability, reduce life-cycle costs, characterize better the effects of drainage and seasonal moisture variations, and prevent premature failures (Olidis and Hein 2004).

While the M-E design method involves two key components (mechanistic and empirical), they are both considered interdependent on each other. The calculation models require input parameters regarding pavement layers, traffic conditions, climatic conditions and materials. The generated output is then compared against parameters used as hypothesis for the original design. If the comparison fails, the design is then modified using an iterative process and re-evaluated. The flowchart of the various components in the M-E design method is shown in Figure 7.

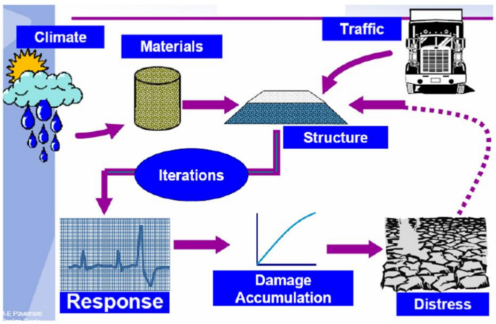

Figure 7: Flowchart for M-E Design (NCHRP 2004)

The main parameters used in M-E method are the mechanistic properties of each pavement layer, including their Poisson's ratio (υ) and resilient modulus (M_R). The Poisson's ratio (ratio of lateral to axial strains exhibited in response to axial loading) typically ranges from 0.15 to 0.5 for pavement materials. The M_R is a measure of the material stiffness after cyclic loading, represented by:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{8}$$

where σ_d is the cyclic deviator stress (or cyclic principal stress difference) and ε_r is the recoverable (elastic) strain. Thus, both M_R and the Young's Modulus *(E)* represent the strain response of the material to applied stresses. However, they are not considered the same due to differences in the rate of load application, as shown in Figure 8. The value of *E* refers to the initial deformation (with some permanent component) of the material, whereas M_R refers to the elastic deformation of the material after cyclic loading.

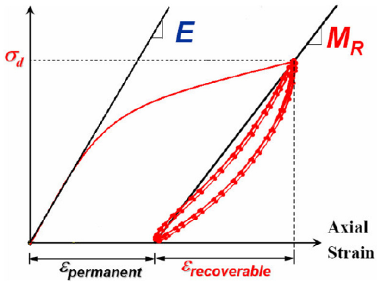

Figure 8: Comparison of Resilient Modulus, M_R, and Modulus of Elasticity, E (Abusaid 2006)

The M-E method uses a hierarchical approach to design, based on the project importance and available information. Level 1 is the highest confidence level, typically reserved for research or very high-volume roads. Level 2 corresponds to moderate confidence level, intended for routine pavement design. Level 3 is the lowest confidence level, typically reserved for low-volume roads. Based on the selected design level, material properties are determined using the specific materials to be used in actual construction (Level 1), or estimated from the correlations using routine tests (Level 2), or are defined using default values from the database (Level 3).

The mechanistic properties of pavement materials are used to estimate stresses and displacements under loading. These estimates are in turn converted into pavement surface distresses using regression models of the Long Term Pavement Performance (LTPP) program database, which contains comprehensive data from field-scale road test sections. Surface distresses are broadly classified into three groups: fracture, deformation, and degradation. These surface distresses can be used to evaluate performance, estimate life cycle and anticipate failure modes of the pavement.

Design of pavements using the M-E approach involves measuring the traffic load cycles that correspond to a limited level of surface distress. This approach could be applied to geosynthetic-reinforced pavements. The M-E design approach is better suited than the AASHTO approach to incorporate geosynthetic benefits. This is because the M-E approach requires input from the user to define the local materials, thus providing a more consistent basis for evaluation of geosynthetic properties.

In the mechanistic model, the contribution of a thin layer such as a geosynthetic has been incorporated as an equivalent resilient modulus and Poissons' ratio. Yet, in the empirical design, calibration of the equivalent damage model in terms of subgrade rutting has not provided similar results for thin and thick asphalt geosynthetic-reinforced flexible pavements. Specifically, in thin asphalt pavements the geosynthetic contribution has been incorporated into the properties of the base course layer, whereas in thick asphalt pavements it has been simulated as an equivalent delay in the onset of fatigue cracking (when compared to the onset in an unreinforced pavement section). Consequently, the benefits of geosynthetics have not been consistently defined using the M-E design.

The M-E design approach has been deemed more appropriate method for estimating field behavior of flexible pavements than a multi-layered elastic analysis because it is more rigorous and adaptable (Al-Qadi, 2006). However, the practicality of the method is compromised since a significant amount of information and test data are required to characterize the pavement and its anticipated performance. Only few test agencies have been are capable of performing the complex tests required to determine properties such as M_R, and even when they are, the associated costs could be unjustifiably high. Finally, as in the AASHTO method, the M-E approach also relies heavily on correlations to material properties.

In summary, prediction of the behavior of flexible pavements is complex, as the overall performance is controlled by numerous factors, including load magnitude, subgrade strength, layer thickness, interlayer mixing, material degradation, cracking and rutting, and seasonal and climactic fluctuations (WDOT 2007, Dougan 2007, Al-Qadi 2006). While beneficial, the use of geosynthetic reinforcement adds complexity to the system understanding by introducing a new set of variables. These include the reinforcement mechanism, geosynthetic types and stiffness, tensile strength, aperture size and placement location. Therefore, due to uncertainty in quantifying the mechanisms of geosynthetic-reinforcement, neither the AASHTO (1993) nor the NCHRP (2004) approaches incorporate specific geosynthetic properties fully in design of pavements.

In addition, the design of geosynthetic-reinforced pavements has relied significantly on empirical data used to calibrate the response of unpaved roads. However, flexible pavements are designed for smaller rutting depths, and are subjected to higher volume traffic loading than unpaved roads. Giroud and Han (2004) developed a procedure for the design of geosynthetic reinforced unpaved roads, which considers stress distribution at depth, base course resilient modulus, and degradation of material stiffness with repeated loading. Unfortunately, the theoretical basis that exists for the design of geosynthetic-reinforced unpaved roads is still lacking in the case of geosynthetic-reinforced pavements. Yet, numerous research studies have been performed to better understand the behavior of reinforced pavements using field-scale testing, laboratory testing and numerical simulations.

4 ASSESSMENT OF THE PERFORMANCE OF GEOSYNTHETIC-REINFORCED FLEXIBLE PAVEMENTS

Assessment of the performance of pavements has been conducted using field scale tests, laboratory tests, and numerical simulations. These three methods not only differ widely, but have also provided different perspectives on performance data, as illustrated in Figure 9. Ultimately, the quality of pavement performance data depends on the cost and the method used for its collection (Reck 2009).

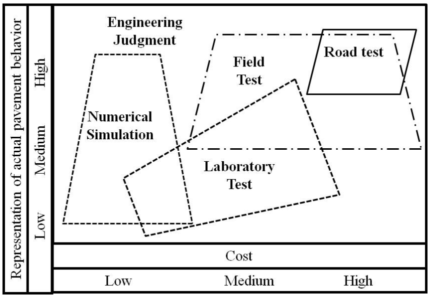

Figure 9: Interrelationship between different facets of pavement design (Hugo et al. 1991)

Full-scale tests include field studies and accelerated pavement tests that simulate actual pavement behavior. In both cases, the cost of data gathering is high and, consequently, only the number of tests typically conducted has been limited. Thus, the testing scope in reported research programs has been often expanded by small-scale laboratory studies or numerical simulations. Laboratory tests are cheaper than field tests and can be performed under controlled conditions. However, it has been difficult to replicate the actual behavior of the pavement system under laboratory tests. Finally, numerical studies have been conducted to simulate both field and laboratory tests. Numerical simulations are particularly valuable for parametric evaluations.

4.1 *Field tests*

Full-scale field tests have been performed on both public roadways and in-service roads. As previously discussed, M-E design processes have been recently developed that require data for calibration and validation purposes (Watts and Blackman 2009). The monitoring of in-service roads is a time consuming process. Consequently, useful data has also been generated using accelerated pavement testing (APT). APT facilities consist of test tracks located either indoor or outdoor (Figure 10). They involve the use of automated, one or two axle, single wheel loads that repeatedly runs over the test track surface. APT may provide a good simulation of the performance of in-service pavements and can be particularly useful to provide rapid indication of pavement performance under severe conditions.

(a) **(b)**

Figure 10: APT test facilities (a) ATLAS at the Illinois Center for Transportation, USA; (b) pavement fatigue carousel at LCPC, France

Several approaches have been implemented to evaluate and compare pavement performance in field-scale test sections. In flexible pavements, the two most commonly quantified variables are surface deflection and cracking (including longitudinal, transverse and fatigue). Surface deflection is the most common performance criterion for both reinforced and unreinforced pavements. Distress has been evaluated using: (1) measurement of existing surface deflections in terms of rutting depth, and (2) measurement of surface deflections in response to an applied load to determine its structural capacity.

Rutting occurs because of the development of permanent deformations in any of the pavement layers or in the subgrade. Rutting is generally measured in square meters of surface area for a given severity level, as defined from data collected with a dipstick profiler every 15 m intervals. Measurements of rutting depth are comparatively easy to obtain, as they are taken at the pavement surface, and provide a simple method of comparing pavement performance among multiple test sections.

Deflection measurements have also been made using non-destructive testing (NDT) devices in order to evaluate the pavement structural capacity and to calculate the moduli of various pavement components. The device most widely used to measure pavement deflections is the Falling Weight Deflectometer (FWD) (Figure 11a). This approach involves applying a series of impulses on the pavement using a trailer-mounted device that is driven to the desired test locations. A loading plate is hydraulically lowered to the pavement surface, after which an impulse is applied to the pavement by dropping a weight from a known height onto the loading plate. The magnitude of the load is measured using a load cell while deflections are measured using seven velocity transducers. An equipment known as a Rolling Dynamic Deflectometer (RDD) (Figure 11b), has been recently developed for assessing the conditions of pavements and determining pavement deflection profiles continuously (Bay and Stokoe 1998). Unlike the FWD, the RDD performs continuous rather than discrete deflection measurements. The ability to perform continuous measurements makes RDD testing an effective approach for expeditious characterization of large pavement sections. The equipment applies sinusoidal forces to the pavement through specially designed rollers. The resulting deflections are measured by rolling sensors designed to minimize the noise caused by rough pavement surfaces.

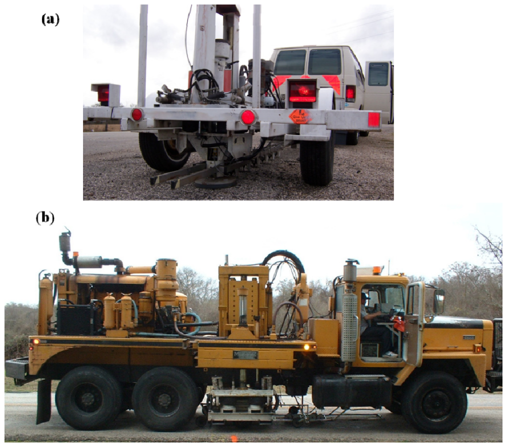

Figure 11: Non-destructive testing methods used in pavement evaluation (a) Falling Weight Deflectometer and; (b) Rolling Dynamic Deflectometer

Field tests on full-scale road sections have been conducted to evaluate the effect of geosynthetic reinforcement in flexible pavement systems. Perkins and Ismeik (1997) compared the results from nine sections, among which four were constructed on indoor test tracks, three on outdoor test tracks, one on a public roadway and one in a field truck-staging area.

The indoor test tracks used a single moving wheel to load the test sections (Brown *et al.* 1982, Barksdale *et al.* 1989, Collin *et al.* 1996, Moghaddas-Nejad and Small 1996). The outdoor test tracks involved a single moving wheel (Barker 1987, Webster 1993), and a two-axle, dual wheel truck to load the pavement (Halliday and Potter 1984).

Additional studies have been recently reported on geosynthetic-reinforced test sections using APT equipment (Cancelli and Montanelli 1999, Perkins 2002, Perkins and Cortez 2005, Al-Qadi *et al.* 2008, Reck *et al.* 2009). Assessment of these test sections indicated that rutting depth continued to be the most common method to evaluate pavement distress. A total of nine field test sections and four APT sections were reported involving measurements from profilometer readings at the end of design loading cycles. However, FWD tests were conducted only at four field sections and at one APT section.

Zornberg and Gupta (2009) reported three case studies conducted in Texas, USA, for geosynthetic-reinforced pavements on which FWD testing was conducted on in-service roads. One of the cases involved a forensic investigation conducted in a newly constructed pavement. Longitudinal cracks were observed in a geogrid-reinforced pavement before it was open to traffic. However, the investigation revealed that the contractor had laid rolls of geogrid leaving a portion of the pavement unreinforced. Cracks only appeared in unreinforced locations within the pavement. Accordingly, the difference in response within and beyond reinforced portions of the pavement illustrated that use of geogrid can prevent pavement cracking.

The second case study reported the field performance of geogrid-reinforced pavements built over highly plastic subgrade soils. The pavement sections had been reinforced using two different types of geogrids that met project specifications. Although a section reinforced with one type of geogrid was found to be performing well, the other section reinforced with second type of geogrid showed longitudinal cracking. The reviews of the material properties lead to the preliminary conclusion that poor performance in the second section was due to inadequate junction efficiency. Further inspection indicated a higher tensile modulus of the geogrid used in the better performing section. This study highlighted the need for better material characterization and the possible inadequacy of commonly used specifications for geosynthetic-reinforced pavements.

The third case involved three pavement sections. The two geogrid-reinforced sections (Sections 1 and 2) had base course thicknesses of 0.20 m and 0.127 m, respectively. On the other hand, a control sections (without geogrid reinforcement) had a 0.20 m-thick base course layer. FWD testing showed a comparatively higher pavement modulus for the geogrid-reinforced section with a 0.20 m-thick base while lower modulus value were obtained for the geogrid-reinforced section with a 0.127 m-thick base. Yet, field visual assessment showed cracking in the control section while the two geogrid-reinforced sections performed well. While the geogrid-reinforced sections outperform the unreinforced section, the results of FWD testing showed a different trend. This study illustrated the inadequacy of the currently available evaluation techniques involving non-destructive testing for the purpose of quantifying the benefits of geosynthetic reinforcements.

The lessons learned from these field case studies, provided the basis for a field monitoring program to evaluate the performance of geosynthetic-reinforced pavements constructed over expansive clays. This involved the rehabilitation of a low-volume road in Texas by use of geosynthetic reinforcements. A comparative evaluation with 32 test sections was conducted. This included 8 different reinforcement schemes (3 reinforcement products and an unreinforced control section, as well as lime stabilized sections). Also, and in order to account for variability due to environmental, construction and subgrade-type, a total of 4 repeats were constructed for each one of the 8 schemes. Therefore, a total of 32 test sections (4 reinforcement types x 2 stabilization approaches x 4 repeats) were constructed (Figure 12).

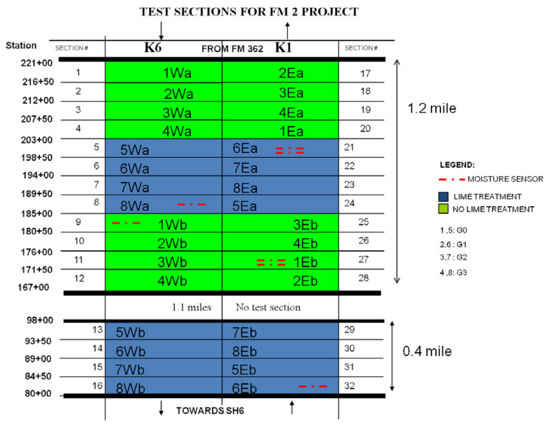

Figure 12: Schematic layout of test sections at FM 2 site (Gupta et.al 2008)

Due to unique characteristics of this field study, the reinforced pavement was considered experimental and an extensive post-construction performance monitoring program was implemented. This included the installation of moisture sensors to characterize the patterns of moisture migration under the pavement. A total of eight horizontal moisture and vertical moisture sensor profiles, each containing an array of four sensors was installed below the pavement. Field monitoring involving visual inspection, surveying and FWD was conducted before reconstruction and immediately after reconstruction of the road. The final construction of the reinforced pavement was completed in January 2006 and performance evaluation of the newly reconstructed road has been conducted on a regular basis since then. The results obtained from the field study are providing good understanding of the underlying mechanisms governing the performance of the geosynthetic-reinforced pavements. Also, the collected data is useful to quantify the mechanisms of longitudinal cracking and effectiveness of the geosynthetic reinforcements in mitigating such distresses.

Overall, the results from field studies reported in the literature have indicated that the geosynthetic-reinforced test sections led to less rutting depth than the unreinforced sections. The improved performance has been attributed to the ability of the geosynthetics to control lateral spreading of the base layer.

4.2 *Laboratory Tests*

A number of laboratory tests have been proposed to quantify the mechanisms governing the performance of geosynthetic-reinforced flexible pavements. The primary objective of laboratory tests has been to quantify the soil-geosynthetic interaction mechanisms in flexible pavement systems either by measuring the geosynthetic index properties or by replicating the field conditions. An important field condition to be replicated is the effect of interface shear provided by geotextiles and interlocking provided by geogrids when used under or within the base course layer of pavements (Figure 13). Depending on the adopted approach, the tests reported in the literature can be grouped into two main categories: unconfined and confined tests. In unconfined tests, geosynthetic properties are measured in-air, while in confined tests they are measured within confinement of soil. The advantages and limitations of the various tests developed in North America in each of these two categories are discussed next.

4.2.1 *Unconfined Tests*

As mentioned, unconfined tests are conducted using geosynthetic specimens in isolation. Advantages of these tests include expedience, simplicity, and cost effectiveness. They can be run in short periods of time using conventional devices, which facilitates the assessment of repeatability of test results. However, correlations are required between the index property obtained from these tests and the field performance of the geosynthetic-reinforced pavements. Tests in this category include the wide-width tensile test, biaxial loading test, junction efficiency test, and torsional rigidity test. While the wide-width tensile test can be conducted using any type of geosynthetics (geogrid, geotextile), the other three tests are specific for the characterization of geogrids.

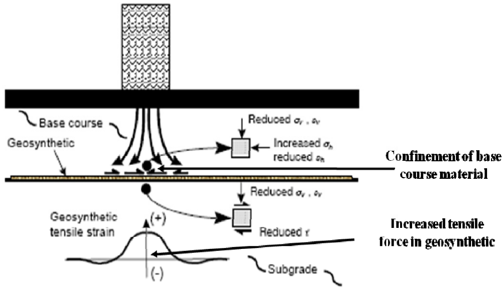

Figure 13: Mechanisms due to soil-geosynthetic interaction in geosynthetic- reinforced pavement that have been tried to be represented in laboratory tests (Perkins 1999)

The tensile strength of geosynthetic materials has often been deemed as the most important property for projects involving reinforcement applications. While tensile strength may not be particularly relevant for the case of pavement design, tensile strength has often been incorporated into pavement design and specifications. The current state of practice for measuring the tensile properties of a geosynthetic involves placing the material within a set of clamps, positioning this assembly in a load frame, and tensioning the geosynthetic until failure occurs. The test is generally performed at a constant strain rate. Currently, two ASTM standards are available for tensile tests. The grab tensile test (D4632) is used for manufacturing quality control, as it involves a narrow geosynthetic specimen. Instead, the wide-width tensile test (D4595) has been used in design applications. The load frame for a wide-width tensile test conducted using roller grips is shown in Figure 14. The tensile test provides the tensile stiffness at different strain values (1%, 2%, and 5%), as well as the ultimate tensile strength. Methods used for unpaved road design have included the tensile stiffness at 5% in product specifications. Based on full scale model studies for the paved roads, Berg *et al.* (2000) reported accumulated in-service tensile strain of 2% in geosynthetics and thus recommended the tensile stiffness at this strain level for design. However, the actual strain level representative of field conditions is certainly smaller for the case of pavement applications.

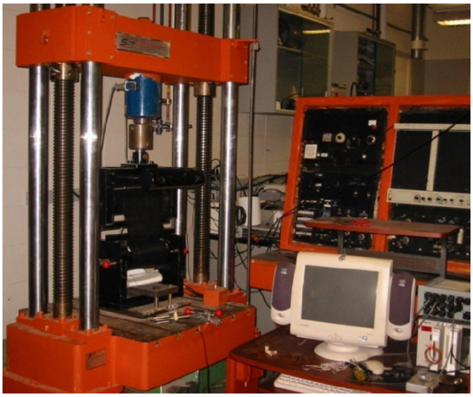

Figure 14: Wide-width tensile test conducted with roller grips at the University of Texas at Austin

Bray and Merry (1999) investigated the stress and strain conditions in wide-width tensile tests. They concluded that strains vary across the specimen from a plane-strain, biaxial condition near the grips, to a uniaxial condition near the center of the specimen. Thus, there may be a misconception that the test measures geosynthetic behavior under the 1-D condition that is representative of field applications. It should be noted that most geogrids tested using uniaxial methods suffer distortions, non-uniform stresses (particularly at the junctions), premature specimen rupture and problems with clamping (McGown *et al.* 2005). Kupec and McGown (2004) suggested a biaxial test method, which focused primarily on geogrids and allowed characterization of the combined strength of tensile ribs and junctions in a single test. The test specimen involved 5 ribs in each direction (i.e. 25 junctions within the central section of the specimen), as shown in Figure 15. Loading was applied to the specimens under isotropic deformation conditions. Test results indicated that the biaxial load-strain time behavior differs significantly from the uniaxial behavior. An increased stiffness was obtained, which was attributed to the behavior of junctions under tensile stresses in the principal direction, to the Poisson's ratio effect and to re-orientation of the fibers in the junction areas.

To address perceived deficiencies of uniaxial tensile test, a complementary uniaxial test, known as the "junction strength test," was developed. It is conducted as per the procedure recommended in GRI-GG2 specifications and involves gripping the cross member of a geogrid rib on both sides of the junction with a clamping device. Load is then applied until the junction breaks. The force required to fail the junction is defined as the junction strength of the geogrid. Junction strength provides quantification of the contribution to stability that may lead to rupture of the reinforcement during the pavement construction and subsequent traffic load. However, the geogrid ability to transfer stress under low strains is a consideration probably of more relevance for the case of flexible pavements. However, junction stiffness requirements for pavement projects have not been properly defined. Also, since this test was originally developed for geogrids with integral junctions, it does not incorporate newer geogrids with entangled fibers or those with heat bonded or laser welded junctions.

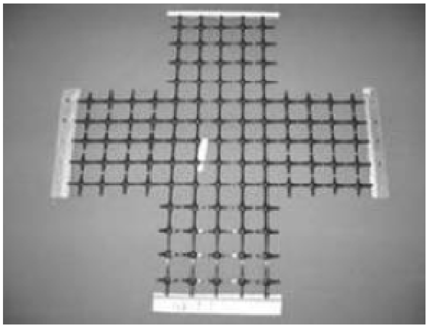

Figure 15: Geogrid specimens for biaxial testing (McGown et al. 2005)

A torsional rigidity test was developed by Kinney and Yuan (1995) to measure the in-plane rotational stiffness of the geogrids (Figure 16). The test aimed at quantifying the performance of geogrid-reinforced paved road tests constructed by the US Army Corps of Engineers at the Waterways Experiment Station. While the test focuses on the interlocking capacity of the geogrid, a relationship between geogrid torsional rigidity and the performance of geogrid reinforced road sections could not be established. The test provides a higher torsional rigidity for stiff geogrids than for flexible geogrids. However, a study conducted by the Texas Research Institute (TRI 2001) reports a lack of correlation between torsional rigidity and the confinement performance of the geogrids.

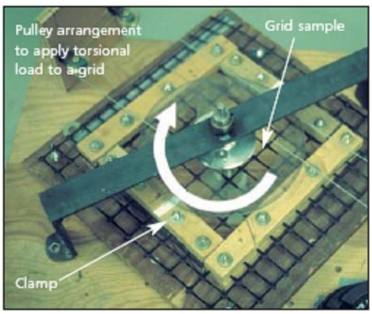

Figure 16: Torsional rigidity tests (Kinney and Yuan 1995)

The geosynthetic behavior observed in the laboratory from unconfined tests has to be correlated with the performance in field applications, which have different loading and boundary conditions. In general, it has been difficult to replicate field conditions using the aforementioned unconfined tests. Consequently, unconfined tests should only be considered as index parameters rather than actual design properties for geosynthetic-reinforced flexible pavements.

4.2.2 *Confined tests*

Geosynthetics used for base reinforcement are under the confinement of soil and subjected to dynamic loading (traffic). These conditions cannot be simulated by monotonic unconfined tests. Geosynthetic-soil confinement depends not only on the macro-structure and properties of geosynthetics but also on the properties of soil and, most importantly, on the interaction between geosynthetics and soil particles (Han *et al.* 2008). The interaction between soil and geosynthetics under confinement, specifically the confined stress-strain properties of the geosynthetics, has been focus of previous research. A Federal Highway Administration (FHWA) sponsored study focusing on existing confined tensile tests for geosynthetics concluded that the unconfined response of geosynthetics is overly conservative and that confinement significantly improves their mechanical response (Elias *et al.* 1998). Recently, a number of confined tests have been proposed, out of which six tests have focused on characterizing the behavior of geosynthetics used to reinforce flexible pavements. These tests include the cyclic plate load test, cyclic triaxial test, cyclic pullout test, bending stiffness test, modified pavement analyzer test, and the pullout stiffness test.

The cyclic plate load test has generally involved large scale laboratory experiments on reinforced and unreinforced pavement sections (Al-Qadi *et al.* 1994, Cancelli *et al.* 1996, Haas *et al.* 1988, Miura *et al.* 1990, Perkins 1999). The test setup designed by Perkins (1999) consisted of a 2 m wide and 1.5 m high reinforced concrete tank (Figure 17). The model pavement section was constructed with a geosynthetic at the interface of the base course and subgrade layers. The load was applied by a pneumatic actuator in the form of a trapezoidal wave pulse, which generated a maximum surface pressure of 550 kPa on the pavement. The force and displacement responses were measured using a load cell and eight surface LVDTs. TBRs ranging from 1 to 70 and BCRs ranging from 20% to 50% were obtained using cyclic plate load tests in sections involving geotextile and geogrid reinforcements (Hsieh and Mao 2005). These tests were reported to have successfully demonstrated the effect of soil confinement and dynamic loading. However, facilities in which cyclic plate loading can be conducted are not readily available, thus restricting the application of this test to research studies. In addition, the cyclic plate loading test was considered to have important drawbacks associated with the testing procedures, time demands, and appropriate simulation of rolling wheel loads (Han *et al.* 2008).

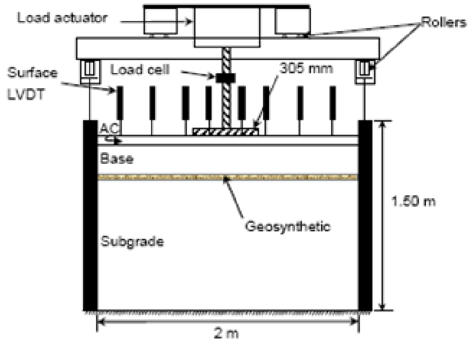

Figure 17: Cyclic plate load test (Perkins 1999)

The cyclic triaxial test (Figure 18) has been used to measure the ability of soils to develop shear stresses induced by cyclic loading (ASTM D5311 2004). The resilient modulus, M_r, of the soil aggregates computed using this test has been specifically used as input in the M-E design (NCHRP 2000). This test was modified by Perkins *et al.* (2004) to quantify the change in resilient modulus and permanent deformation behavior due to the addition of geosynthetics to the aggregate layer of pavements. The results from cyclic triaxial tests indicate that the use of reinforcements does not affect the resilient modulus of the aggregates, although it reduces significantly the pavement permanent deformations. Also, reinforcements were observed to increase the stiffness of the aggregate in zones above and below the geosynthetic (the thickness of these zones was observed to be similar to the specimen radius, or 150 mm). However, relatively poor repeatability of the test results was achieved, making it difficult to quantify differences in the performance expected from the different geogrid products. Also, an appreciable permanent deformation was not observed until reaching mobilized friction values of approximately 30°.

Cyclic pullout tests were conducted by Cuelho and Perkins (2005) by modifying the standard pullout test (ASTM D6706) to resemble the loading protocol used in a cyclic triaxial test (Figure 19). Cyclic shear load cycles (ranging from 100 to 300) were applied at different confinement level beginning with a seating load of 51 kPa until pullout failure was reached. Based on the test results, a parameter known as geosynthetic-soil resilient interface shear stiffness (G_i) was defined to describe the reinforcement-aggregate interaction under cyclic loads. This parameter is defined as:

$$G_i = \frac{\tau_i}{\Delta_i} \tag{9}$$

where Δ_i is the relative displacement between the aggregate and reinforcement and τ_i is the shear stress applied to the interface. The units of G_i are kN/m^3. The parameter, G_i was assumed to closely resemble M_r as it depends on both the shear load and confinement. Therefore, the three parameter log-log equations for M_r reported in NCHRP (2001) was modified and used to calibrate G_i for a given soil-geosynthetic interface, as follows:

$$G_i = k_1 . P_a . \left(\frac{\sigma_i}{p_a}\right)^{k_2} . \left(\frac{\tau_i}{p_a} + 1\right)^{k_3} \tag{10}$$

where σ_i is the normal stress on the interface, p_a is the normalized atmospheric pressure, P_a is the atmospheric pressure per unit length and k_1, k_2 and k_3 are dimensionless material. The purpose of this test was to provide a property useful to characterize the interface shear moduli in finite element simulations conducted to calibrate the M-E approach. However, pullout test results conducted on six geosynthetics indicated that correlations between the predicted and measured values were erratic. The results were sensitive to small changes in displacement magnitudes. Also, the shear load was cycled while normal load was kept constant, which may not be representative of field conditions. Accordingly, additional research was deemed necessary to improve the testing equipment and to establish testing protocols.

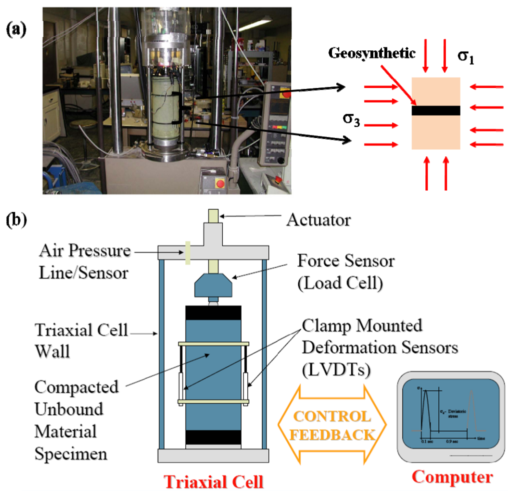

Figure 18: Cyclic triaxial test (a) Test equipment (Perkins et al. 2004); (b) Schematic of test setup (Tutumluer 2004)

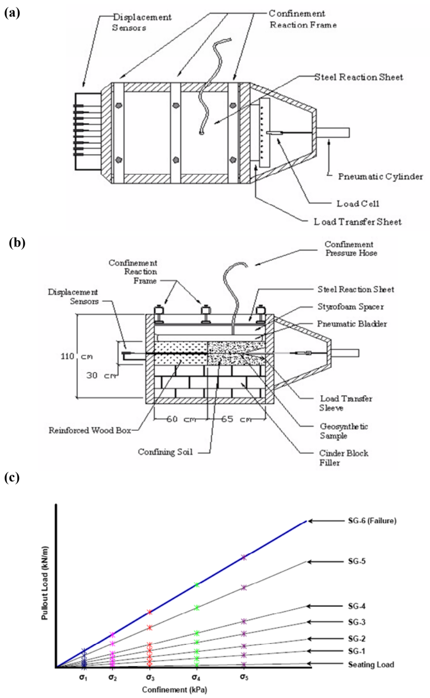

Figure 19: Cyclic pullout test (a) Plan view (b) Side view (c) Loading protocol (Cuelho and Perkins 2005)

The bending stiffness test was developed by Sprague *et al.* (2004) as a small-scale index test procedure aimed at predicting the behavior of geosynthetics used for reinforcement of pavements. The test apparatus is a modified version of the multi-axial tension test for geomembranes (ASTM D 5617) (Figures 20a and 20b). Details regarding the testing procedures are provided by Abusaid (2006) and Finnefrock (2008). Testing involves applying a uniform vacuum pressure of 5 kPa on the soil-geosynthetic interface to induce a confinement representative in a pavement structures. Subsequently, cycles of air pressure cycles are applied to the soil-geosynthetic system and the center-point deflection is measured by a dial gauge to quantify the response. A property identified as the bending stiffness (*BS*) is obtained from the test results, which is defined as the ratio between the deviator stress σ_d and the recoverable deformation Δ_r (Figure 23c). Specifically, *BS* is defined as the slope of the curve representing the applied pressure versus the center-point deflection, as follows:

$$BS = \frac{\sigma_d}{\Delta_r} \tag{11}$$

BS is reported in units consistent with the resilient modulus, M_r, and thus is not a measure of the strength but rather of the stiffness of the system. Test results reported by Finnefrock (2008) show reinforcement benefits of 20% to 25% in terms of *BS* ratio, indicating a clear difference between geogrid-reinforced and unreinforced specimens. However, the relative performance among different geogrid products could not be well identified due to scatter in the test results. Also, a theoretical analysis conducted by Yuan (2005) indicates significant influence of edge shear resistance on the test results. The tests performed on geotextile-reinforced base course sections did not indicate benefits over control sections.

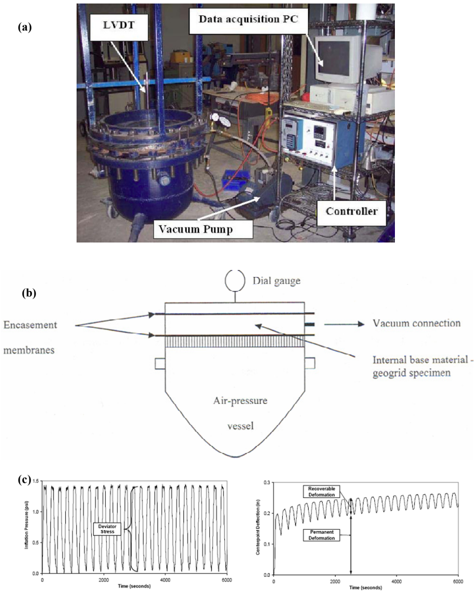

Figure 20: Bending stiffness test (a) Actual test apparatus (b) Schematic of the test (c) Deviator stress and recoverable deformation plots for a typical test (Abusaid 2006)

Han *et al.* (2008) proposed a test method involving the use of an asphalt pavement analyzer (APA) to evaluate the benefits of geosynthetic-reinforcement in the base course layer of the pavement (Figure 21a). The APA is a multifunctional wheel-loaded test device used to quantify permanent deformation, fatigue cracking, and moisture susceptibility of both hot and cold asphalt mixes (Figure 21b). A conventional box was modified in order to conduct the test on a geosynthetic-reinforced base course. The loaded wheel is moved back and forth on the surface of base course as shown in Figures 21c to 21h. A relationship was established between the measured rutting depth and the number of passes. The results could be used to compare the response of geosynthetic-reinforced layers with the unreinforced base layer. Besides evaluating the TBR for pavement sections, the authors proposed a parameter known as rut reduction ratio (RRR), defined as the ratio between the rutting in the reinforced base and that of in the unreinforced base for a given service life (8,000 cycles). Geosynthetics leading to a lower RRR value are expected to result in better field performance. Tests were conducted using four geosynthetics (three geogrids and one geotextile) and two base course materials. TBR values ranging from 1 to 36 and RRR values ranging from 0.3 to 1.2 were obtained. When a surcharge was applied to the soil-geosynthetic system to simulate confinement in the field, lower rutting depths were observed as compared to unconfined tests for a similar setup.

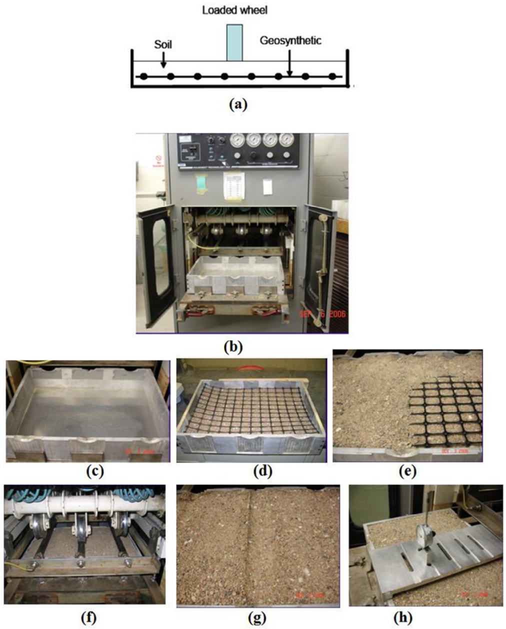

Figure 21: Modified pavement analyzer test (a) schematic of the test (b) APA testing machine (c) modified box (d) geosynthetic placed in the middle of the box (e) base course layer over the geosynthetic (f) test with loaded wheels (g) rut observed at the end of test (h) rut measurement using dial gauge (Han et al. 2008)

A Pullout Stiffness Test (PST) was recently developed by Gupta (2009) at the University of Texas, Austin in order to quantify the soil-geosynthetic interaction in reinforced pavements. The equipment involves a modified large-scale pullout test modified to capture the stiffness of the soil-geosynthetic interface under small displacements. Research conducted using the PST has shown that monotonic pullout tests (Figure 22) aimed at characterizing the soil-geosynthetic interaction under low displacements are promising. Although these pullout tests did not replicate the cyclic nature of traffic load conditions, it simulated the interface transfer mechanisms between soil and geosynthetic reinforcements that are expected in the field.

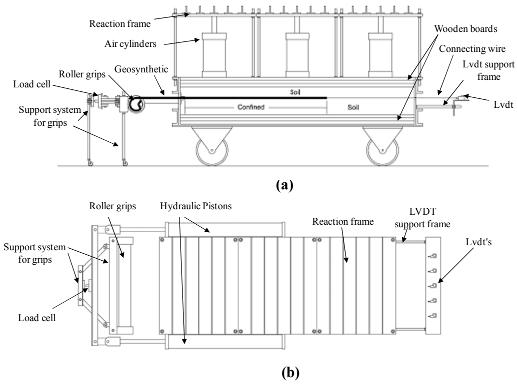

Figure 22: Pullout stiffness test to quantify soil-geosynthetic interaction (a) Side view; (b) Plan view (Gupta 2009)

An analytical model was proposed to predict the confined load-strain characteristics of soil-geosynthetic systems under small displacements using the results obtained from the PST. This approach takes into account both the confined stiffness (J_c) and ability of geosynthetic to mobilize shear or interlock (τ_y), which are two important parameters governing the performance of geosynthetic interfaces. The two parameters can be combined to define a unique coefficient of soil-geosynthetic interaction (K_{SGI}) that characterizes the soil-reinforcement interface. This coefficient is computed as:

$$K_{SGI} = 4.\tau_y.J_C \tag{12}$$

A comprehensive field monitoring program is under way (Figure 12) to relate the field performance to laboratory PST results for a number of geosynthetic reinforcements. While ongoing field monitoring is still in progress, good agreement has been obtained so far between the field performance and the properties defined from PST testing. Thus, a new performance-based test method in the form of a pullout stiffness test is promising as a performance-based test to evaluate the soil-geosynthetic confinement.

An overall assessment of the various tests developed so far in North America for geosynthetic-reinforced pavements indicates that unconfined tests are simple, economical and expeditious, although they do not capture the important aspects associated with confinement and the type of soil. Also, unconfined tests have provided only index measures of the actual mechanisms, requiring subsequent correlations with field performance. It should be noted that field studies sometimes led to performance trends that contradicted the trends obtained using properties from unconfined tests. Accordingly, and based on the current body of literature, unconfined tests are considered inadequate for assessment of the performance of geosynthetic-reinforced pavements.

A summary of the confined test methods developed for the evaluation of geosynthetic-reinforced pavements is presented in Table 1. The tests provide quantification of the soil-geosynthetic interaction behavior, although they are comparatively more expensive and time consuming than unconfined tests. The tests quantify the performance of the soil-reinforcement system in the terms of reduced deflections (e.g. TBR, *BS*, RRR) or increased confinement modulus (e.g. M_r , G_i ,K_{SGI}). Results from confined tests are deemed more appropriate as input in design methods such as the AASHTO and M-E design approaches. The various studies indicated that reinforced systems provided improvement over control sections without geosynthetics. However, drawbacks were also identified in several of the proposed confined test approaches. Specifically, these tests require specialized equipment and, at least in several of the proposed methods, the variability of test results was significant. Overall, confined testing approaches were considered more representative and appropriate to assess the improvement of geosynthetic reinforcements in pavements than unconfined testing methods. The main characteristics and relative merits of the various confined tests are summarized in Table 1.

Based on this evaluation, it may be concluded that a reasonable test method should include the following features: (a) ability to capture the mechanism of lateral restraint; (b) provide parameter(s) suitable for M-E design; (c) provide good repeatability of test results; (d) utilize parameter(s) that distinguish between the performance of different geosynthetics; (e) be sensitive under low displacements; and (f) be easy to conduct. The PST approach was developed keeping these features in mind, and it appears promising for design of geosynthetic-reinforced pavements.

4.3 *Numerical Studies*

The design of flexible pavements involves understanding the behavior and the interaction between various materials (asphalt, base course, subgrade, geosynthetic reinforcements). Current design methods are empirical in nature. This is partly because of the inability of available analytical tools to predict the time-dependent behavior of pavements under actual traffic loads. However, numerical methods can be used to provide insight into the mechanics of pavement systems. The most commonly used methods are the finite elements method (FEM) and the discrete elements method (DEM).

Table 1 Features of confined tests

Test Type	Cyclic plate load test	Cyclic triaxial test	Cyclic pullout test	Bending stiffness test	Modified asphalt pavement analyzer	Pullout Stiffness Test
References	Perkins (1999)	Perkins *et al.* (2004)	Cuelho and Perkins (2005)	Sprague *et al.* (2004)	Han *et al.* (2008)	Gupta (2009)
Loading type	Cyclic	Cyclic	Cyclic	Cyclic	Moving wheel	Monotonic
Design property	TBR	M_r	G_i	BS	RRR	K_{SGI}
Suitable design method	AASHTO	M-E	M-E	AASHTO	AASHTO	M-E
Ease of running test	Difficult	Difficult	Moderate	Moderate	Easy	Moderate
Control section	Yes	Yes	No	Yes	Yes	No
Repeatability of test results	-	No	No	No	Yes	Yes
Ability to distinguish among various geosynthetics	-	No	No	No	Yes	Yes

Finite elements have been used in several studies to simulate the behavior of geosynthetics used to reinforce flexible pavements. Many of these studies were performed in combination with laboratory or field test studies so that comparisons between model predictions and experimental results could be made. A summary of important features of these studies is presented in Table 2.

Results from the finite element studies have been generally reported in the form of surface deformation of the given system under the applied load. Comparisons were generally made between the magnitudes of surface deformation for unreinforced and reinforced pavements. Finite element modeling of flexible pavements was conducted by Perkins (2001) using representative sections such as that shown in Figure 23a. The numerical results indicated a reduction in lateral strain at the bottom of the base and a reduction in shear at the top of the subgrade due to the presence of the reinforcement (Figure 23b).

The Discrete Element Method (DEM) has also been recently used to model soil-geosynthetic interaction, with particular emphasis on assessment of the interlocking of geogrid with base course material. This method is expected to capture the interaction between geogrid and soil in terms of load transfer mechanism and deformation behavior. Pullout test results were simulated by Konietzky *et al.* (2004) to model interlocking effect of geogrids under static and cyclic loading. Also, McDowell *et al.* (2006) simulated the use of biaxial geogrids to determine the optimum ratio of geogrid aperture size to soil particle size (Figure 24).

DEM modeling of a low volume road reinforced with geogrids was conducted by Kwon *et al.* (2008). The results indicated that the use of geogrids led to locked-in stresses during placement, compaction, and in-service loading, which resulted in a stiffer soil layer above the geogrid. However, additional research is still required to establish a correlation between the results of DEM pullout test simulations and actual field performance.

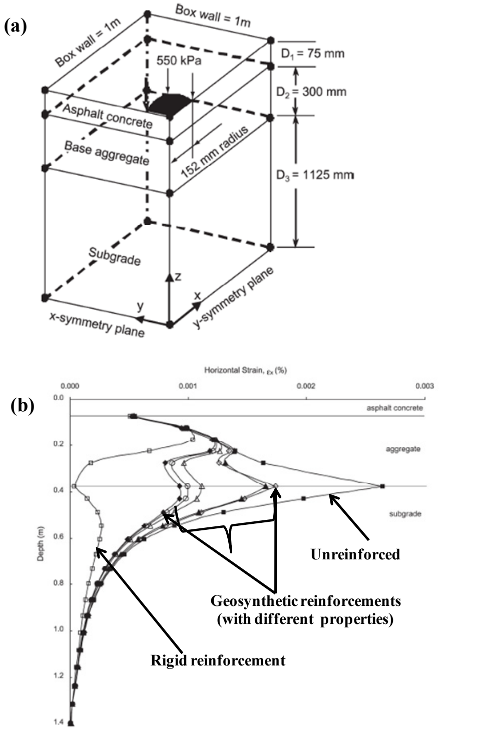

Figure 23: Flexible pavement (a) Finite element model (b) Horizontal stress vs. strain profile for various cases (Perkins and Edens 2002)

Table 2 FEM studies for geosynthetic-reinforced flexible pavement design (adapted from Perkins 2001)

References	Type of analysis	Geosynthetic constitutive model	Geosynthetic element type	Interface element type	Load type	Validation
Burd and Houlsby (1986)	Plane strain	Isotropic linear-elastic	Membrane	None	Monotonic	None
Barksdale *et al.* (1989)	Axi-symmetric	Isotropic linear elastic	Membrane	Linear elastic perfectly plastic	Monotonic	Field results
Burd and Brocklehurst (1990)	Plane strain	Isotropic linear- elastic	Membrane	None	Monotonic	None
Miura *et al.* (1990)	Axi-symmetric	Isotropic linear elastic	Truss	Linear elastic joint element	Monotonic	Field results
Dondi (1994)	Three-dimensional	Isotropic linear- elastic	Membrane	Elasto plastic Mohr-Coulomb	Monotonic	None
Wathugala *et al.* (1996)	Axi-symmetric	Isotropic elasto-plastic	Solid Continuum	None	Single cycle	None
Perkins (2001)	Three-dimensional	Anisotropic elasto-plastic	Membrane	Mohr-Coulomb	Multiple cycles	Lab and test tracks
Kwon *et al.* (2005)	Axi-symmetric	Isotropic linear-elastic	Membrane	Linear-elastic	Monotonic	Test tracks

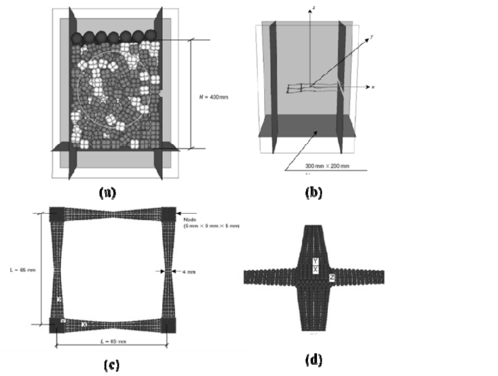

Figure 24: Discrete element model (a) pullout test with cubic clumps (b) pullout test with embedded geogrid (c) aperture of geogrid (d) detailed view at node (McDowell et al. 2006)

The use of results from FEM simulations has been recently proposed in combination with M-E design of geosynthetic-reinforced pavements. However, the developed models use geosynthetics as a structural element directly included within the materials but without simulating the interface conditions. Also, due to the significant computational effort required to simulate realistic traffic loads, simulations have often been conducted using a single static load cycle. These simplifications have led to results that may underestimate the benefit of geosynthetic reinforcement when compared with the field performance (Perkins and Ismeik 1997). To account for this deficiency, finite element simulations have often used increased values of geosynthetic stiffness. These stiffness values have been over an order of magnitude greater than those obtained from laboratory tests. The use of results from DEM simulations has been recently suggested as an alternative to represent better the interaction characteristics of soil-geosynthetic interface. The results obtained using this approach account for geosynthetic mechanisms such as the locked-in stresses experienced during the service life of pavement. The results have indicated that addition of geosynthetic-reinforcement to the pavement structure leads to an overall stiffening of the base course layer.

Mechanistic response models have been proposed by Perkins and Svanø (2004) to account separately for compaction and traffic loading stages in pavement design. These models represent rational but complex ways of incorporating the benefit of geogrid reinforcement and evaluating their performance in base-reinforced pavements (Kwon *et al.* 2008). Studies are under way in order to improve the use of FEM- and DEM-based models for implementation into M-E design methods.

5 CONCLUSIONS

The results of field, laboratory and numerical studies have demonstrated the benefits of using geosynthetics to improve the performance of pavements. However, selection criteria for geosynthetics to be used in reinforced pavements are not well established yet. The purpose of this paper was to summarize information generated so far in North America to quantify the improvement of geosynthetics when used as reinforcement inclusions in flexible pavement projects.

Previous research has led to a reasonably good understanding of the benefits achieved with the use of geosynthetics in pavement design but, for the most part, only from the empirical point of view. That is, while methods have been developed for designing geosynthetic-reinforced flexible pavements, quantification of the reinforcement mechanisms, identification of properties governing the pavement performance and, ultimately, acceptable design guidelines are yet unavailable.

Efforts are currently under way in the North America to develop design models consistent with the AASHTO and M-E approaches. The TBR and BCR ratios have been used in the AASHTO approach but are limited because the approaches are specific to the products and test conditions under which these ratios have been calibrated. Thus, M-E methods are considered more generic and, consequently, more promising as framework to incorporate the use of geosynthetics in current pavement design. However, due to the complex nature of flexible pavements, research to identify and quantify the properties governing the performance of reinforced pavements and its incorporation into M-E design is still under way.

The available literature involving field and laboratory test results is conclusive in that the mechanical properties of the geosynthetics used for pavement applications are improved under the confinement provided by the soil. Field test sections showed improved performance in the reinforced sections over the unreinforced sections in terms of reduced surface deflections. Overall, available experimental evidence indicates that the improved performance of geosynthetic-reinforced pavements can be attributed to lateral restraint mechanisms. Attempts have been made to quantify the lateral restraint in terms of the interface shear stiffness property of the soil-geosynthetic system.

A number of confined laboratory tests have been recently developed with the objective of quantifying the interface shear stiffness of the soil-geosynthetic system. Several of these tests have applied cyclic loads to the soil-geosynthetic system in an attempt to simulate the dynamic nature of traffic-induced loading. However, probably due to the fact that measurements are sensitive to small changes in displacements, currently available methods have resulted in significant scatter in test results. This has compromised the repeatability of the approaches and has made it difficult to differentiate the performance among different geosynthetics. Ongoing research focusing on confined testing under low displacements using monotonic loading pullout stiffness test appears promising to quantify relevant mechanisms in pavement reinforcement design.

Overall, it may be concluded that significant advances have been made in the area of geosynthetic reinforcement of pavements. While the state of practice is rapidly improving, further research is still needed to provide a better theoretical basis to the currently available empirical design approaches.

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